DESIGN EXAMPLES FOR STEEL BOX GIRDERS

U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
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Final Report

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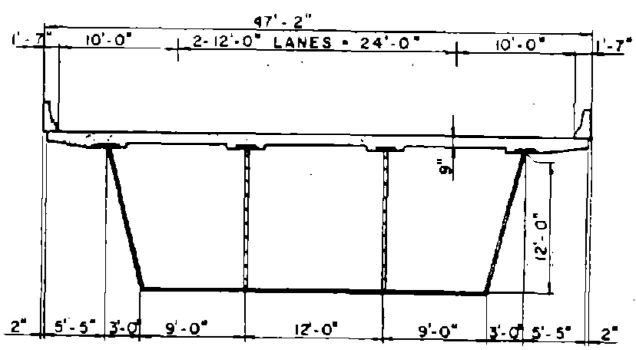

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16. Abstract (Limit: 200 words)

The Proposed Design Specifications for Steel Box Girder Bridges as contained in Report No. FHWA-TS-80-205 are evaluated. The results of comparative designs done using the AASHTO code and the proposed specification are summarized. The differences in the designs are explained with reference to the differing design requirements of the two specifications. The practicality and ease of application of the proposed specification are discussed. The results of parametric studies done to investigate the application of the proposed specification to the design of principal elements of box girders are included. The conclusions and recommendations based on the evaluation are also included. Appendix A contains comparative design examples contrasting the use of the AASHTO code with the proposed specification. Appendix B contains a discussion of areas of the proposed specification that could benefit from additional clarification or comment.

17. Document Analysis a. Descriptors

Steel Box Girders	Flanges
Bridges	Webs
Specifications	Design

b. Identifiers/Open-Ended Terms

c. COSATI Field/Group

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Foreword

This Tech Share report documents the results of an evaluation of the Proposed Design Specifications for Steel Box Girders as presented in Report No. FHWA-TS-80-205. The results of comparative designs done using the AASHTO code and the proposed specification are summarized. The differences in the designs are explained with reference to the differing design requirements of the two specifications. The AASHTO Subcommittee on Bridges and Structures has voted to insert a reference to the proposed specifications in the 1986 interim AASHTO specifications, Section 10.51.

Copies of this report are being distributed to FHWA Region and Division offices and to each State highway agency. Additional copies of this report and copies of FHWA-TS-80-205 can be obtained by public agencies from the FHWA R,D&T Report Center, HRD-11, McLean, Virginia 22101 and by other interested parties from the National Technical Information Service, Port Royal Road, Springfield, Virginia 22161.

Ronald E. Heinz
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Engineering

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Implementation

NOTICE

PREFACE

The Proposed Design Specifications for Steel Box Girder Bridges, Report No. FHWA-TS-80-205, dated January 1980, grew out of the engineering profession's need for a set of design rules reflecting the current state-of-the-art. Following publication of this report, the AASHTO Technical Subcommittee on Steel Design recommended that additional study be done to assess the validity of the proposed specifications.

In April 1981, the American Iron and Steel Institute (AISI) contracted with Sverdrup & Parcel (S&P) to begin an initial study of the proposed specifications. This study, which was sponsored by AISI's Bridge Task Force Committee under its Project 319, consisted of designing the principal elements of a 3-span continuous multi-box composite bridge with a 150-ft main span and a 3-span continuous single box multi-cell bridge with orthotropic deck and main span of 650 ft. Each bridge was designed using the proposed specifications and the current AASHTO Specifications.

Subsequently, the Federal Highway Administration responded to a request to sponsor additional studies and on August 27, 1982 entered into Contract No. DTFH61-82-C-00034 with the AISI for these additional studies. AISI in turn subcontracted the work to S&P.

This work was divided into the following tasks:

Task A - Design the principal elements of a 3-span continuous single box multi-cell box girder bridge with a composite concrete deck, using the current AASHTO and proposed specifications.

Task B - Compare the opposing designs prepared under Task A and the initial study.

Task C - Evaluate the application of the current AASHTO and proposed specifications with regard to the practicality and ease of the application of the proposed specifications.

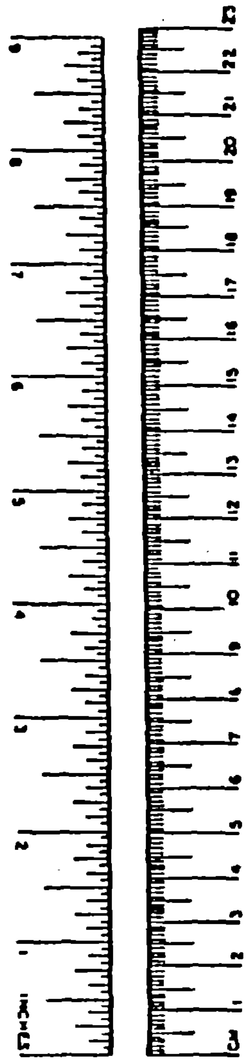
Task D - Conduct a parametric study of the application of the proposed specifications to the principal elements of steel box girders.

This report presents the results of the findings of Tasks A through D, together with conclusions and recommendations. In addition, comparative design examples are presented in Appendix A.

METRIC CONVERSION FACTORS

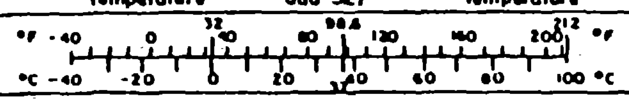
APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
	LENGTH			
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
	AREA			
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.6	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
	MASS (weight)			
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
	VOLUME			
tsp	teaspoons	5	milliliters	ml
tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
	LENGTH			
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
	AREA			
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
	MASS (weight)			
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
	VOLUME			
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	36	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



ABBREVIATIONS

abt	about
bott	bottom
brg	bearing
btwn	between
conn	connection
eq	equal
ext	exterior
flg	flange
ft	foot or feet
in	inch
int	interior
ksi	kips per square inch
lb	pound
longit	longitudinal
pl	plate
psi	pounds per square inch
seg	segment
spa	spaces or spacing
sq	square
stiff	stiffener
symm	symmetric
trans	transverse
typ	typical
wt	weight

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APPENDICES

A. INTRODUCTION

A.1. BACKGROUND

The Proposed Design Specifications for Steel Box Girder Bridges, as published in Report No. FHWA-TS-80-205, hereinafter referred to as the proposed specifications, reflect the engineering profession's need for a set of design rules which are applicable to long span steel box girder bridges of composite or orthotropic design. This need was identified through previous work of the ASCE-AASHTO Subcommittee on Box Girders in the early '70's. At that time, parallel studies and developments in several other countries were evaluated and found to be either inappropriate and/or underdeveloped for use as a practical specification. For instance, the Merrison Rules[1] were found to be complex and unwieldly for use.

In response to this need, the U.S. Department of Transportation, Federal Highway Administration, sponsored a study to develop a set of design rules which were practical and easy to use and would result in a safe, economical steel superstructure. The rules were to be written in the format of the AASHTO Standard Specifications for Highway Bridges and were to be based on Load Factor Design. The project was completed in June, 1979 and the new design rules were submitted to the AASHTO Subcommittee on Bridges and Structures for consideration and adoption.

The proposed specifications, intended as an addition to the AASHTO Highway Bridge Specifications, are applicable for box girder bridges, regardless of span, and their use for the design of short to moderate-span box girder bridges, instead of the current AASHTO rules, would be optional. Special provisions necessary for horizontally curved girder bridges, girders with haunches, skewed girder bridges, cable-stayed girder bridges or girder-stiffened suspension bridges are not included in the proposed specifications.

The AASHTO Technical Subcommittee on Steel Design recommended that additional study be done in order to assess the validity of the proposed specifications. Preparation of typical bridge designs made with these specifications was necessary. Comparison of bridge designs made with the current AASHTO Specifications and with the new specifications was required to highlight differences in the two codes. These comparisons were recommended for spans of moderate length as well as for long spans. In addition, parametric studies, to further evaluate the practicality of the proposed specifications, were recommended.

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1 Inquiry into the Basis of Design and Method of Erection of Steel Box Girder Bridges, Report of the Committee - Appendix I, Interim Design and Workmanship Rules, H. M. Stationary Office, London, 1973, 1974.

A.2. PURPOSE

The purpose of this study is to investigate the practical effect and impact that the usage of the proposed specifications will have upon the designer, the design, and the steel fabricator. For this purpose, comparative designs were done for three bridges with different cross sections and span lengths. The main spans varied from 150 to 650 ft. Parametric studies were also done to evaluate the effect of various input parameters on practical designs.

B. SPECIFICATIONS APPLICABLE FOR THIS COMPARISON

B.1. PROPOSED DESIGN SPECIFICATIONS FOR STEEL BOX GIRDER BRIDGES

As previously mentioned, these specifications are contained in Report No. FHWA-TS-80-205 and contain specific provisions for most types of box girder bridges. This specification is based on Load Factor Design.

New methods of determining design strengths are given for unstiffened and longitudinally stiffened bottom flanges in compression. The new strength values are affected by geometric imperfections, residual stresses and out-of-straightness of the longitudinal stiffeners.

The elastic buckling and postbuckling contributions to the web strength are computed independent of the flange properties. The postbuckling strength is determined by utilizing a lower bound for the tension field strength corresponding to the "true Basler" solution which assumes negligible flange rigidity.[2] The combined effect of shear and axial stresses are considered in the determination of both elastic and postbuckling strengths.

The webs are designed on the basis that the individual panels or sub-panels are rigidly supported around their periphery. Therefore, the web stiffeners are designed to remain straight up to the point at which the web ultimate strength is reached.

B.2. AASHTO SPECIFICATIONS

These specifications are the 1977 Twelfth Edition adopted by "The American Association of State Highway and Transportation Officials" and interim specifications dated 1978, 1979, 1980, 1981 and 1982. These specifications contain only provisions for composite multi-box girder bridges of moderate span subject to geometric limitations. The Load Factor Design method was used for all designs and is the basis of comparison with the proposed specifications.

Transverse distribution of live loads as given by the simple AASHTO distribution equation was used for the analysis of the short span structures only. The intermediate and long span structures were analyzed in a manner consistent with the proposed specifications.

As presented in the AASHTO code, the design of unstiffened and longitudinally stiffened bottom flanges in compression is based on the analogy between column and plate buckling.

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2 Fujii, T., "On an Improved Theory for Dr. Basler's Theory," Final Report of the Eighth Congress of I.A.B.S.E., New York, September, 1968.

The design of webs is based on tension field strength according to Basler.[3,4] The interaction of shear and flexure is considered indirectly in AASHTO by an empirical formula. Longitudinal stiffening is limited to one stiffener only.

B.3. COMPARISON OF SPECIFICATIONS

The design examples are a reflection of the difference between the proposed specification and the present AASHTO; however, a direct comparison of the codes will render a better understanding of the differences. In this section, the code differences and their effects upon the design are presented. Reference will be made to this discussion in the later section dealing with the comparison of the designs.

B.3.1. Strength of Unstiffened and Stiffened Plate Panels

(a) Unstiffened Flanges - AASHTO Code

Under the current AASHTO code, stocky panels may be designed for the yield stress.

$$\text{when } b/t \leq \frac{6140}{\sqrt{F_y}}; \qquad F_{cr} = F_y$$

where b = flange width between webs
t = flange thickness
F_{cr} = critical buckling stress
F_y = yield strength of steel

For slender panels, the Euler equation is used.

$$\text{when } b/t \geq \frac{13,300}{\sqrt{F_y}}; \quad F_{cr} = K \frac{\pi^2 E}{12\,(1 - \nu^2)\,(b/t)^2}$$

with $K = 4.0$, $\nu = 0.3$, and $E = 29 \times 10^6$ psi,

$$F_{cr} = 105\,(t/b)^2 \times 10^6 \text{ (AASHTO Article 1.7.64(E)(3))}$$

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3 Basler, K. "Strength of Plate Girders in Shear," Journal of the Structural Division, ASCE, Vol. 87, No. St7, Oct., 1961, pp 151-180.

4 Basler, K., "Strength of Plate Girders in Combined Bending and Shear," Journal of the Structural Division, ASCE, Vol. 87, No. St7, Oct., 1961, pp. 181-197.

For intermediate values of b/t, a transition curve is used.

$$\frac{6140}{\sqrt{F_y}} < \frac{b}{t} \leq \frac{13,300}{\sqrt{F_y}};\ F_{cr} = 0.592\ F_y\ (1 + 0.687 \sin \frac{c\pi}{2})$$

Where c is as defined in the AASHTO code.

(b) Unstiffened Flanges - Proposed Specifications

The proposed specification includes the effects of residual stresses as well as geometric imperfections in determining the strength of unstiffened flanges. The strength curve for the new code is shown on Figure 1. For easy comparison, the current AASHTO curve has been shown by a dashed line. It can be seen that the AASHTO curve yields a higher value of F_u for $32 < b/t \leq 70$ (F_y = 36 ksi). It should be emphasized that the proposed specification, by including certain material effects in the allowable strength, contains a "material resistance" factor. Since material factors are included in the current AASHTO load factors, if the proposed specification is made a part of the AASHTO specifications, the AASHTO load factors may need to be reviewed. However, the conservatism introduced by the proposed specification does not appear to be very significant as the effect is confined to the highly stressed compression zone only (see Conclusions and Recommendations).

(c) Stiffened Flanges

The stiffened flange strength as computed by the proposed specification likewise yields a lower strength than an "extended" AASHTO version. Due to the complexity of parameters, a direct comparison was not attempted. For manual computations, the strength curves as presented in Figure 1.7.206(A) of the proposed specification are convenient. However, for computer design, the strength-slenderness parameter relationship must be given in a more direct and usable expression.

B.3.2. Web Panels

The shear strength is given as the sum of beam shear strength, V_B, and the tension field strength (postbuckling strength), V_T.

The AASHTO Specifications utilize the Basler solution for the web design. The Basler solution can be stated as follows:

$$V_u = \tau_{cr}\ D\ t_w + F_y\ D\ t_w \frac{(1 - \tau_{cr}/\ \tau_y)}{2\sqrt{(1 + \alpha^2)}}$$

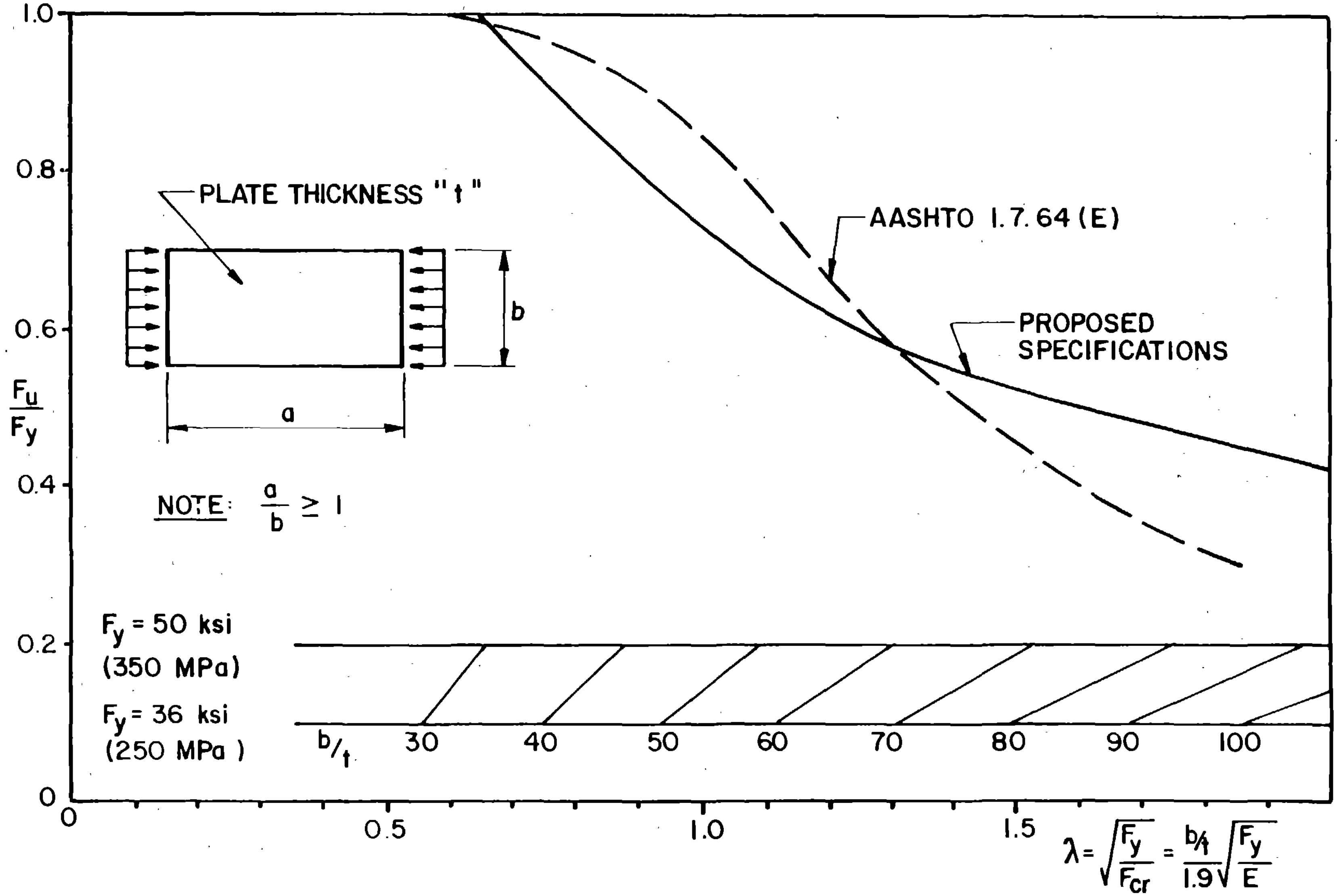

FIGURE I. COMPRESSION STRENGTH OF UNSTIFFENED PLATE PANEL

where V_u = maximum shear capacity
D = depth of web between flanges
d_o = distance between transverse stiffeners
α = d_o/D (aspect ratio)
t_w = web thickness
F_y = yield stress of web
τ_{cr} = critical buckling stress of web
τ_y = yield stress in shear of web

Utilizing the following relationships,

$$F_y/\tau_y \cong \sqrt{3} \text{ and } C = \tau_{cr}/\tau_y$$

where $$C = \frac{18{,}000}{\sqrt{F_y}} \times \frac{t}{D} \times \frac{1}{\cos\phi} - 0.3 \leq 1.0$$

ϕ = the angle of the panel diagonal with the horizontal;

the above equation takes on the form:

$$V_u = \tau_y D t_w C + \tau_y D t_w \frac{\sqrt{3}}{2} \frac{1 - C}{\sqrt{1 + \alpha^2}}$$

which is the same as AASHTO 1.7.59(E)(3),

$$V_u = V_p \left[C + \frac{0.87\ (1 - C)}{\sqrt{1 + (d_o/D)^2}} \right]$$

The proposed specification utilizes the "true Basler" solution, as given below:

$$V_u = \tau_{cr} D t_w + F_y D t_w \frac{(1 - \tau_{cr}/\tau_y)}{2\ (\sqrt{1 + \alpha^2} + \alpha)}$$

Using the approximate von Mises yield condition,

$$F_T = F_y \left(1 - \frac{\tau_{cr}}{\tau_y}\right)$$

where F_T = tension field stress

gives

$$V_u = V_B + V_T$$

where $V_B = \tau_{cr} D t_w$ and $V_T = F_T D t_w/(2\ (\alpha + \sqrt{1 + \alpha^2}))$.

Here, V_B is due to beam action of the web and V_T is due to the tension field action. The equations above are as they appear in the proposed specification.

Comparing the second part (tension field action) in the two codes, one will notice that the only difference is in the denominators, where

$De_{AASHTO} = \sqrt{1 + \alpha^2}$ and $De_{proposed} = \sqrt{1 + \alpha^2} + \alpha$. Since these values depend only on the aspect ratio α and since the tension field action directly depends on these denominators, the ratio

$$R = \frac{V_T \text{ AASHTO}}{V_T \text{ proposed}} = \frac{\sqrt{1 + \alpha^2} + \alpha}{\sqrt{1 + \alpha^2}}$$

gives an excellent comparison between the codes, see Table 1.

TABLE 1. COMPARISON OF TENSION FIELD STRENGTHS

α	0.5	0.75	1.0	1.25	1.5
R	1.45	1.6	1.71	1.78	1.83

As can be seen, the tension field action according to AASHTO is between 45% to 83% larger for aspect ratios between 0.5 and 1.5, respectively.

Comparing the first part, V_B (beam action), the two codes seem to be identical; however, there is a great difference in the evaluation of τ_{cr}. While AASHTO uses a theoretical value for C which only depends on the panel geometry and the yield stress of the web material, the proposed code finds the critical buckling shear stress by means of an interaction formula, taking into account the combined effects of shear, flexure, and axial forces.

It is obvious that determining the governing case using the proposed interaction formula requires additional effort on the part of the designer.

The results from our designs show that generally, for stiffened panels, the V_B is slightly increased when using the proposed specification; however, V_T is heavily decreased as expected, resulting in an overall lower V_U value which is reflected in the increase of web and/or stiffener material. The approximate total effect on V_u when using the proposed specification can be found for any panel. Table 3 is a summary of web studies based on the proposed specifications. By using the fraction of V_T utilized shown in the last column and the above ratios from Table 1, the V_T of the panels can be found for both the proposed specification and AASHTO. By assuming that V_B is the same by both codes, then the values of V_u can be determined. For the panels of Table 3, the proposed specification results in a reduction of V_T of 35 percent and in V_u of 24 percent.

Recent experimental work on plate girder webs has confirmed the conservatism of the proposed specification.[5] The reference cited reports on the testing of eight web panels. The mean of the panel strengths as determined by the proposed specification was 76 percent of the actual ultimate capacity determined by the tests. The Basler solution upon which the AASHTO code is based resulted in a mean strength of 104 percent of the ultimate capacity. The authors of the report concluded that the Basler solution with a 0.92 reduction factor would result in adequate and conservative web design. Tests to date have indicated that box girder flanges are sufficiently rigid to allow the development of significant amounts of tension field action. However, the authors also emphasized that additional tests will be required to verify the validity of using the web strength of plate girders to predict the web strength of box girders.

For unstiffened webs, the proposed specification bases the carrying capacity of the web on the buckling shear, V_B, and postbuckling strength is disregarded. The buckling stress curve for the proposed specification is shown in Figure 2. Included for comparison is the applicable curve from the AASHTO specification. Note that the proposed specification curve is based on shear acting alone and that the value of F^{o}_{vcr} will be reduced when the coinciding moments are considered.

B.3.3. Web Stiffeners

AASHTO requirements for longitudinal and transverse stiffeners require that the stiffeners be rigid enough to preserve the straight boundaries assumed when computing shear buckling of the webs. Numerical data from Bleich[6] for transverse stiffeners yields the following form.

$$I = 2.5\ D t_w^{3} \left[\frac{D}{d_o} - 0.7 \frac{d_o}{D} \right]$$

If the second term constant is set to 0.8 and introducing

$$J = 2.5\ (D/d_o)^2 - 2$$

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5 Cooke, N., Moss, P. J., Walpole, W. R., Langdon, D. W., Harvey, M. H., "Strength and Serviceability of Steel Girder Webs," Journal of Structural Engineering, ASCE, Vol. 109, No. 3, March, 1983, pp. 785-807.

6 Bleich, F., Buckling Strength of Metal Structures, McGraw-Hill Book Co., Inc., New York, 1952.

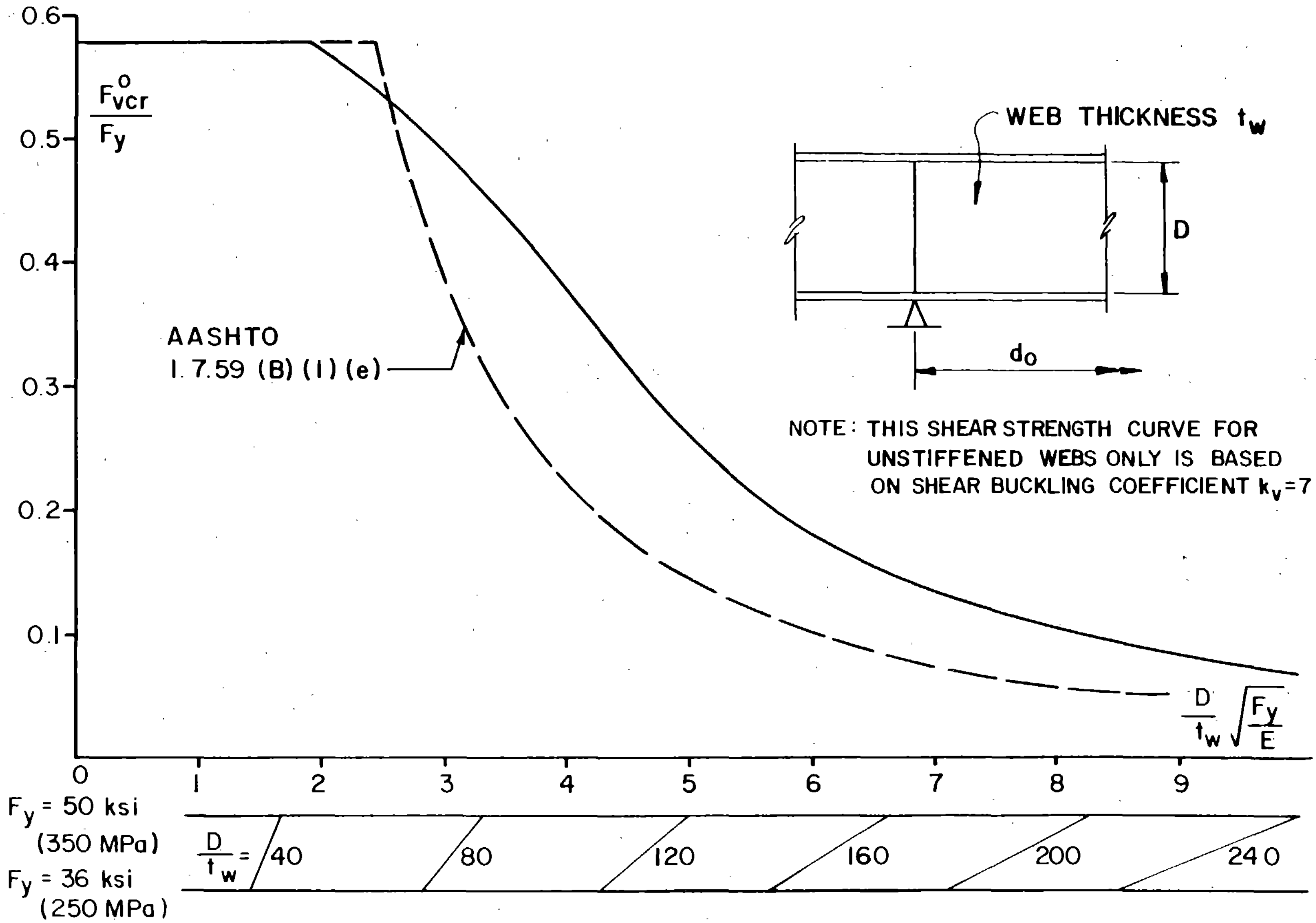

FIGURE 2. CRITICAL BUCKLING STRESS OF UNSTIFFENED WEB DUE TO SHEAR ACTING ALONE

the resulting requirement for I is:

$$I = d_o\, t_w^{\,3} J \qquad \text{AASHTO 1.7.59(E)(5)}$$

where I = moment of inertia of transverse stiffener with reference to midplane of web

and other nomenclature is as previously defined.

When longitudinal stiffeners are used, the transverse stiffener must support them. According to P. B. Cooper:[7]

$$S_T = S_L\, D/d_o$$

AASHTO reduced this requirement drastically to:

$$S_T = (1/3)\, S_L\, D/d_o \qquad \text{AASHTO 1.7.59(F)(3)}$$

where S_T = section modulus of transverse stiffener

S_L = section modulus of longitudinal stiffener

The rigidity requirement of the longitudinal stiffener is similar to the transverse:

$$I \geq D\, t_w^{\,3}\, [2.4\left(\frac{d_o}{D}\right)^2 - 0.13]$$

If the tension field strength is utilized, the vertical component of this force has to be carried by the transverse stiffener. The required area is:

$$A_s = [0.15DBt_w\,(1 - C)\,(V/V_u) - 18t_w^{\,2}]\; Y \qquad \text{AASHTO 1.7.59(E)(5)}$$

where B = correction factor for one-sided stiffeners
V = shear force on the cross section
Y = yield strength ratio, web/stiffener

The above stiffener requirements are all easy to apply. All numbers are either chosen sizes or known values (B, C, V, V_u, Y) and their implementation consumes only a minimal effort.

The proposed specification has similar rigidity and strength requirements for transverse stiffeners. Based on requirements of straight boundaries, the code stipulates, conservatively, that the relative rigidity coefficient of a stiffener should be at least equal to

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[7] Cooper, P. B., "Strength of Longitudinally Stiffened Plate Girders," Journal of the Structural Division, ASCE, Vol. 93, No. St2, April, 1967, pp. 419-451.

the "theoretical optimum rigidity", γ^*, multiplied by an empirical factor, m_T.

$$I \geq 0.09\, m_T\, \gamma^*\, Dt_w^{\,3}\, \frac{f'_v}{F^o_{vcr}}$$

The ratio f'_v/F^o_{vcr} indicates the extent to which the shear capacity of the web is utilized.

As in AASHTO, a strength requirement subject to the tension field vertical force, P_{VT}, is specified.

$$P_{VT} = \frac{F_T t_w d_o}{2}\left[1 - \frac{\alpha}{\sqrt{1+\alpha^2}}\right]\frac{V - V_B}{V_T}$$

Without a closer examination, these requirements, which seem to be similar, are quite different in design effort. It is a simple task to find a stiffener which satisfies an I and A requirement. It becomes a larger task when the stiffener moment of inertia must be computed, including an effective width of web. Additionally, checking a trial stiffener as a column for a given force is more involved than simply meeting an area requirement.

Similarly, under the proposed specifications, the longitudinal stiffener has a rigidity requirement.

$$I_L \geq 0.09\, nm_L\, \gamma^*_{L(\sigma+\tau)}\, Dt_w^{\,3}$$

n, m_L = multipliers, dependent on the number of longitudinal stiffeners and the web D/t_w ratio

where

$$\gamma^*_{L(\sigma+\tau)} = \sqrt{\left[\gamma^*_{L\sigma}\left(\frac{f'_c}{F^o_{ccr}} + \frac{f'_b}{F^o_{bcr}}\right)\right]^2 + \left(\gamma^*_{L\tau}\,\frac{f'_v}{F^o_{vcr}}\right)^2}$$

All stresses are those for the governing adjacent sub-panel and are fully defined in the proposed specification.

The values of $\gamma^*_{L\sigma}$, $\gamma^*_{L\tau}$ are determined from graphs in Figure 1.7.213(B) of the proposed specifications, where 12 curves are shown which make interpolation (between $0.7 < \alpha \leq 1.5$) difficult. All rigidity coefficients should be given in an explicit form for computer design. Additionally, the longitudinal stiffener is checked as a column with the force and location of application depending on whether or not the stiffener is continuous or discontinuous. The amount of work spent on stiffener sizing is comparable to that required for flange design or the design of the web panel.

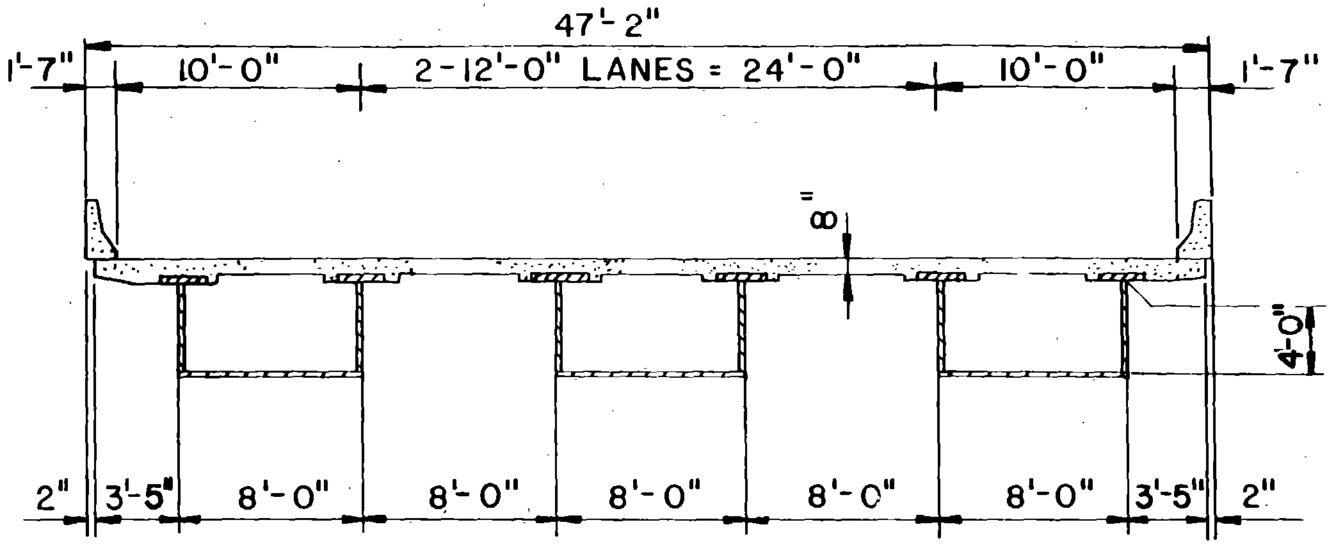
C. DESCRIPTION OF THE THREE DESIGNS SELECTED FOR COMPARISON OF CODES

To insure that the evaluation of the proposed specification would be for a range of structure lengths, our work assignment called for the design of 3-span continuous box girders with main-span lengths of 150, 400, and 650 ft.

The short span bridge had a span arrangement of 115-150-115 ft. The bridge was a 2-lane bridge with 10 ft shoulders. The cross section (see Figure 3) shows a multi-box composite bridge. It consists of three single cell boxes where the geometry fulfills the AASHTO geometric requirements. The slab was set to an 8-inch thickness. The loads for parapets were assumed to be 410 lbs/ft and a future wearing surface of 25 lbs/sq. ft. was used. Live load was set to HS 20-44 and the concrete deck was assumed to have a 28-day strength of $f'_c = 4{,}000$ psi.

The intermediate span bridge had a span arrangement of 310-400-310 ft. For purposes of variation, this cross section was arranged as a multi-cell, single composite box with inclined webs. Loads and concrete specifications were the same as for the short span. It was our specific intention with our cross section choice to let this section fall outside the geometric confines of AASHTO Article 1.7.64. We could easily have modified the cross section to two single cells by removing the bottom flange between interior cells (see Figure 4). The AASHTO lateral live load distribution, therefore, was not applicable, and a refined analysis determining simultaneous bending and torsional moments and shear forces was employed. Since only minor section changes between codes result, the same analysis was applicable for both codes.

The long span box girder, due to its 650-ft main span (500 ft side spans), was assumed to have an orthotropic deck. It is a two-cell single box with variable depth. The cross section and pertinent data are shown on Figure 5. The deck itself was only designed in general for sizing purposes and was used for both designs unless different deck plate thicknesses were required when considering the deck plate and main bridge system to act integrally. Generally, all provisions of the AASHTO specification were considered applicable. The only code provision explicitly modified was that dealing with bottom flanges in compression, 1.7.64(E). This provision does not provide for transverse flange stiffeners. The approach taken in the design was to use the Service Load provision, 1.7.49(D)(4), extended for Load Factor Design with certain simplifying assumptions removed.

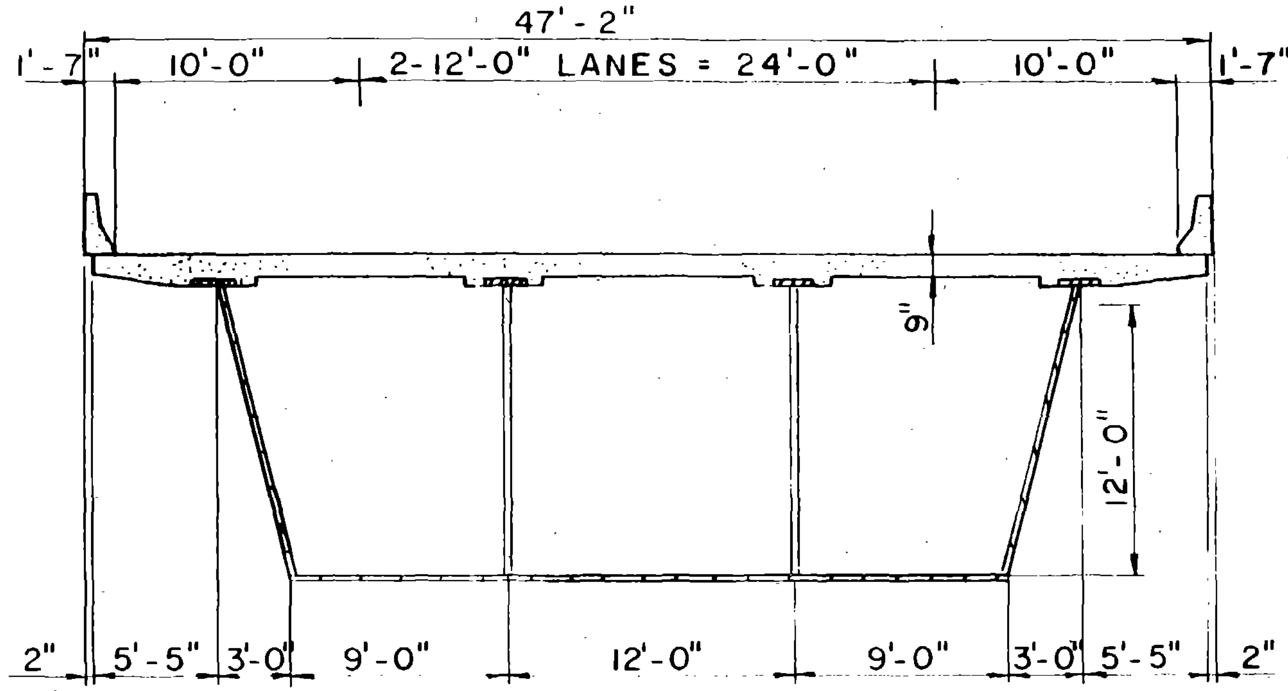


LOADS:	PARAPET	410 LBS. / FT.
	FUTURE WEARING SURFACE	25 LBS. / SQ. FT.

LIVE LOAD: HS 20-44

CONCRETE DECK: f'c = 4000 psi

FIGURE 3. TYPICAL SECTION – SHORT SPAN

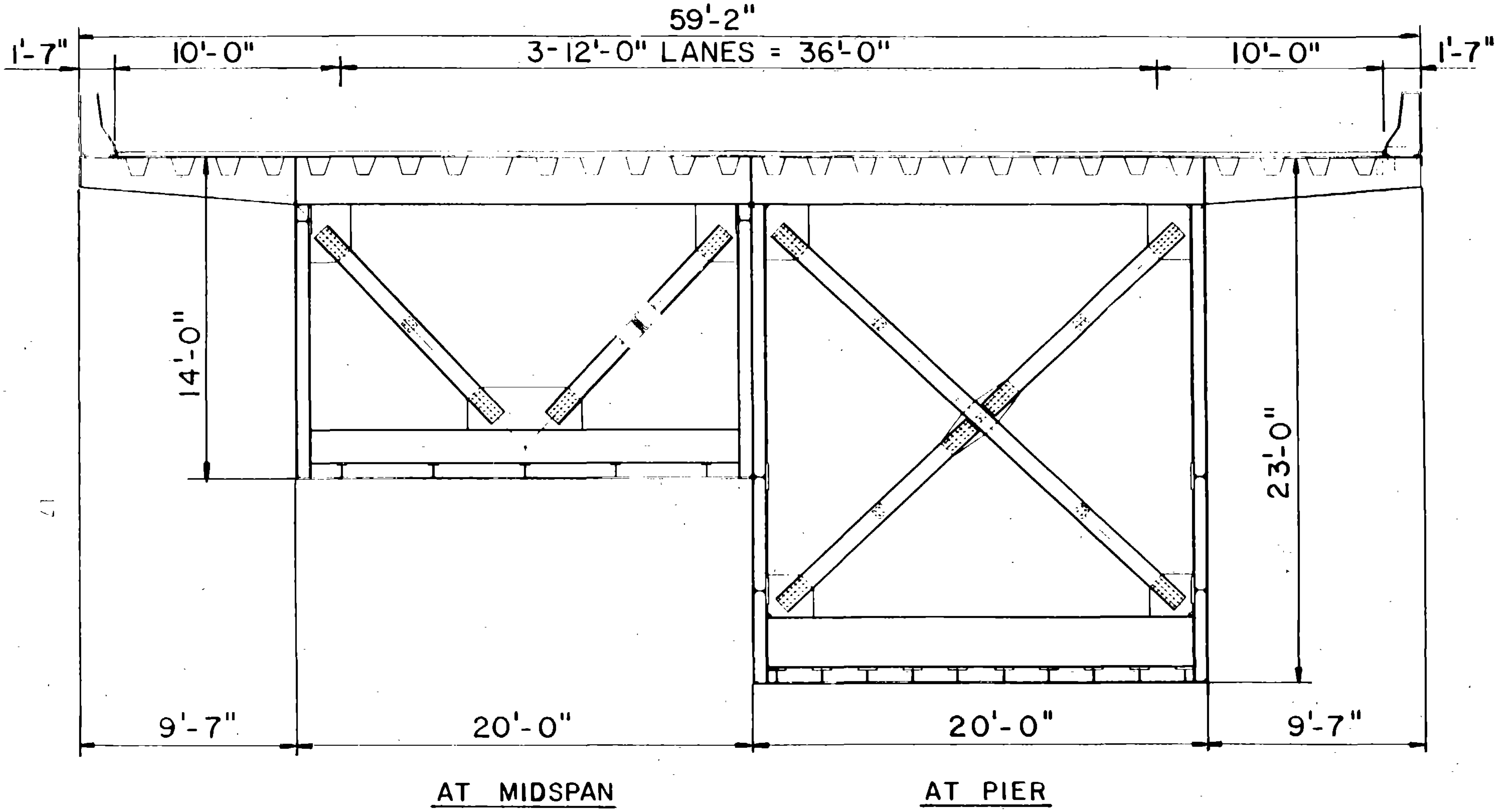


LOADS:	PARAPET	410 LBS. / FT.
	FUTURE WEARING SURFACE	25 LBS. / SQ. FT.

LIVE LOAD: HS 20-44

CONCRETE DECK: $f'c = 4000$ psi

FIGURE 4. TYPICAL SECTION – INTERMEDIATE SPAN



LOADS	PARAPET	150 LBS./FT
	WEARING SURFACE	24 LBS./SQ. FT.
LIVE LOAD	HS 20-44	

FIGURE 5. TYPICAL SECTION - LONG SPAN

D. FINAL RESULTS OF THE SIX DESIGNS, THEIR COMPARISON, CHANGES AND DEPARTURES FROM CURRENT SPECIFICATIONS AND PRACTICES

In the previous section, a short description of the three bridge studies was presented. Each of these bridges was designed using the AASHTO and the proposed specification. In Table 2, a weight comparison is given, showing weights in tons for half the structure. Differences refer to the AASHTO design as a base. Weights shown for top and bottom flanges include weights for flange plates and longitudinal and transverse stiffeners, where applicable. The short span designs used A36 material throughout. The designs for the intermediate and long span structures used Grade 50 material at supports and A36 elsewhere. The ratio of Grade 50 to A36 was similar for comparative designs.

It is realized that for a true comparison, fabrication costs of the designs should be considered. However, this study does not include an evaluation of economic factors, but merely points out the effects of the different requirements on the weight and arrangement of the principal elements.

D.1. SHORT SPAN COMPARISON

Typical cross sections for these designs are shown as Figures 6 and 7 and typical girder elevations are shown on Figures 12 and 13. Both designs follow the rules for lateral live load distribution in accordance with AASHTO Article 1.7.49(B). It can be seen by comparing the two designs that the only difference in the bottom flange is at the point of contraflexure where a 9/16" plate was substituted in the AASHTO design for the 1/2" plate used in the proposed specification design. However, directly over the support, the proposed specification required a 1/8" thicker plate due to the lower allowable in the compression flange. The increase in weight of the bottom flange, when using the proposed specification, was 11.9%. The reduction in the bottom flange at the point of contraflexure was caused by the thicker web plate (3/8-inch) required by the proposed specification as a minimum (1.7.210(D)). This requirement causes a 21.3% increase in web plate material. Due to the heavier bottom flange and web, the top flange yielded an 8.3% savings in the proximity of the support.

The heavier web required by the proposed specification resulted in a 49.4% savings in transverse stiffeners.

The net result, when considering all of the principal design elements, amounted to a 9.3% increase in steel weight when using the proposed specification.

TABLE 2. COMPARISON OF PRINCIPAL ELEMENT WEIGHTS BASED ON DESIGN BY THE AASHTO CODE AND THE PROPOSED SPECIFICATIONS

		SHORT SPAN (115 ft-150 ft-115 ft)			INTERMEDIATE SPAN (310 ft-400 ft-310 ft)			LONG SPAN (500 ft-650 ft-500 ft)		
		AASHTO CODE	PROPOSED SPEC.	% DIFF	AASHTO CODE	PROPOSED SPEC.	% DIFF	AASHTO CODE	PROPOSED SPEC.	% DIFF
BOTTOM FLANGE		54.5	61.0	11.9	260.8	264.6	1.5	732.7	741.4	1.2
WEB	PLATE	30.0	36.4	21.3	224.6	218.5	-2.7	431.3	406.8	-5.7
	LONGIT. STIFF.	-	-	-	10.2	18.6	82.4	17.4	28.9	66.1
	TRANS. STIFF.	1.7	0.86	-49.4	27.8	41.0	47.5	38.6	74.9	94.0
	PLATE + STIFF.	31.7	37.3	14.2	262.6	278.1	5.9	487.3	510.6	4.8
TOP FLANGE		22.9	21.0	-8.3	145.1	154.1	6.2	894.0	903.0	1.0
TOTAL-ALL ELEMENTS		109.1	119.3	9.3	668.5	696.8	4.2	2114.0	2155.0	1.9

NOTES:
1. Weights given are for one-half of structure.
2. Units are tons.
3. % DIFF uses AASHTO weight as base.
4. Both A36 and Grade 50 material are included in the weights shown.
5. Weights for top and bottom flanges include plate and longitudinal and transverse stiffeners where applicable.

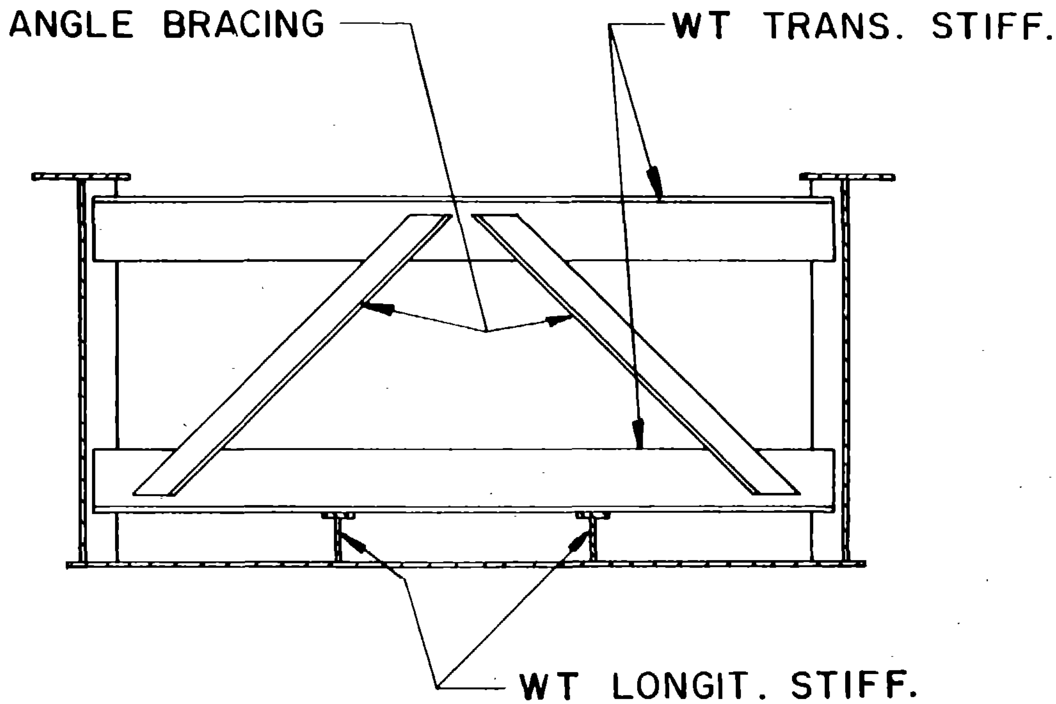


FIGURE 6. TYPICAL CROSS SECTION — AASHTO DESIGN
SHORT SPAN

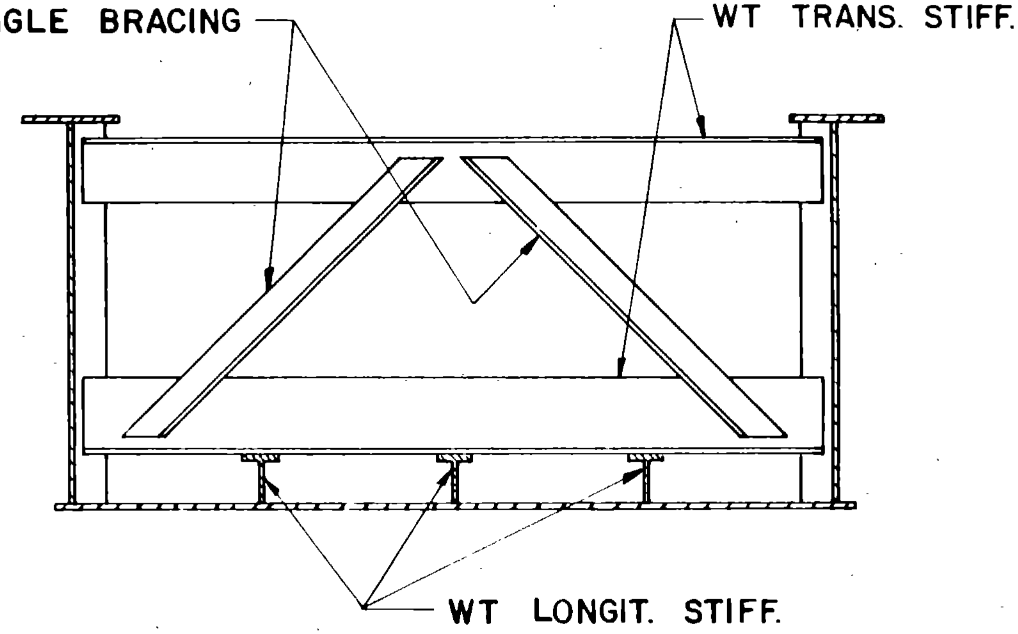


FIGURE 7. TYPICAL CROSS SECTION - PROPOSED SPECIFICATION SHORT SPAN

D.2. INTERMEDIATE SPAN COMPARISON

As discussed under Section C, the span length, basic geometry and loads were the same for both designs. The resulting design moments, shears, and torques were therefore identical for design under both codes. For comparative typical cross sections, see Figures 8, 9 and 10. Typical girder elevations are shown on Figures 14 and 15.

For the bottom flanges, both designs utilize the same plate thicknesses with approximately the same plate cut-offs. The differences in weight result from the differences in flange stiffener arrangement between the two designs.

D.2.1. Bottom Flange in Compression

As noted in the comparison of codes in Section B, the proposed specification for bottom flanges in compression causes a reduction in design critical stress as compared to AASHTO. In order to increase the critical stress, four methods exist:

1. Use thicker flange plate.
2. Reduce longitudinal stiffener spacing.
3. Reduce transverse stiffener spacing.
4. Use larger (stiffer) longitudinal stiffener.

Methods 1 and 2 are principally aimed at reducing the w/t ratio while 3 and 4 will reduce the L/r ratio. A study showed that the reduction of L/r was the most weight-efficient for this case. Hence, the method used was to reduce the transverse stiffener spacing to increase the critical stress to a value comparable to that for the AASHTO design. Thus, bottom flange plates remain unchanged in most instances, with transverse stiffeners making up the weight difference.

D.2.2. Bottom Flange in Tension

The major difference in the designs resulted from slenderness considerations. The AASHTO code has no explicit limits for slenderness of stiffened plates in tension. Values used were $w/t \leq 120$ and $L/r \leq 200$ (AASHTO 1.7.5). Section 1.7.208(G) of the proposed specification contains specific slenderness provisions which require the ratio w/t and L/r not to exceed 120. These code differences did not affect the total weight; however, the ratio of stiffener weight to plate weight changed, and this could influence fabrication costs. The total weight for bottom flanges (compression and tension) showed an increase of 1.5% under the proposed specification.

D.2.3. Web Plate

The web thickness used over the supports in the proposed design is 1/16" thinner than that in the AASHTO design. This is the

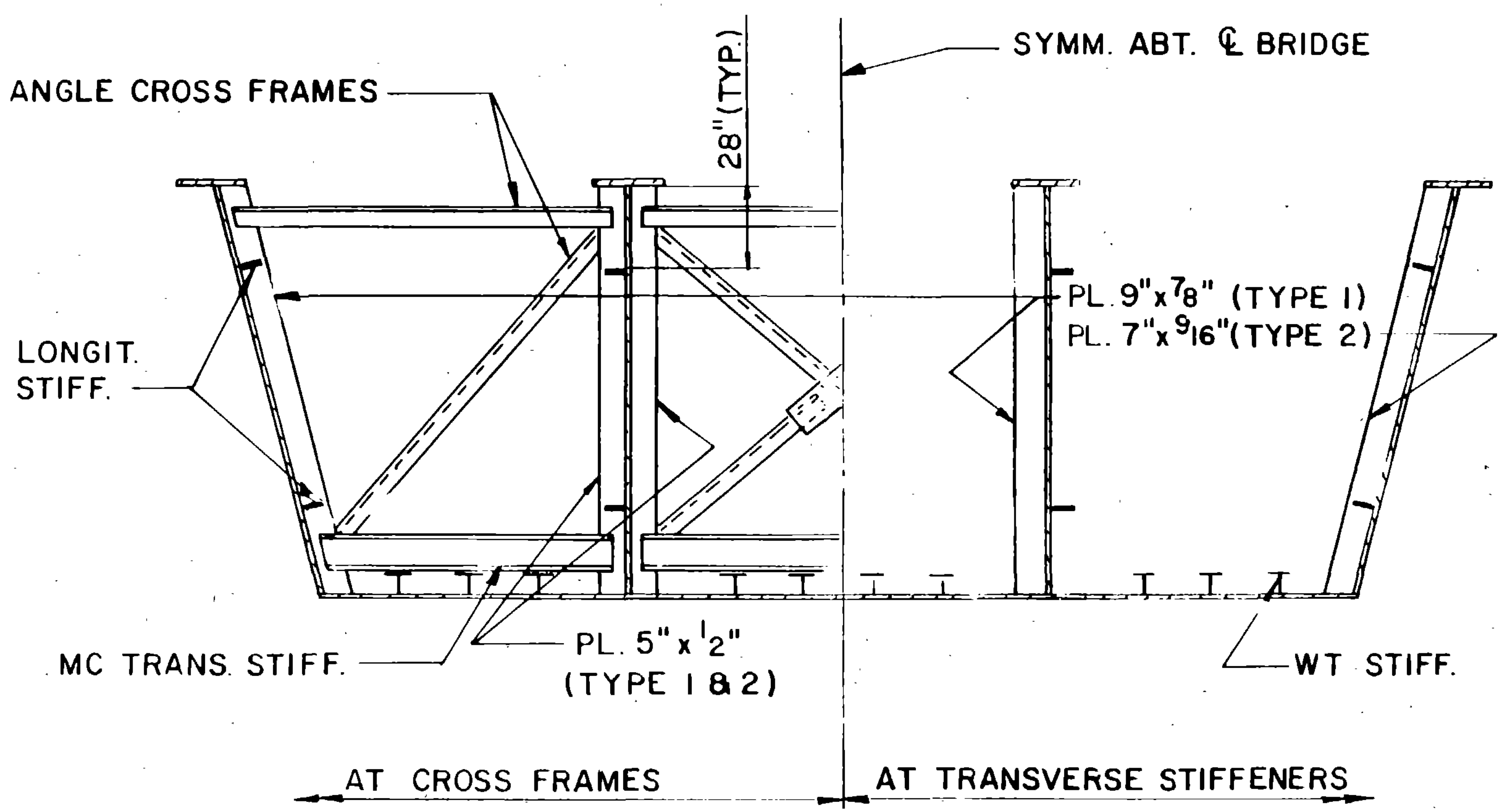


FIGURE 8. TYPICAL CROSS SECTION—AASHTO DESIGN INTERMEDIATE SPAN

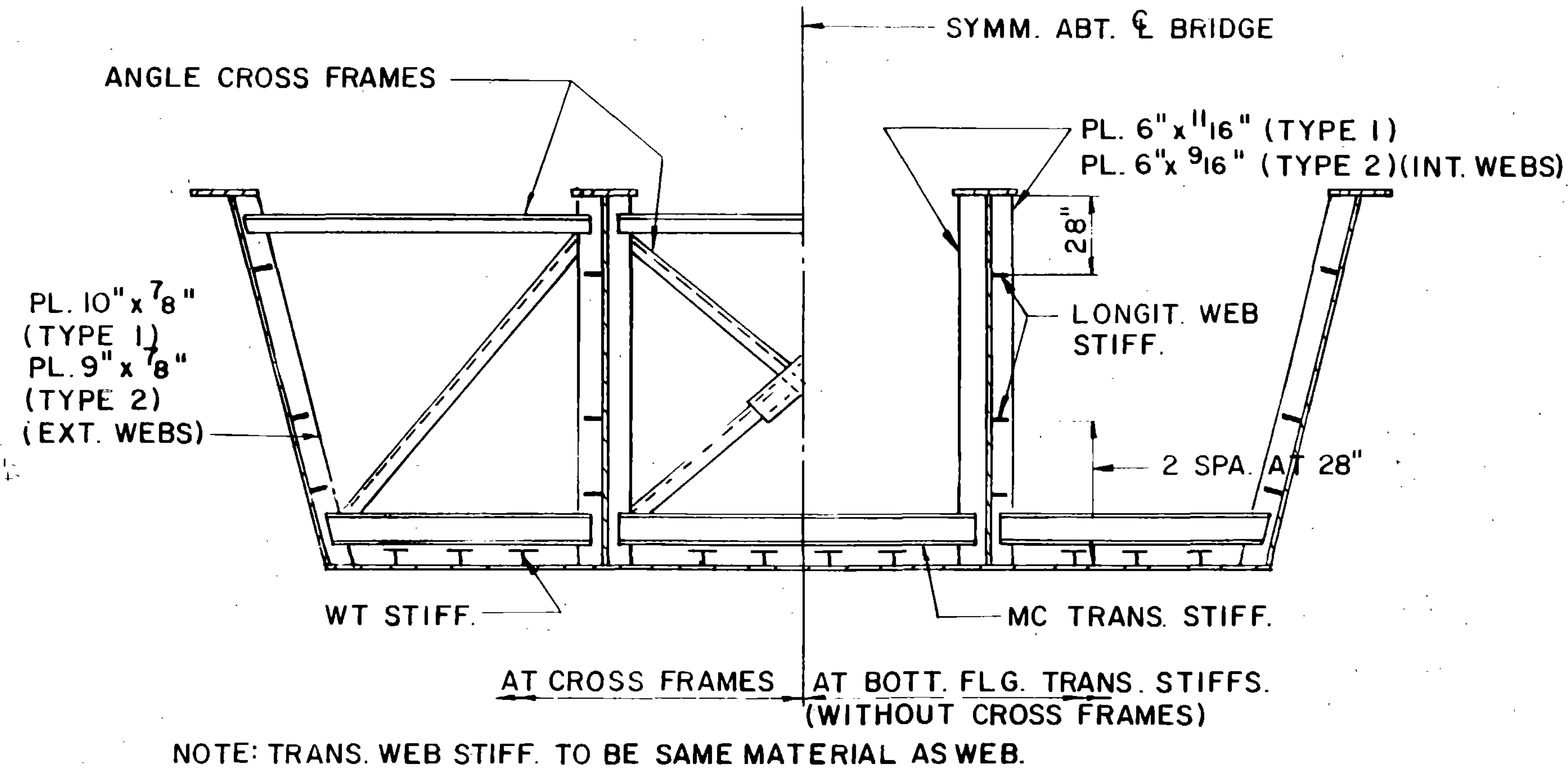


FIGURE 9. TYPICAL CROSS SECTIONS - PROPOSED SPECIFICATION
INTERMEDIATE SPAN

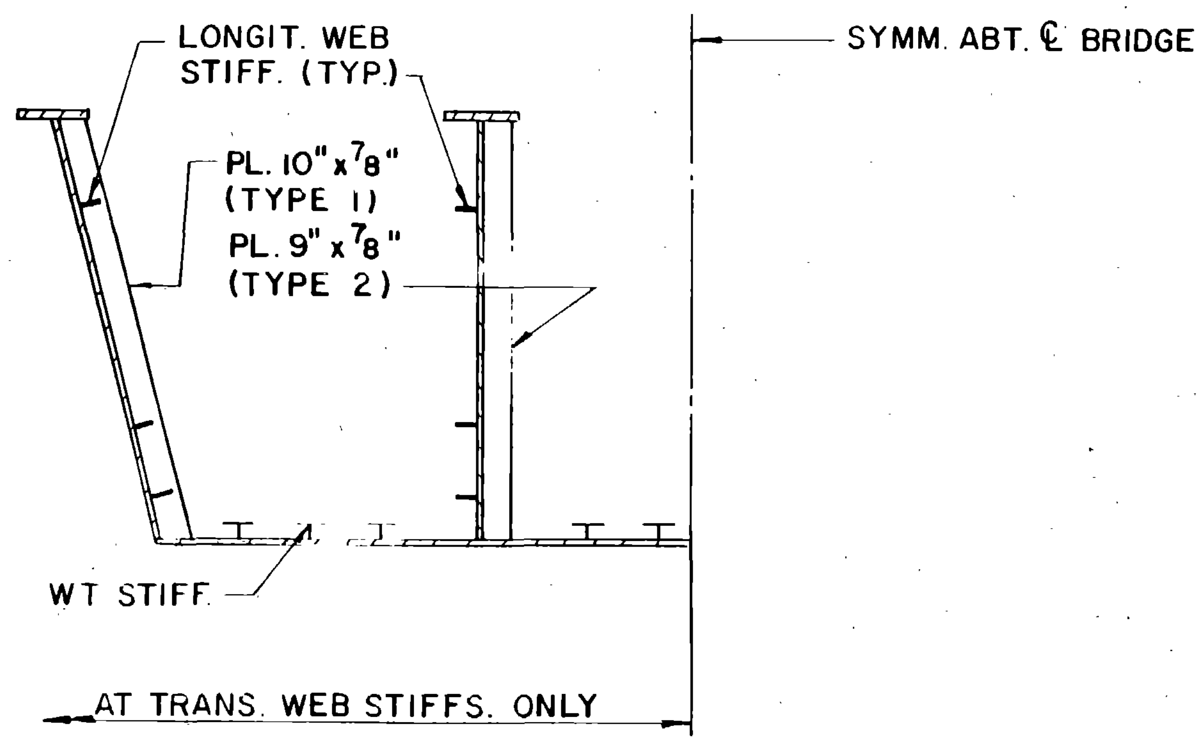


FIGURE 10. TYPICAL CROSS SECTION–PROPOSED SPECIFICATION
INTERMEDIATE SPAN

reason for the difference between 449.2 kips/web for the AASHTO design and 437.0 kips/web for the proposed specification design. The choice of the thinner web was made possible by the use of more than one longitudinal stiffener which is provided for in the proposed specification, in contrast to AASHTO. However, the total web plus stiffener weight increased by 5.9%.

D.2.4. Top Flange Plate

The additional flange forces specified by the new code in Sections 1.7.211 and 1.7.212 result from postbuckling behavior of the webs and consist of two parts: Part 1 is due to the inability of the web to carry bending compression forces after it buckles. Part 2 is the horizontal component of the tension field force due to truss action. In the positive moment area, the above forces are additive to the simple bending forces and a larger area is required. In the negative moment area, the above forces have opposite sign and their net effect is to reduce the flange force. This reduction offsets the increased flange force due to the use of the thinner web in this area. Therefore, a slight reduction in the top flange area over the support was possible by using the proposed specification.

For the top flange as a whole, the increase in material required by the proposed code in the positive moment area was greater than the reduction in the negative moment area. The net effect was an increase of 6.2%.

D.3. LONG SPAN COMPARISON

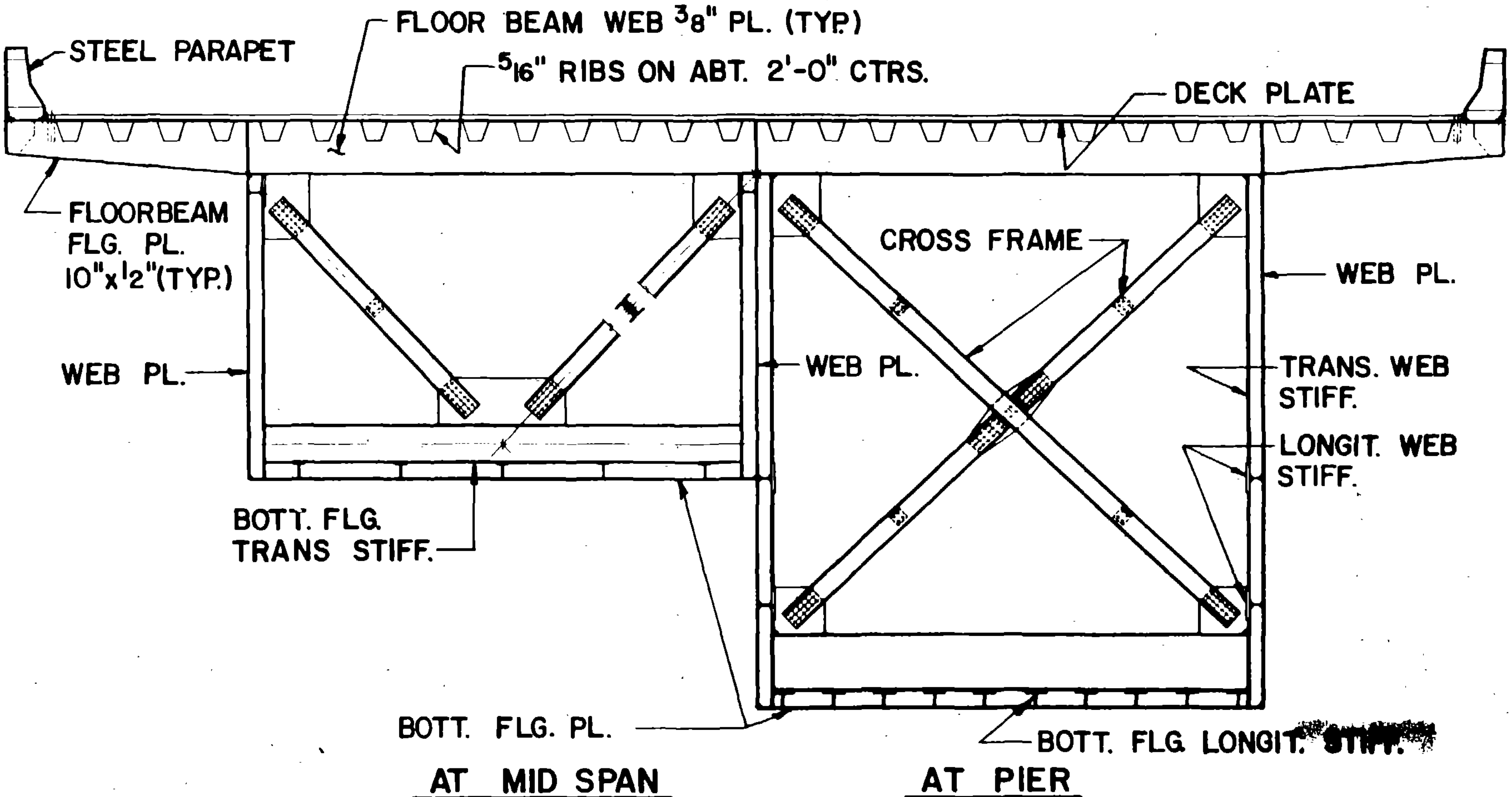
The origin of the differences for the long span structures was essentially the same as for the intermediate span structures discussed above. Although such things as effective flange width, effects of combined stress, etc., became slightly more pronounced, these were not traceable in the final weight computation. For typical cross section for long span structures, see Figure 11. Figures 16 through 19 show typical girder elevations.

D.3.1. Bottom Flange

The main difference was in the part of the bottom flange in compression. A 1.2% increase in weight resulted when using the proposed specification.

D.3.2. Web Plate

The effect of using multiple longitudinal stiffeners showed up as a 5.7% savings in web plate material, up from the 2.7% savings for the intermediate span. Naturally, the increased use of stiffeners would diminish the gain. For web plate plus stiffeners, the weight



NOTE : DESIGN BY PROPOSED SPECIFICATION IS SHOWN. AASHTO DESIGN SIMILAR

FIGURE 11. TYPICAL CROSS SECTION - LONG SPAN

increase diminished from 5.9% for the intermediate span to 4.8% for the long span.

D.3.3. Top Flange Plate

Since the orthotropic deck supplied adequate area for the additional flange force in the positive moment area, the only increase was in the negative moment area. Here, the effect of a thinner web was not offset by the additional flange forces as for the intermediate span. However, the total increase for the total top flange was only 1.0% compared to 6.2% for the intermediate span.

The total effects of all main components was an increase of 1.9% of which 1.1% was caused by web plate including stiffeners and 0.8% by flange plates including stiffeners.

It should be mentioned at this time that all six designs are practical designs where transverse stiffeners in deck, web, and bottom flange are spaced with the same or a multiple of the same spacing. It is easy to optimize one main component, but to have an overall workable solution, some tradeoffs must be made. The designers have tried to make the comparison as fair as possible, and the overall differences have been accounted for by differences in the codes.

E. EVALUATION OF PRACTICALITY AND EASE OF APPLICATION OF THE PROPOSED SPECIFICATION

The basis for evaluating the proposed specification for practicality and ease of application was the alternate designs previously discussed in Sections C and D. To illustrate the differences in application, design examples have been prepared. These are included as Appendix A of this report. Reference will be made to these examples throughout the present discussion. The design examples cover the design of the full cross section for the intermediate span structure at the point of maximum positive and negative moment. These locations were selected for illustration since they are generally the ones of most interest to designers.

Certain areas of the proposed specification that contain apparent discrepancies or require further clarification have been identified during the course of the work covered by this study. Some of these areas have been mentioned in the comparative design examples of Appendix A and additional comments are made below. A complete list and description of all such points is included as Appendix B of this report.

The proposed specification contains comprehensive requirements for box girder bridges. Due to the nature of the structures designed and the limited scope of this study, all requirements of the proposed specification have not been evaluated. The main emphasis has been on the design of stiffened plates, whether flanges or webs.

E.1. EFFECTIVE WIDTH OF FLANGES

Section 1.7.64 of the AASHTO code contains provisions for the effective width of bottom tension flanges. The concrete top flange effective width is determined from Section 1.7.48(C). There are no specific provisions for bottom flanges in compression. Section 1.7.51 contains provisions for orthotropic deck effective widths.

The proposed specification covers effective flange widths in Section 1.7.204. This section applies to both top and bottom flanges in tension or compression. It apparently applies to steel flanges only as Section 1.7.209 refers the designer back to the AASHTO code for composite concrete flanges.

The proposed specification limits the use of effective widths to cases of service load and/or those for which the effects of shear lag in compression flanges are significant. It appears that for most boxes of usual proportions, the steel flanges will be fully effective in the ultimate condition.

The provisions of the proposed specification give the design engineer needed information and present it in a generally suitable form. Figure 1.7.204(d), however, would benefit from an expanded scale and an extension of the curves in their rapidly declining range. As can be seen from Appendix A, Section A.3.1, the application of the proposed specification as regards effective widths is relatively straightforward.

E.2. STIFFENED BOTTOM FLANGE DESIGN

E.2.1. Design Forces

Section 1.7.206(C) of the proposed specification specifies forces for the design of compression flanges and Section 1.7.208(D) specifies forces for tension flanges. Flanges must be designed for axial forces due to longitudinal bending and additional forces, ΔF, when tension field action is utilized in the web design. In addition, shear forces from flexure and torsion have to be considered in the flange design. The forces are applied at the four-tenths point of the flange panel measured from the higher stressed end.

The proposed specification, by specifying forces and location of application, gives the designer some valuable guidance that is lacking in the AASHTO code. Design by the proposed specification will naturally result in some additional design time as compared to strictly following the AASHTO code. However, for large box girders, these additional forces would need to be considered regardless of the design code used.

E.2.2. Compression Flanges

The proposed specification contains provisions for both transversely stiffened and unstiffened flanges. The computation of the flange strength involves only the computation of w/t and L/r ratios for the particular panel under consideration. As can be seen in Appendix A, Section A.5.4.a), the application of the proposed specification is very straightforward. For manual computations, the use of Figure 1.7.206(A) to determine flange strength is quite simple. However, for future computer applications, mathematical formulations will be required.

The design of longitudinal stiffeners of compression flanges is covered in Section 1.7.207. There are two basic requirements; a maximum value for the effective slenderness ratio, C_s, and a maximum value for the width to thickness ratio of stiffener outstanding elements. The parameter C_s is a measure of the stiffener's local torsional buckling stress. A limit on C_s is necessary due to the assumption that the local torsional buckling of the stiffeners does not govern the flange strength. There are two limits on C_s depending on whether or not the maximum stress in the stiffener is greater or less than one-half the yield stress.

The definition of the maximum stress, f_{max}, may require some clarification. It has been assumed that f_{max} includes the effect of ΔF, the additional force from tension field action. This can be inferred from the proposed specification, but is not specifically stated.

It is unfortunate that the limits of C_s, contained in Section 1.7.207(A), are stepped at $0.5\ F_y$. The inclusion of a transition curve would be logical if feasible. Equations are given to determine C_s for particular stiffener types (tees, plates, and angles). If practical, a general mathematical formula that could be used to determine C_s for other shapes (channels, bulb tees, etc.) would be useful.

As can be seen from the design example of Appendix A, the use of the proposed specification to determine flange strength and longitudinal stiffener adequacy is straightforward and is not time consuming.

E.2.3. Tension Flanges

By the proposed specification, the strength of tension flanges is computed as the effective flange area times the yield stress of the material. The AASHTO code defines tension flange strength in a similar fashion. The proposed specification also specifies the use of a reduced equivalent yield stress due to the effect of combined axial and shear stresses.

As shown in Appendix A, Section A.3.2.c), the proposed specification places slenderness limitations on tension flanges. The AASHTO code has no such specific limits. These requirements have a negligible effect on the total area of tension flanges, but a larger percentage of the flange area will consist of stiffeners when design is by the proposed specification. This presents no design difficulty, however.

E.3. STIFFENED WEB DESIGN

E.3.1. Design Forces

Stiffened web design is covered in two sections of the proposed specification; Section 1.7.211 for transversely stiffened webs and Section 1.7.212 for transversely and longitudinally stiffened webs. Section 1.7.211(C) specifies design forces to be used. The effects of shear and direct forces from flexure and other effects are to be considered. These forces are to be applied at the cross section of the panel midway between transverse stiffeners.

The proposed specification also requires that coincident shears and direct stresses be used for web design. While this is not a direct departure from the current AASHTO code, due to the method used to compute the web strength in the proposed specification, some additional design and analysis difficulties arise. Design by the AASHTO code is conventionally based on maximum shear and moment envelopes. As long as the design shear force is less than six-tenths of the web shear capacity, no interaction is considered. For other cases, an interaction equation is used to determine the reduction in moment capacity due to the high shear. Technically, this reduction applies to the coinciding moment. Often, the shear capacity of the web is increased to avoid having to increase the flange material in highly stressed regions.

Since the proposed specification includes the combined effects of shear and direct stresses in the computation of the web capacity, it is now necessary to have coinciding values of shear and moment at all points along the girder. The proposed specification also refers to the "governing" coinciding case. It is not clear whether this refers to the maximum shear and coinciding moment case or some other coinciding case. Some clarification of this point might be in order. It might also be considered, as an alternate, to allow the use of maximum envelope values of shear and moment. The conservatism so introduced is not believed to be significant.

E.3.2. Strength of Webs

Determining web capacity by the proposed specification requires the computation of the shear buckling strength and the tension field strength (if utilized). These computations are illustrated in Section A.3.4 of Appendix A. As can be seen by comparing to the AASHTO computations of Section A.2.5, the amount of work is increased by the use of the proposed specification. The proposed specification does, however, provide a logical and consistent approach to the determination of web strength. Its greater required effort is understandable in light of its greater flexibility in handling stiffener arrangements.

The additional flange force due to tension field action can be computed after each web panel is solved. It is not clear how to compute these forces for composite sections. One method is illustrated in Appendix A. Some clarification of this point would be desirable.

E.3.3. Slenderness Limitations

The slenderness limits of the proposed specification are based on considerations similar to that for the AASHTO requirements. One area needs some additional clarification, however. The definition of D_c, the clear distance between neutral axis and compression flange, should be extended to include composite flanges.

E.4. WEB STIFFENERS

E.4.1. Transverse Stiffeners

By the proposed specification, transverse intermediate stiffeners are checked for strength, rigidity, and torsional buckling stability. The AASHTO code requires similar checks. Due to the way in which the rigidity and strength requirements are handled in the proposed specification, some additional design time will be required when compared to AASHTO.

The proposed specification requires the moment of inertia of the stiffener to be computed including an effective width of web. The AASHTO code assumes the neutral axis of the stiffener is at the centerline of the web. This AASHTO assumption somewhat simplifies the computations.

The AASHTO code covers the strength requirement by stipulating a necessary stiffener area. The proposed specification requires that the stiffener be designed as a column to support the tension field vertical force. Therefore, design by the proposed specification requires additional design effort.

The check for local torsional buckling stability of the stiffener is treated in the proposed specification in a manner similar to that discussed above for longitudinal flange stiffeners. For web stiffeners, f_{max} is defined as the maximum calculated factored compression stress in the stiffener outstand. The location where f_{max} is to be computed needs to be clarified. For one-sided stiffeners, the maximum compression in the stiffener will be at its intersection with the web. The free edge of the stiffener will normally be in tension. It seems somewhat conservative to require that the effective slenderness coefficient of this stiffener be limited to the same value as stiffeners where the maximum compressive stress occurs at the free edge.

E.4.2. Longitudinal Stiffeners

Longitudinal stiffeners are treated similar to transverse stiffeners in the proposed specification. Therefore, the discussion above is applicable in general terms.

E.5. SUMMARY

The proposed specification has been found to be a comprehensive specification for box girder bridges. Generally, it is no more difficult to apply than the AASHTO code. The total design effort is greater, however, with the proposed specification since it is more flexible and refined in many areas as compared to AASHTO. Some areas would benefit from additional explanations and clarifications as discussed above.

F. PARAMETRIC STUDIES

This section summarizes the results of parametric studies used to investigate the application of the proposed specification to the principal elements of steel box girders. The proposed specification emphasizes two main elements; the webs, and the bottom compression flanges. Therefore, the parametric studies were confined to these elements.

The main focus of these studies has been to determine the effect of various parameters on element weight. A total economic analysis was not considered as part of this study.

F.1. WEBS AND WEB STIFFENERS

Webs from each of the structures designed were studied. Table 3 summarizes the web panels which were investigated. Cases SNW and MNW were located at the interior supports of the respective structures. Cases MPW were located at the 0.4 point of Span 1 and cases LPW were located at the 0.25 point of Span 1.

The effects of three basic parameters were investigated. These parameters were: the transverse stiffener spacing, the web thickness, and the number of longitudinal stiffeners. The effect of any given parameter was determined by varying that parameter while holding the others constant.

F.1.1. Effect of Transverse Stiffener Spacing

This parameter was evaluated by considering the following groups of cases:

1) SNW - BASE, 1, 2
2) MPW - BASE, 1, 2, 3
3) MNW - 1, 2, 3
4) LPW - BASE, 1, 2
5) LPW - 3, 4
6) LPW - 5, 6
7) LPW - 7, 8, 9

See Table 3 for explanation of symbols and results.

It can be seen that, in general, the weight of web plus stiffeners decreases slightly as the web panel becomes longer, reaches a minimum, and then increases for still longer panels. Therefore, the longest web panel will not, in general, be the least-weight solution.

TABLE 3. SUMMARY OF PARAMETRIC WEB STUDIES - PROPOSED CODE

CASE	n	F_y (ksi.)	d_o (in.)	D (in.)	t_w (in.)	TRANS. STIFF. (in. x in.)	LONG. STIFF. (in. x in.)	WEB WEIGHT (lb./ft.)	STIFF. WEIGHT (lb./ft.)	V (kips)	V_B (kips)	$\frac{V-V_B}{V_T}$
SNW-BASE	0	36	30	48	.375	4 x .375	-	61.2	8.2	470	363	.88
SNW-1	0	36	20	48	.375	5 x .375	-	61.2	15.3	470	400	.53
SNW-2	0	36	35	48	.375	4 x .375	-	61.2	7.0	470	355	1.0
MPW-BASE	1	36	300	148.5	.4375	6 x .4375	12 x .625	220.9	29.9	129	52	.45
MPW-1	1	36	432	148.5	.4375	6 x .50	15 x .6875	220.9	38.6	129	51	.63
MPW-2	1	36	144	148.5	.4375	5 x .375	9 x .4375	220.9	20.0	129	53	.24
MPW-3	1	36	72	148.5	.4375	5 x .375	7 x .375	220.9	21.1	129	53	.15
MNW-BASE	2	50	60	148.5	.4375	10 x .875	6 x .50	220.9	90.4	1029	617	.87
MNW-1	2	50	60	148.5	.50	10 x .5625	6 x .50	252.5	64.0	1029	765	.51
MNW-2	2	50	78	148.5	.50	10 x .875	6 x .50	252.5	73.4	1029	695	.69
MNW-3	2	50	90	148.5	.50	10 x 1.125	7 x .5625	252.5	83.0	1029	609	.88
MNW-4	1	50	60	148.5	.50	10 x 1.125	6 x .50	252.5	105.1	1029	453	.94
LPW-BASE	1	36	90	168	.50	7 x .50	7 x .625	285.6	37.1	362	168	.42
LPW-1	1	36	120	168	.50	8 x .625	7 x .625	285.6	38.7	362	158	.51
LPW-2	1	36	168	168	.50	11 x .875	9 x .625	285.6	51.8	362	145	.68
LPW-3	1	36	90	168	.5625	8 x .4375	7 x .5625	321.3	35.6	362	215	.29
LPW-4	1	36	120	168	.5625	7 x .625	8 x .625	321.3	37.8	362	205	.35
LPW-5	1	36	90	168	.4375	9 x .75	6 x .50	250.0	53.0	362	126	.58
LPW-6	1	36	72	168	.4375	8 x .625	5 x .4375	250.0	47.0	362	138	.50
LPW-7	2	36	90	168	.50	6.5 x .375	6.5 x .50	285.6	32.9	362	313	.11
LPW-8	2	36	120	168	.50	5 x .375	7 x .625	285.6	30.2	362	311	.13
LPW-9	2	36	168	168	.50	5 x .375	9 x .625	285.6	33.7	362	311	.17
LPW-10	1	36	168	168	.50	5.5 x .4375	7 x .625	285.6	23.1	362	-	-

NOTES:
1. SNW refers to the short span negative moment area.
2. MPN refers to the intermediate span positive moment area.
3. MNW refers to the intermediate span negative moment area.
4. LPW refers to the long span positive moment area.
5. BASE is the resulting panel from the design examples.
6. n = the number of longitudinal stiffeners.
7. When two longitudinal stiffeners are used only the one closest to the compression flange is indicated.
8. V = factored applied shear, per 1 box for SNW cases, per web for others.
9. V_B = buckling shear capacity per box or web.
10. V_T = tension field shear capacity per box or web.

This conclusion can be explained by considering the effect that the panel length has on the transverse and longitudinal stiffener sizes. For very short panels the rigidity requirement for the transverse stiffeners is relatively large and tends to govern the stiffener design. The column force, P_{VT}, from the tension field action, is small and does not control. As the web panel increases in length the rigidity requirement decreases but P_{VT} increases due to the greater utilization of the tension field strength. Thus, the transverse stiffener size decreases as the rigidity requirement diminishes but ultimately becomes larger due to the increased strength requirement.

The longitudinal stiffeners increase in size as the panel length becomes longer. The rigidity requirement increases for larger panel lengths, and it is this factor that dominates the longitudinal stiffener design. For the cases studied the strength requirement did not control.

The general trends discussed above applied to all groups of cases except Group 1). In Group 1), it was observed that the stiffener weight continued to decrease with increasing panel length. These cases did not use a longitudinal stiffener, therefore, the increasing weight of this element was not present. In addition, the web plate used was relatively stocky and thus the P_{VT} force at the maximum panel length was small and did not govern the transverse stiffener design. Therefore, it can be concluded that for webs in which a large proportion of the total shear force is resisted by V_B (buckling action), longer relative panel lengths result in least weight. This is particularly true for webs with no longitudinal stiffeners.

F.1.2. Effect of Web Thickness

This parameter was evaluated by considering the following groups of cases:

1) MNW - BASE, 1
2) LPW - BASE, 3, 5
3) LPW - 1, 4

From the limited cases studied, it is seen that for all groups the case with the thinner web results in the least weight. Holding the panel length constant and reducing the web thickness reduces the buckling capacity of the panel and forces more of the shear load to be carried by tension field action with resulting higher P_{VT} forces. For the cases considered, the increase in transverse stiffener material was more than offset by the reduction in web material.

F.1.3. Effect of Number of Longitudinal Stiffeners

This parameter was evaluated by considering the following groups of cases:

1) MNW - 1, 4
2) LPW - BASE, 7
3) LPW - 1, 8
4) LPW - 2, 9

It was observed that increasing the number of longitudinal stiffeners resulted in a reduction in total stiffener material. The additional stiffener raises the buckling capacity of the web and reduces the column force in the transverse stiffener. There is also a reduction in the required stiffness of the transverse stiffener when the number of longitudinal stiffeners is increased.

F.2. COMPRESSION FLANGES AND STIFFENERS

Table 4 is a summary of the flange panels studied. The flange from each of the structures designed was studied. The location of study was taken at the interior support for all structures.

The investigation was done by holding the w/t ratio constant, varying the longitudinal stiffener rigidity and hence r, and using Figure 1.7.206(A) of the proposed specification to solve for L, the distance between transverse stiffeners. Then either w or t was incremented to get a new w/t ratio and the process was repeated. Basically, three different longitudinal stiffeners were used, a WT7, WT9, and WT12. The stiffener rigidities approximately doubled for each increase in stiffener size. The basic parameters investigated were, therefore, the w/t ratio and the stiffener rigidity.

F.2.1. Effect of w/t

For a given number of longitudinal stiffeners, the effect of the w/t ratio is investigated by varying t; for example, see cases SF-2, 3, 4 as compared to SF-13, 14, 15. From this type of analysis, it was observed that total flange weight decreased with increased w/t. The increased transverse stiffening required with the thinner plate was offset by the reduction in plate area. Of course, there is a practical limit to the degree to which w/t can be increased. In comparing case MF-2 to MF-5, it is seen that the case with the larger w/t has the greater weight. This is so because the longitudinal stiffener is not very rigid, requiring a very close transverse stiffener spacing to develop the required strength. When a larger longitudinal stiffener is used (cases MF-3 and MF-6) the higher w/t ratio does indeed result in the least weight.

TABLE 4. SUMMARY OF PARAMETRIC COMPRESSION FLANGE STUDIES - PROPOSED CODE

CASE	w (in.)	t (in.)	F_y (ksi.)	λ_{PL}	LONG. STIFF.	r (in.)	L (in.)	λ_{COL}	F_u/F_y	b (in.)	TRANS. STIFF.	FLG. WT. (lb./ft.)
SF-2	24	.875	36	.51	WT9 x 23	3.36	131	.43	.90	96	MC7 x 17.6	379.5
SF-3	24	.875	36	.51	WT7 x 19	2.60	77	.33	.94	96	MC7 x 17.6	377.7
SF-4	24	.875	36	.51	WT12 x 27.5	4.39	216	.53	.87	96	MC7 x 17.6	387.8
SF-5	32	1.000	36	.59	WT7 x 19	2.28	61	.30	.90	96	MC8 x 20	410.0
SF-6	32	1.000	36	.59	WT9 x 23	2.97	103	.39	.88	96	MC7 x 17.6	402.0
SF-7	31	1.000	36	.59	WT12 x 27.5	3.74	167	.50	.85	96	MC7 x 17.6	405.1
SF-8	32	1.125	36	.53	WT7 x 19	2.20	129	.66	.81	96	MC7 x 17.6	433.5
SF-9	32	1.125	36	.53	WT9 x 23	2.87	179	.70	.79	96	MC7 x 17.6	437.9
SF-9A	32	1.125	36	.53	WT12 x 27.5	3.79	250	.74	.77	96	MC7 x 17.6	444.0
SF-11	32	.9375	36	.63	WT9 x 23	3.03	41	.15	.92	96	MC10 x 21.9	416.5
SF-12	32	.9375	36	.63	WT12 x 34	4.40	157	.40	.86	96	MC7 x 17.6	398.0
SF-13	24	.9375	36	.47	WT7 x 19	2.55	109	.48	.89	96	MC7 x 17.6	391.0
SF-14	24	.9375	36	.47	WT9 x 23	3.30	168	.57	.85	96	MC7 x 17.6	398.0
SF-15	24	.9375	36	.47	WT12 x 27.5	4.33	244	.63	.82	96	MC7 x 17.6	408.0
SF-16	48	1.25	36	.71	WT7 x 19	1.81	82	.55	.78	96	MC7 x 19.1	466.0
SF-17	48	1.25	36	.71	WT9 x 23	2.38	128	.60	.77	96	MC7 x 17.6	461.0
SF-18	48	1.25	36	.71	WT12 x 34	3.58	204	.64	.75	96	MC7 x 17.6	467.0
SF-19	48	1.125	36	.79	WT7 x 19	1.88	18	.11	.86	96	MC12 x 30.9	566.0
SF-20	48	1.125	36	.79	WT9 x 23	2.46	33	.15	.85	96	MC10 x 21.9	468.0
SF-21	48	1.125	36	.79	WT12 x 34	3.70	89	.27	.82	96	MC7 x 17.6	435.0
MF-2	28.8	1.25	50	.50	WT7 x 19	2.22	102	.61	.83	144	MC18 x 42.7	748.0
MF-3	28.8	1.25	50	.50	WT9 x 23	2.90	138	.63	.82	144	C15 x 33.9	739.0
MF-4	28.8	1.25	50	.50	WT12 x 34	4.26	235	.73	.77	144	MC12 x 30.9	767.0
MF-5	28.8	1.125	50	.56	WT7 x 19	2.29	49	.28	.92	144	MC18 x 58	801.0
MF-6	28.8	1.125	50	.56	WT9 x 23	2.98	86	.38	.89	144	MC18 x 42.7	714.0
MF-7	28.8	1.125	50	.56	WT12 x 34	4.37	198	.60	.84	144	MC12 x 30.9	709.0
MF-8	28.8	1.375	50	.46	WT7 x 19	2.16	119	.73	.77	144	C15 x 33.9	790.0
MF-9	28.8	1.375	50	.46	WT9 x 23	2.82	162	.76	.75	144	MC13 x 31.8	793.0
MF-10	24	1.125	50	.47	WT7 x 19	2.44	89	.48	.89	144	MC18 x 42.7	715.0
MF-11	24	1.125	50	.47	WT9 x 23	3.16	132	.55	.86	144	C15 x 33.9	703.0
MF-12	24	1.125	50	.47	WT12 x 34	4.58	236	.68	.80	144	C12 x 30.9	740.0

TABLE 4. SUMMARY OF PARAMETRIC COMPRESSION FLANGE STUDIES - PROPOSED CODE (continued)

CASE	w (in.)	t (in.)	F_y (ksi.)	λ_{PL}	LONG. STIFF.	r (in.)	L (in.)	λ_{COL}	F_u/F_y	b (in.)	TRANS. STIFF.	FLG. WT. (lb./ft.)
MF-13	24	1.000	50	.52	WT7 x 19	2.51	38	.20	.98	144	2-MC18 x 42.7	909.0
MF-14	24	1.000	50	.52	WT9 x 23	3.25	69	.28	.95	144	MC18 x 42.7	694.0
MF-15	24	1.000	50	.52	WT12 x 34	4.69	185	.52	.87	144	MC12 x 30.9	684.0
LF-2	24	1.125	50	.47	WT7 x 19	2.44	168	.91	.67	240	W24 x 55	1168.0
LF-3	24	1.125	50	.47	WT9 x 23	3.17	225	.94	.65	240	W21 x 50	1178.0
LF-4	24	1.000	50	.52	WT7 x 19	2.51	148	.78	.74	240	W24 x 55	1076.0
LF-5	24	1.000	50	.52	WT9 x 23	3.25	202	.82	.72	240	W21 x 50	1082.0
LF-6	24	.875	50	.60	WT7 x 19	2.60	112	.57	.83	240	W24 x 68	1030.0
LF-7	24	.875	50	.60	WT9 x 23	3.36	173	.68	.80	240	W24 x 55	997.0
LF-8	24	.875	50	.60	WT12 x 34	4.80	298	.82	.72	240	W21 x 44	1055.0

NOTES:
1. The bottom flange at the interior support was the location of study for all structures.
2. SF cases are for the short span structure.
3. MF cases are for the intermediate span structure.
4. LF cases are for the long span structure.
5. FLG. WT. shown includes flange plate plus all stiffeners.

Therefore, it can be concluded that to efficiently utilize increased w/t ratios, the rigidity of the longitudinal stiffener will ultimately need to be increased.

By using a different number of longitudinal stiffeners w is changed and the effect of varying both w and t is investigated. For a given number of stiffeners, as discussed above, there is a practical maximum for w/t. It is observed from the studies done that this practical maximum decreases in magnitude for increasing values of w.

F.2.2. Effect of Longitudinal Stiffener Rigidity

The longitudinal stiffener rigidity was varied to investigate the tradeoff between longitudinal and transverse stiffener weight. Obviously, L and r move in the same direction. Larger transverse stiffener spacings require larger longitudinal stiffeners.

Generally, for any given w/t ratio, the total weight of flange stiffeners did not vary significantly for the different longitudinal stiffeners used. The only cases in which this was not observed were those where the w/t ratio was near its maximum. As discussed above, in these cases, the longitudinal stiffener rigidity was such that very small transverse stiffener spacings were required.

G. CONCLUSIONS AND RECOMMENDATIONS

The proposed design rules for steel box girder bridges were meant to be practical and easy to apply. At the same time, they should reflect the current state-of-the-art and be based on ultimate load principles to accommodate integration with AASHTO load factor design. Furthermore, their use should result in a safe, economical steel superstructure.

It is the general conclusion derived from this comparative study that the above set of goals are fully met and that the proposed specification truly lives up to the latest state-of-the-art. The proposed specification permits the designer flexibility in planning stiffener arrangements in order to fully optimize the structure.

The differences arising from the inclusion of geometric imperfections and residual stresses in the actual girder weight are minor for the long span box girder. The load factors in AASHTO were initially derived for shorter span bridges and since at present the recommendation for truss load factors calls for a factor of 1.5, a reduction of the load factors does not seem justified for long span structures.

It is recommended that the strength of stiffened and unstiffened compression flanges be given in equations to facilitate computer based design.

The tension field strength has been reduced to the "true Basler" solution which reflects the latest thinking in that field and particularly because the flanges of box girders are less rigid than those of plate girders. Furthermore, the flange stability may be endangered with a fully developed tension field. This reduction in strength seems justified, specifically for long span bridges. However, further testing aimed at determining the ultimate strength of box girder webs is needed.

The proposed specification allows the tension field action, V_T (1.7.211(B)) to be disregarded if no advantage is derived from its application. This mainly would occur on shorter spans where the D/t ratio is less and no longitudinal stiffener is provided. However, for these structures the AASHTO code presents a far easier, faster and more economic design.

Short span bridges, as they are built today according to AASHTO and tradition, have a higher built-in safety factor due to lateral distribution factors and the common practice of using moment and shear envelopes rather than coinciding forces.

The requirement in the new code of using coinciding forces puts a large multiplier on design work since wherever the influence of several forces exists the multiple combinations of flexure, torsion, shear and axial loads make the determination of the governing loading case complex.

It is suggested that the proposed specification be modified to allow the use of the worst combination from a set of loads, not necessarily the envelope values. This could greatly ease design efforts without much overdesign.

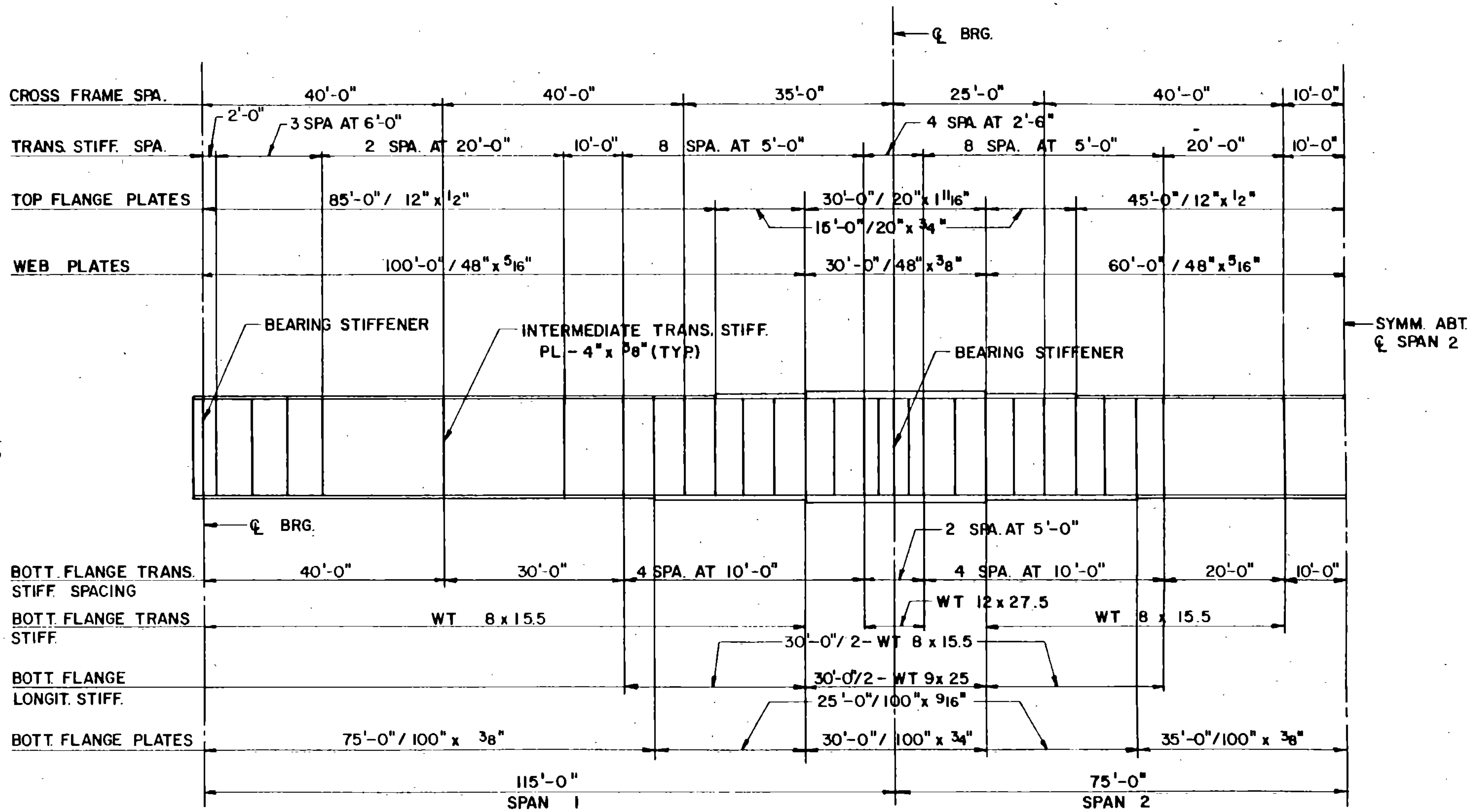
It seems unduly restrictive in the proposed specification to require an analysis which includes the effects of torsional distortions. In some well braced box shapes these effects are of minor impact and may be evaluated and superimposed later.

When comparing the codes, in general, the proposed specification represents the latest state-of-the-art. Its level of sophistication is higher. It, therefore, allows the designer more freedom (stiffener arrangement, etc.). Its requirements for exactness are greater (coinciding forces). Therefore, it is understandable that the effort on the part of the designer both in understanding and using the proposed specification is more demanding.

For long span structures these demands are acceptable and justifiable and the proposed specification fills a long needed gap.

For short span bridges the existing AASHTO specifications have proven that they fully serve their purpose in giving easy, economical and safe design results.

The difficult question remains, however, whether to use the AASHTO code or the proposed specification for the design of any specific structure. The answer should not be entirely a matter of economy, but should involve an assessment of the structural adequacy of the finished bridge. At what point is it beneficial to use multiple longitudinal web stiffeners? When is the safety of the structure being infringed upon by using the Basler solution for web design? At what span length does the AASHTO compression flange design become inappropriate due to the absence of transverse stiffeners? These questions are hard to answer and are dependent on many parameters such as the girder cross section, plate thicknesses, stiffener arrangements, etc. Many of these parameters are related to the span length. Without any knowledge of the specific structure deciding on the span length to change from the AASHTO code to the proposed specification becomes very difficult and subjective. However, it appears that this transition from AASHTO to the proposed specification should occur between span lengths of 250 and 300 ft.



NOTE: ALL MATERIAL SHALL BE A36.

FIGURE 12. INTERIOR ELEVATION OF TYPICAL GIRDER WEB - AASHTO DESIGN - SHORT SPAN

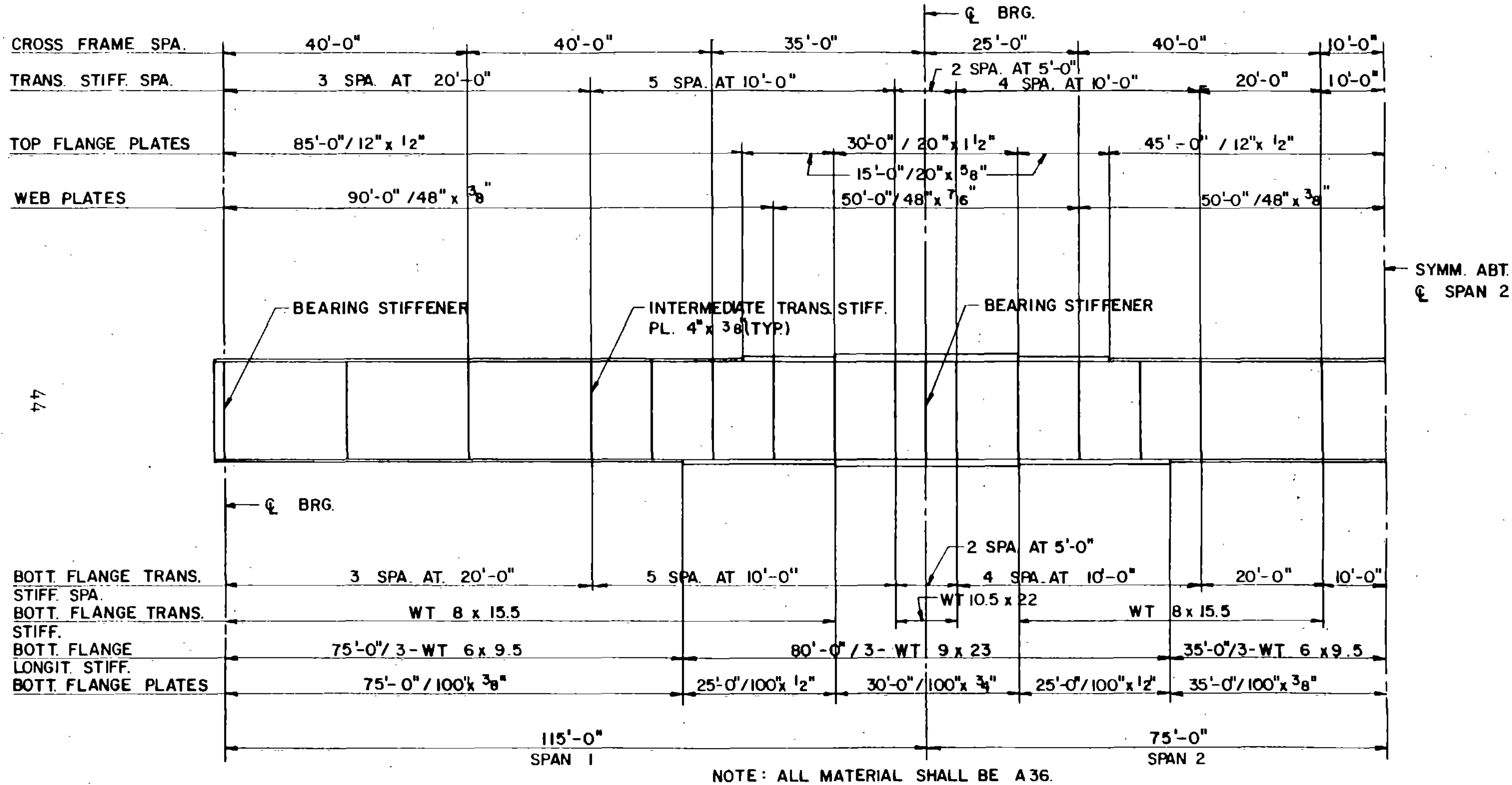
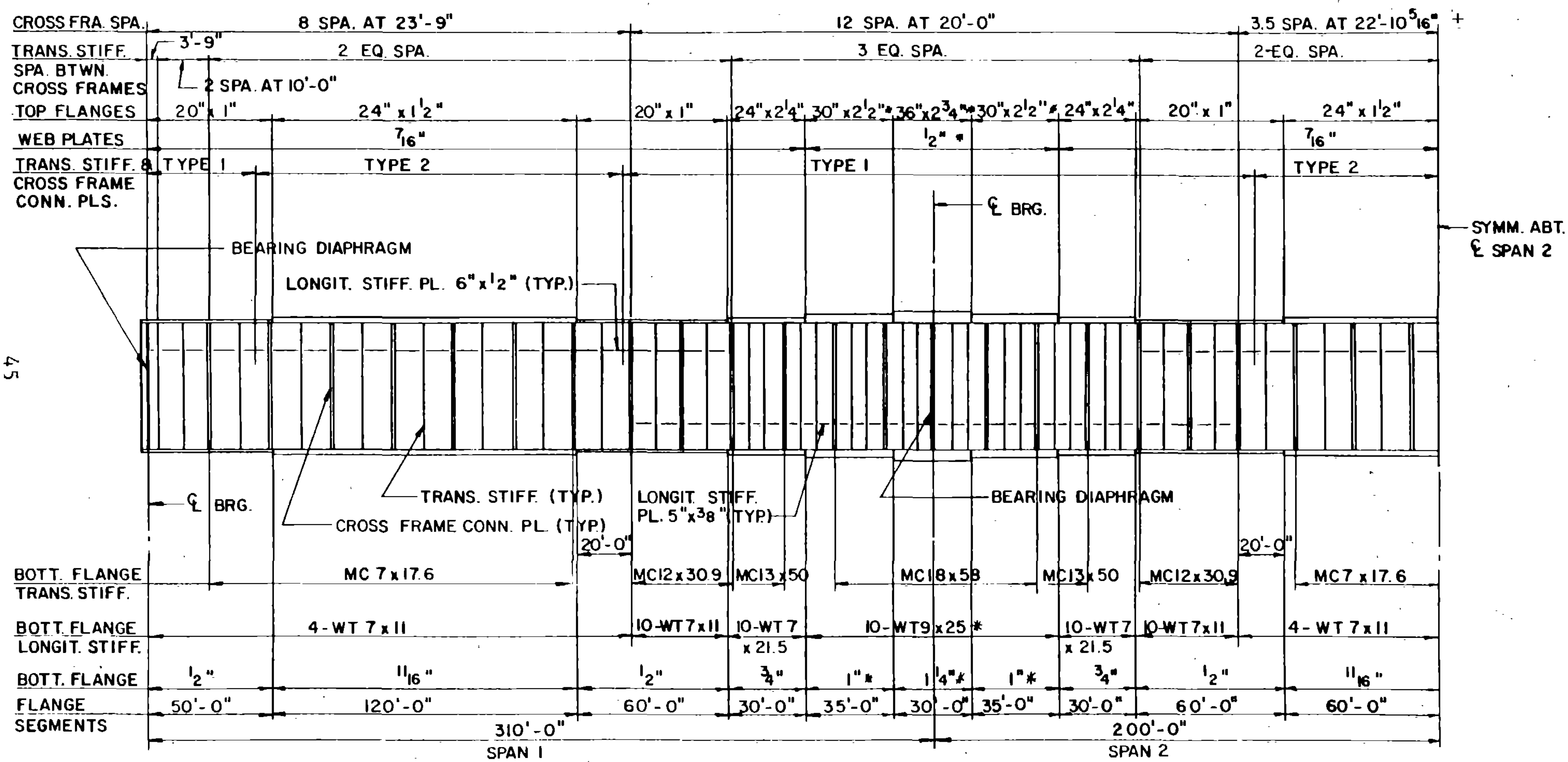
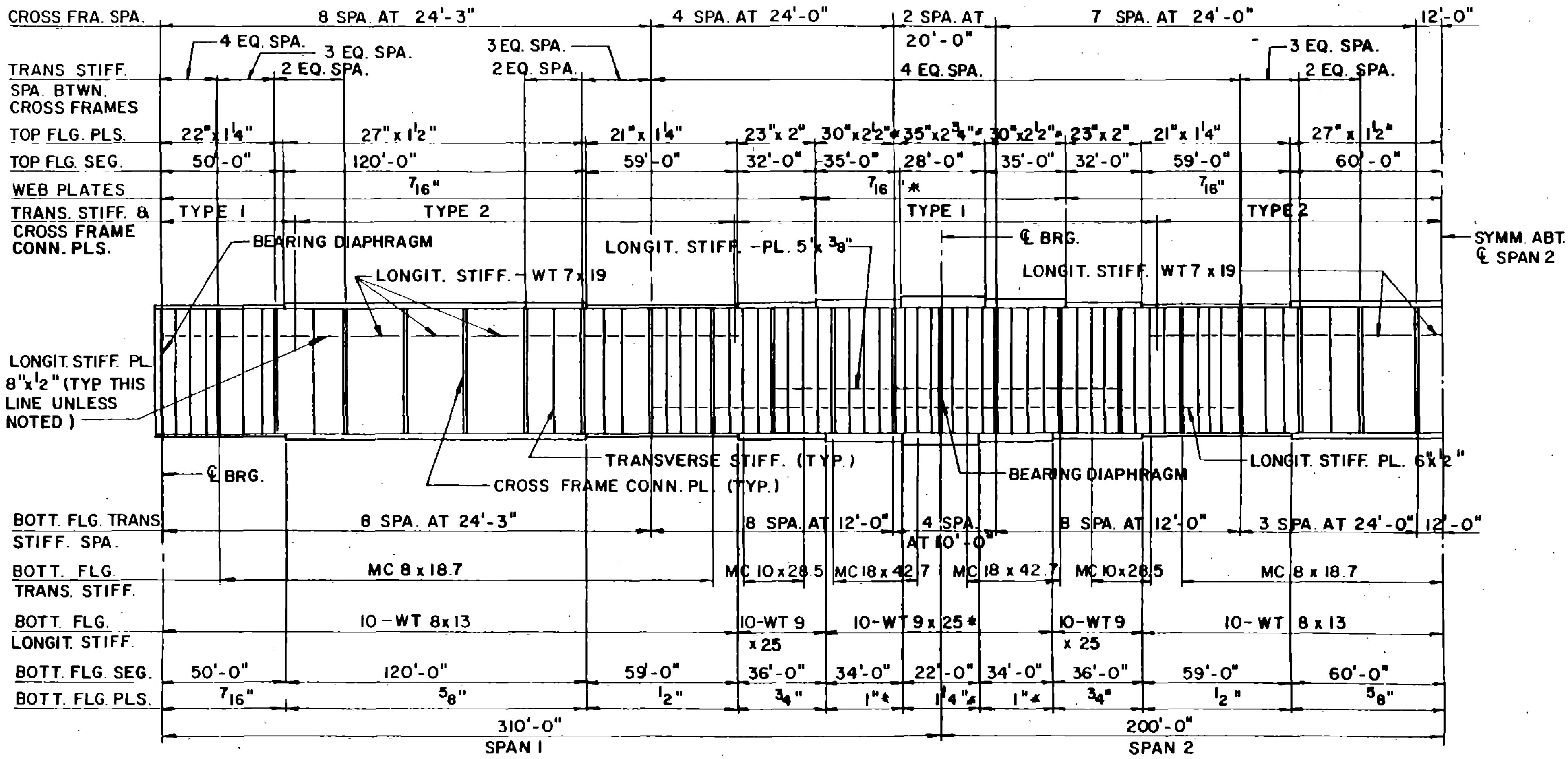


FIGURE 13. INTERIOR ELEVATION OF TYPICAL GIRDER WEB - PROPOSED SPECIFICATION SHORT SPAN



NOTE: ALL MATERIAL TO BE A36 UNLESS OTHERWISE NOTED.
MATERIAL NOTED BY AN * SHALL HAVE A YIELD POINT OF 50 ksi.
INTERIOR WEBS ARE SAME AS EXTERIOR WEB EXCEPT AS NOTED.

FIGURE 14. INTERIOR ELEVATION OF EXTERIOR WEB-AASHTO DESIGN-INTERMEDIATE SPAN



NOTE: ALL MATERIAL TO BE A36 UNLESS OTHERWISE NOTED.
MATERIAL NOTED BY AN * SHALL HAVE A YIELD POINT OF 50^{ksi}.
INTERIOR WEBS ARE SAME AS EXTERIOR WEBS EXCEPT AS NOTED.

FIGURE 15. INTERIOR ELEVATION OF EXTERIOR WEB-PROPOSED SPECIFICATION — INTERMEDIATE SPAN

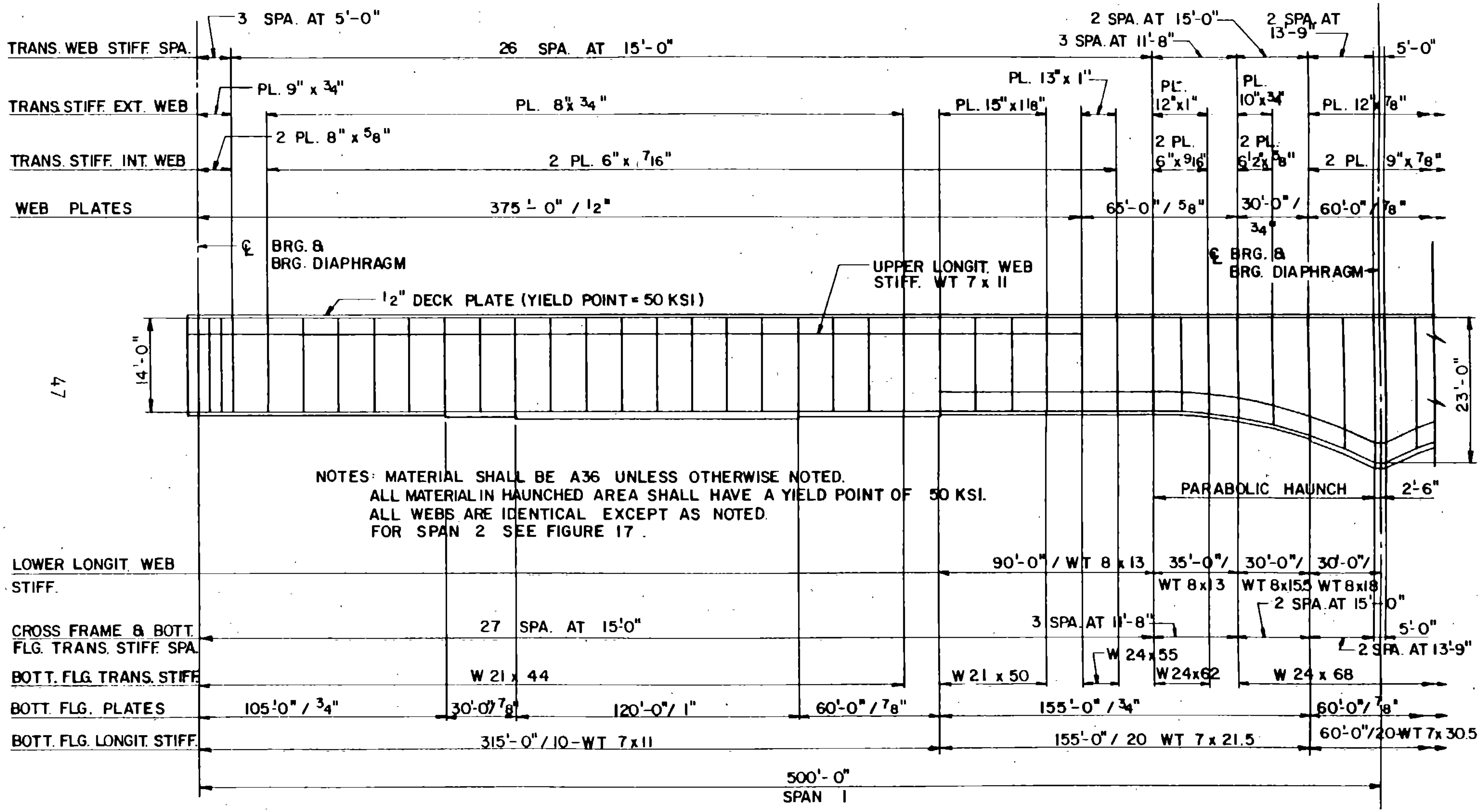


FIGURE 16. INTERIOR ELEVATION OF GIRDER WEB — AASHTO DESIGN — LONG SPAN

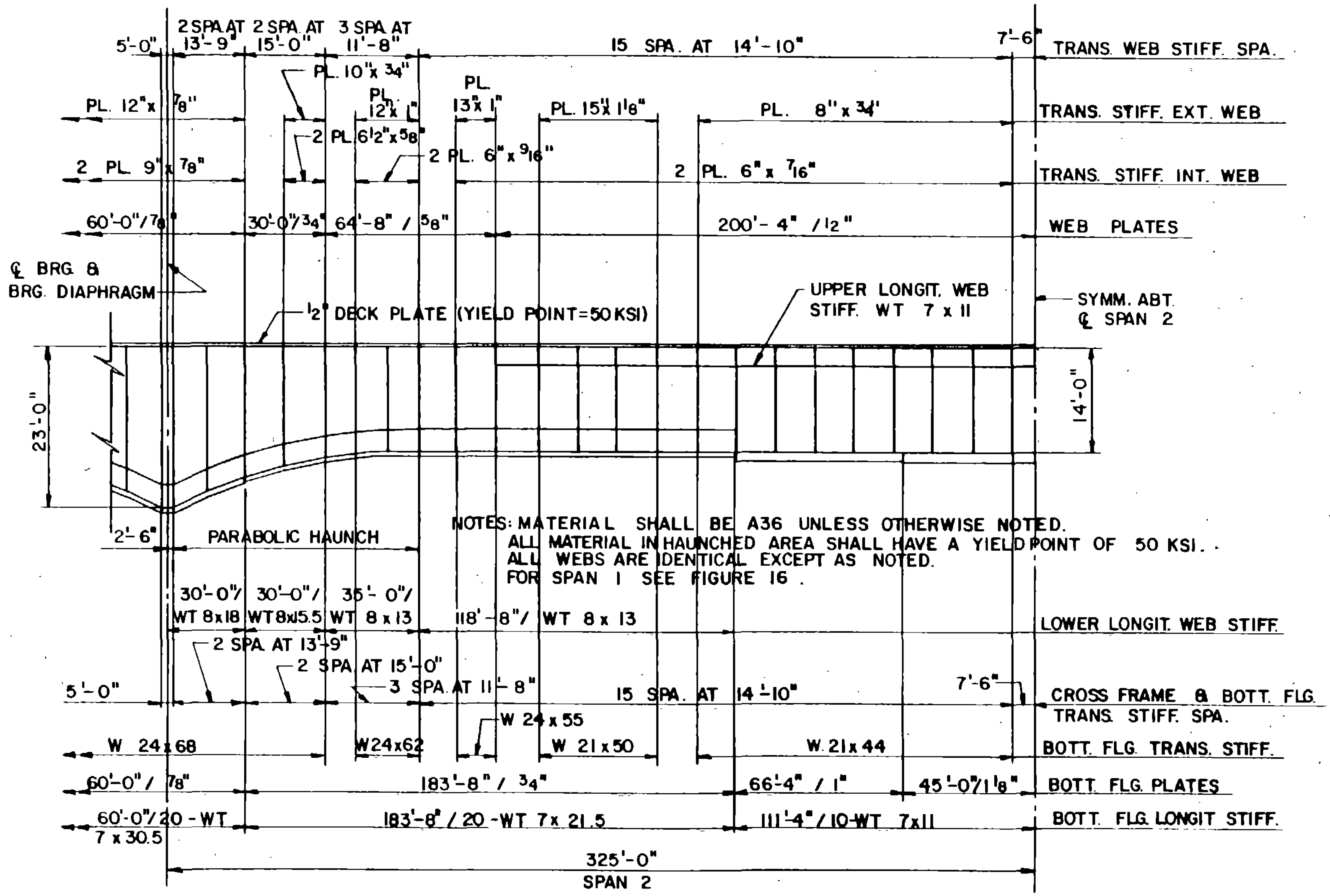


FIGURE 17. INTERIOR ELEVATION OF GIRDER WEB — AASHTO DESIGN — LONG SPAN

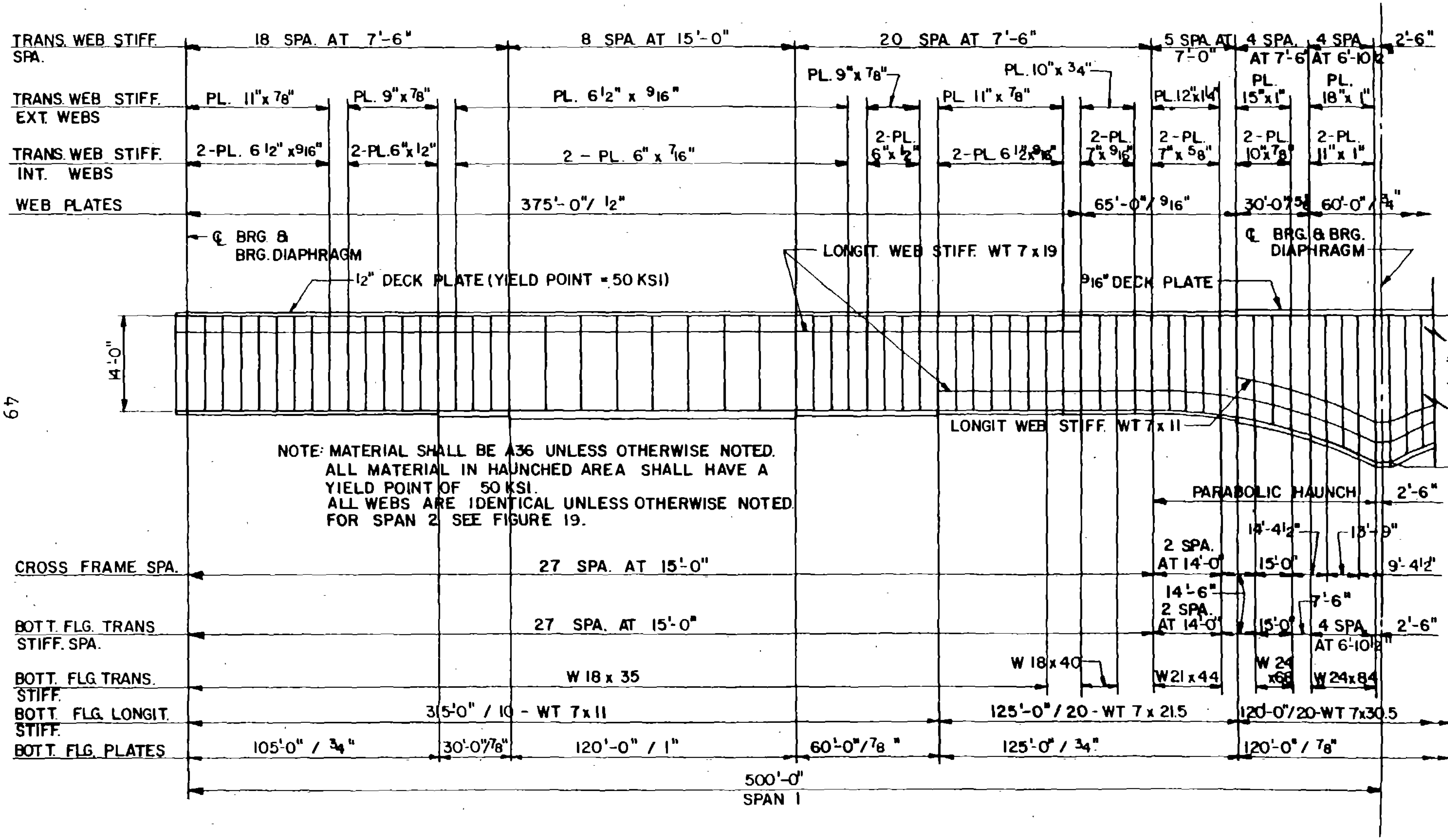


FIGURE 18. INTERIOR ELEVATION OF GIRDER WEB - PROPOSED SPECIFICATION - LONG SPAN

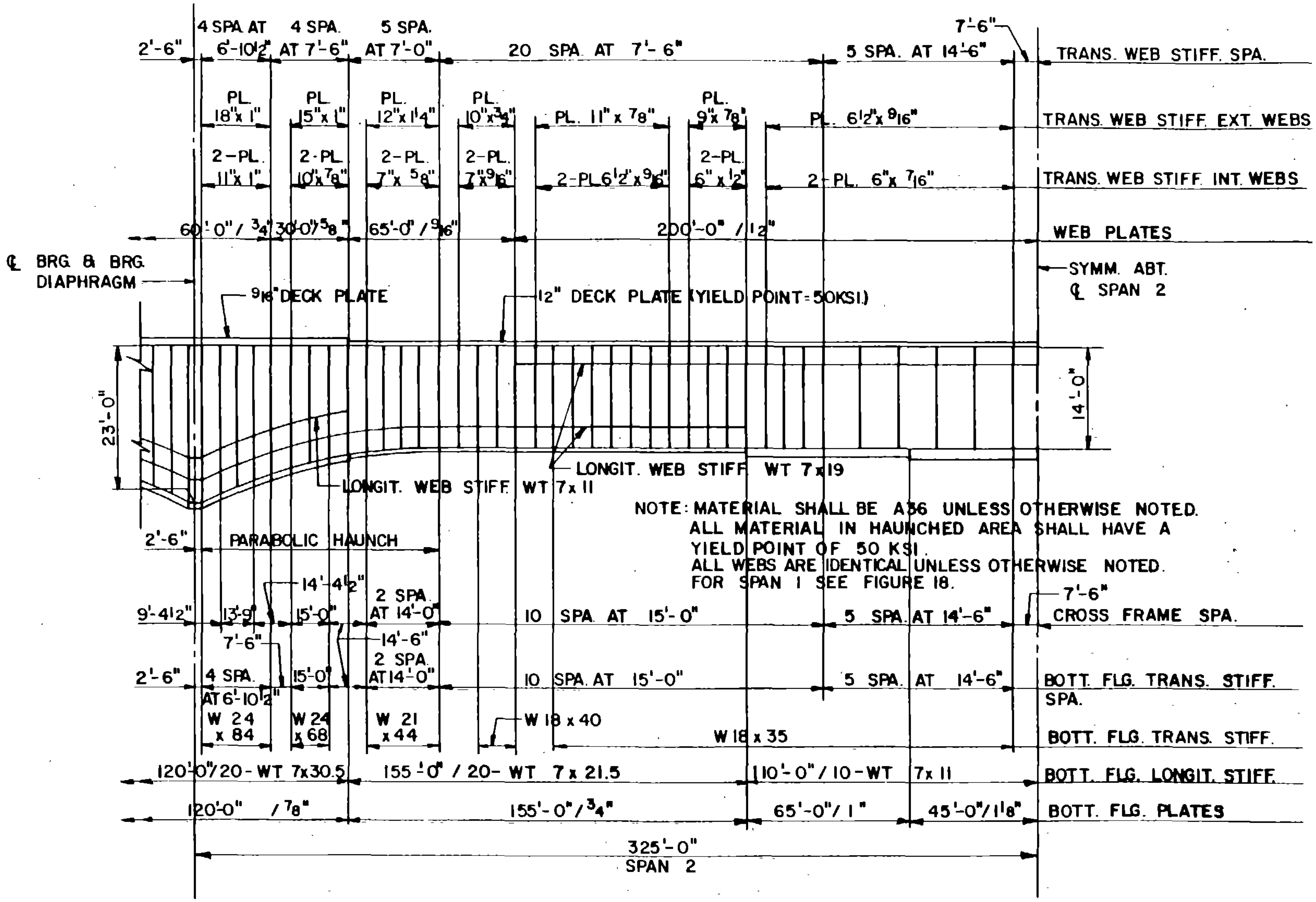


FIGURE 19. INTERIOR ELEVATION OF GIRDER WEB — PROPOSED SPECIFICATION — LONG SPAN

APPENDIX A
COMPARATIVE DESIGN EXAMPLES

CONTENTS

CONTENTS (Cont'd)

A.1. INTRODUCTION

The following design examples will be used to illustrate the differences in application and use of the proposed code versus the current AASHTO specification. The intermediate span structure will be used for the examples. The sections investigated are shown in Figure A-1. The methodology will be to design the section first using the AASHTO specification and then a second time by the proposed code.

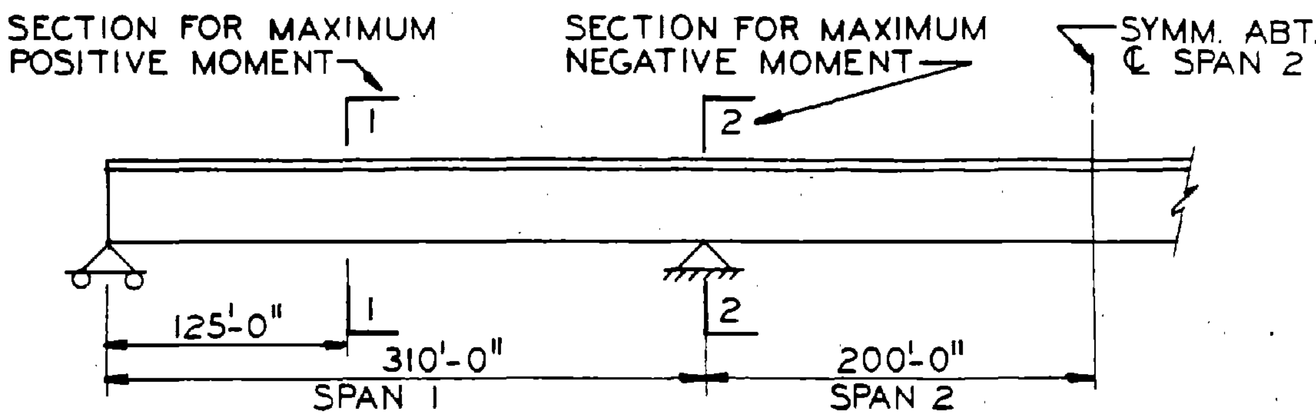


FIGURE A-1. PART ELEVATION OF STRUCTURE SHOWING SECTIONS TO BE DESIGNED

A.2. DESIGN FOR MAXIMUM POSITIVE MOMENT BY AASHTO SPECIFICATIONS

A.2.1. General Information

The main material of the box assumed at this location for the elastic analysis is shown in Figure A-2. The resulting moments and shears are shown in Table A-1. The live load used was AASHTO HS-20. The dead load included a 25 psf wearing surface and 410 plf for each parapet. The live load values in Table A-1 result from 2.7 lanes loaded (3 lanes x .9 reduction factor). Torque moments result from lanes being shifted to the right or left gutterline.

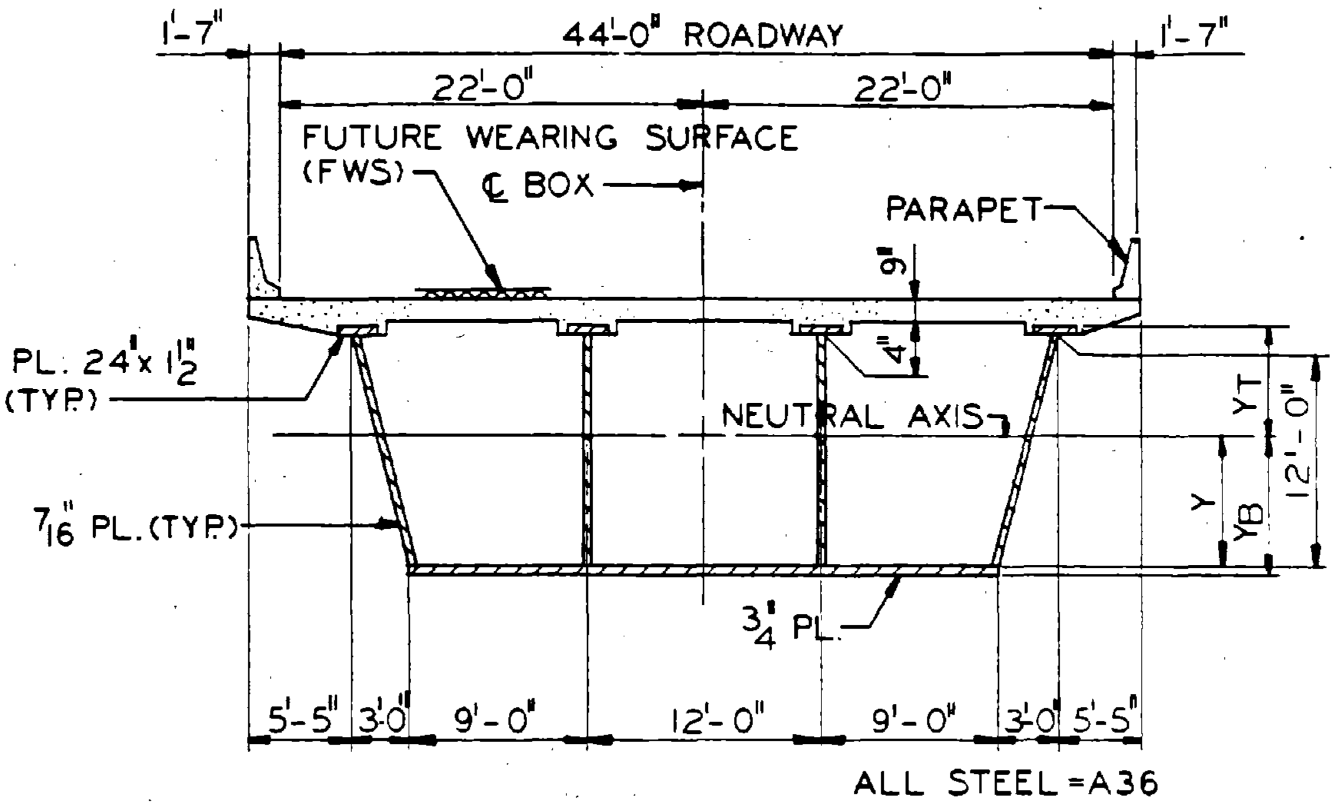


FIGURE A-2. CROSS-SECTION FOR POSITIVE MOMENT DESIGN

TABLE A-1. MOMENTS AND SHEARS FOR POSITIVE MOMENT DESIGN - FULL BOX - UNFACTORED

	Sign Convention	Non-Composite Dead Load	Superimposed Dead Load	Live Load + Impact Max. Moment	Max. Shear
Longit. Moment (ft.-kip)	$\boxed{+}$	49,370	11,990	22,530	16,410 / 6212
Torque Moment (ft.-kip)	$\boxed{+}$	-	-	406	730 / 192
Shear (kip)	$\boxed{+}$	-150	-29	60 / 6	131 / -113

A.2.2. Section Properties

The effective width of the top slab is computed from Article 1.7.48(C) and is found to be 108 in. per web or 432 in. for the full box. The bottom flange effective width is found from Article 1.7.64(D) and equals the full flange width. Table A-2 summarizes the resulting section properties. Note that 4-WT7X11 bottom flange longitudinal stiffeners not shown in Figure A-2 are included in the computations for properties.

TABLE A-2. SECTION PROPERTIES - POSITIVE MOMENT DESIGN - AASHTO

Loading Application	Y (in.)	I (in.^{4})	Y_T (in.)	Y_B (in.)	S_T (in.^{3})	S_B (in.^{3})
NCDL	57.5	2,524,400	88	58.25	28,690	43,340
SDL (n = 24)	75.7	3,705,900	69.8	76.45	53,090	48,470
LL+I (n = 8)	97	5,086,900	48.5	97.75	104,880	52,040

A.2.3. Slenderness and Bracing Requirements

a) Top Flange

The width to thickness ratio is governed by Article 1.7.61(C).

$$b'/t \leq 2200 \sqrt{1.3\ f_{DL}}$$

Conservatively, say f_{DL} = 36,000 psi.

Therefore, $b'/t \leq 2200 / \sqrt{1.3 \times 36{,}000} = 10.2$

Actual $b'/t = 12/1.5 = 8 < 10.2$ O.K.

Check top flange lateral bracing using Article 1.7.59(D)(1). Use unbraced length, L_b = 25 ft.

$$M_u = F_y\, S\left[1 - \frac{3F_y}{4\pi^2 E}\left(\frac{L_b}{b'}\right)^2\right]$$

note: b' = .9 x 12 = 10.8 in. (Article 1.7.60(A))

$$= 36 \times 28{,}690\left[1 - \frac{3 \times 36}{4 \times \pi^2 \times 29{,}000}\left(\frac{25 \times 12}{10.8}\right)^2\right] \times 1/12$$

$$= 79{,}800 \text{ ft. - kips}$$

Dead load factored moment equals 64,180 ft.-kips (49,370 x 1.3). Therefore, buckling does not control.

b) Web

Article 1.7.60(B) or (C) governs. Maximum web depth, D, of the inclined web is approximately 148.5 in.

Check whether D or D_c is to be used,

$$D_c = 144 - 97 = 47 \text{ in.} < D/2$$

Therefore, D is used in code requirements.

Actual depth to thickness ratio:

$$D/t = 148.5/.4375 = 339$$

Allowable for transverse stiffener only:

$$D/t \leq 36{,}500/\sqrt{36{,}000} = 192$$

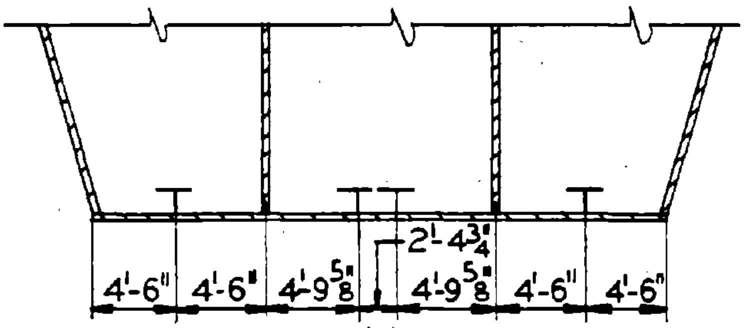
Allowable for transverse and longitudinal stiffeners:

$$D/t \leq 73{,}000/\sqrt{36{,}000} = 385$$

Therefore, a longitudinal stiffener is required. Note that a 3/8 in. web is not permissible since resulting D/t violates the allowable.

c) Bottom Flange

See Figure A-3(a) for the arrangement of longitudinal flange stiffeners. This arrangement results from eliminating six out of ten stiffeners used in the negative moment area, see Section A.4.



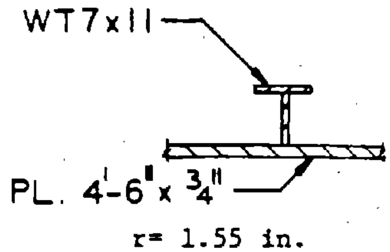


FIGURE A-3. BOTTOM FLANGE STIFFENERS - POSITIVE MOMENT DESIGN - AASHTO

For design by the AASHTO code, a limit on plate slenderness of 120 was arbitrarily assumed. The maximum width-thickness ratio is

$$(w/t)_{max} = 54/.75 = 72 \qquad \text{O.K.}$$

The limit for the slenderness ratio, L/r, for the stiffeners of the tension flange was taken as 200 based on Article 1.7.5. Using an unbraced length of 25 ft.,

$$L/r = 25 \times 12/1.55 = 194 \qquad \text{O.K.}$$

Note that section is non-compact since the requirements of Article 1.7.59(A) are not satisfied. Therefore, the steel stresses should not exceed the yield stress or the buckling strength of the plates as applicable.

A.2.4. Flange Design

Table A-3 summarizes the maximum stresses at the top and bottom flanges.

TABLE A-3. STRESS RESULTS - POSITIVE MOMENT DESIGN - AASHTO

Location	NCDL (ksi.)	SDL (ksi.)	LL+I (ksi.)	Total (ksi.)
Top Flg.	26.8	3.5	5.6	35.9
Bott. Flg.	17.8	3.9	11.3	33.0

Since the bottom flange is somewhat understressed, reduce thickness to 11/16 in. Resulting section properties are shown in Table A-4 and stresses in Table A-5.

TABLE A-4. REVISED SECTION PROPERTIES - POSITIVE MOMENT DESIGN - AASHTO

Loading Application	Y (in.)	I ($in.^4$)	Y_T (in.)	Y_B (in.)	S_T ($in.^3$)	S_B ($in.^3$)
NCDL	59.5	2,445,600	86	60.19	28,440	40,630
SDL (n = 24)	77.8	3,570,700	67.7	78.49	52,740	45,490
LL+I (n = 8)	98.9	4,867,000	46.6	99.59	104,440	48,870

TABLE A-5. REVISED STRESS RESULTS - POSITIVE MOMENT DESIGN - AASHTO

Location	NCDL (ksi.)	SDL (ksi.)	LL+I (ksi.)	Total (ksi.)
Top Flg.	27.1	3.5	5.6	36.2
Bott. Flg.	19.0	4.1	12.0	35.1

Stresses for the revised section are acceptable. The bottom flange slenderness is O.K. by inspection. Change in section properties would not be significant enough to require a reanalysis of structure.

A.2.5. Web Design

The design of webs will be based on one-fourth of the direct load being carried by each web. The approximate shear flow due to torsion in the outer webs and flanges is equal to

$$Q = .00119\ M_{TORQUE}$$

where M_{TORQUE} is in ft.-kips and resulting shear flow is in kip/ft.

Table A-6 is a summary of the factored moments and shears as computed using the values from Table A-1.

TABLE A-6. FACTORED MOMENTS AND SHEARS - POSITIVE MOMENT DESIGN - AASHTO

Loading Case	Total Mom. Full Box (ft.-kip)	Shear- Ext. Web (kip)
Max. Mom.	128,580	-70
Max. Shr.	97,290	-129

Check need for transverse stiffeners per Article 1.7.59(E)(5). Maximum factored shear force allowable without transverse stiffeners is

$$V_u = 1.015 \times 10^8 \times t_w{}^3/D \times 10^{-3}$$
$$V_u = 1.015 \times 10^5 \times .4375^3/148.5 = 57. \text{ kips}$$

which is less than the maximum shear shown in Table A-6. Therefore, transverse stiffeners are required.

Try stiffener spacing equal to the maximum allowed by AASHTO, Article 1.7.59(E)(5).

$$d_o = 1.5D = 1.5 \times 144 = 216 \text{ in.}$$

Per Article 1.7.59(F)(3) the shear capacity is computed using Article 1.7.59(E)(3). With

t_w = .4375 in.
D = 148.5 in.
d_o = 216 in.
F_y = 36 ksi.

it can be shown that

C = .0392
V_u = 696 kips

Therefore, maximum stiffener spacing is adequate and controls.

Check for possible moment capacity reduction as per Article 1.7.59(E)(4). For the maximum moment case, $V/V_u = 70/696 = .1$. Since ratio is less than .6, no reduction is required.

Since cross-frames are spaced at 25 ft. intervals, intermediate stiffeners will be required at 12.5 ft. spaces. For this value of d_o,

C = .0933
V_u = 879 kips

from Article 1.7.59(E)(3).

A.2.6. Web Stiffener Design

a) Longitudinal Stiffener

The provisions of Article 1.7.59(F)(3) (a) through (c) govern.

a) $b'/t \leq 2{,}600/\sqrt{F_y} = 13.7$ for $F_y = 36{,}000$ psi.

b) $I \geq Dt_w^3 \left[2.4(d_o/D)^2 - .13\right]$

$\geq 28.8 \text{ in.}^4$ for subject panel

c) $r \geq d_o \times \sqrt{F_y} \;/23{,}000$

1.24 in.

Try plate 6 x 1/2 in. (A36) with web plate equal to $18t_w$ acting with it.

$$b'/t = 6/.5 = 12 \quad \text{O.K.}$$

$$I \cong 6^3 \times .5/3 = 36 \text{ in.}^4 \quad \text{O.K.}$$

$$A = 6 \times .5 + 18 \times .4375^2 = 6.45 \text{ in.}^2$$

$$r = \sqrt{36/6.45} = 2.36 \text{ in.} \quad \text{O.K.}$$

Therefore, use plate 6 x 1/2 in. for longitudinal stiffener.

b) Transverse Stiffener

The provisions of Article 1.7.59(E)(5) apply with depth of subpanels used instead of total panel depth as stipulated in Article 1.7.59(F)(3). Stiffeners will be designed as a single plate from A36 material.

The width-to-thickness requirement is the same as for longitudinal stiffeners.

$$b'/t \leq 13.7$$

The minimum area requirement is

$$A = \left[.15\ BDt_w(1-C)\ (V/V_u) - 18t_w^2\right]Y$$

where B = 2.4 for single plate
D = subpanel depth $\cong$ .8 x 148.5 = 118.8 in.
t_w = .4375 in.
C = .0933
V = 129 kips
V_u = 879 kips
Y = 1.0

Therefore, minimum $A = -.96 \text{ in.}^2$ and area requirement does not control.

The minimum moment of inertia required is

$$I = d_o\, t_w^3\, J$$

where

$$J = 2.5\ (D/d_o)^2 - 2 \geq .5$$

For subject panel:

$$J = .5$$

and

$$I = 6.3\ \text{in.}^4$$

Try plate 4 x 3/8 in.:

$$I \cong .375 \times (4 + .4375/2)^3/3 = 9.4\ \text{in.}^4 \quad \text{O.K.}$$
$$b'/t = 4/.375 = 10.7 \quad \text{O.K.}$$

Since a longitudinal stiffener is used, Article 1.7.59(F)(3) also requires that

$$S_t \geq 1/3 \times (D/d_o) \times S_1$$

where S_t = section modulus of transverse stiffener
D = full panel depth
S_1 = section modulus of longitudinal stiffeners

For plate 6 x 1/2 in. longitudinal stiffener:

$$S_1 = 36/6 = 6\ \text{in.}^3$$

Therefore,

$$S_t \geq 1/3 \times (148.5/150) \times 6 = 2\ \text{in.}^3$$

Actual section modulus of 4 x 3/8 in. plate:

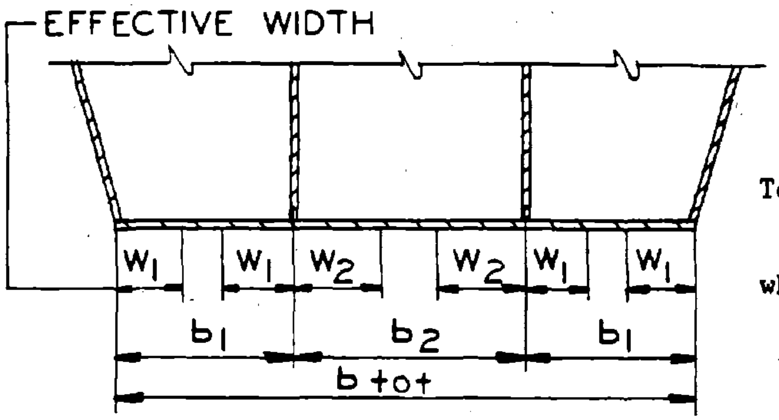
$$9.4/4.22 = 2.2\ \text{in.}^3 \qquad \text{O.K.}$$

Therefore, a plate 4 x 3/8 in. is adequate for transverse stiffeners at this location. Note that in the final design, a larger stiffener plate is used. This is because the stiffener is designed to satisfy the requirements of a range of locations rather than just one location.

A.3. DESIGN FOR MAXIMUM POSITIVE MOMENT BY THE PROPOSED SPECIFICATION

A.3.1. Shear Lag Effect

As per Article 1.7.208(B), the effect of a non-uniform longitudinal stress distribution due to shear lag must be investigated for the bottom flange. Article 1.7.204 is used to compute the effective flange width. Figure A-4 illustrates the effective width concept. Using an L of 215 ft corresponding to the distance from end support to the point of NCDL contraflexure and assuming the stiffener area A_s is equal to zero, the resulting values of ψ_1 and ψ_2 from Figure 1.7.204 are 1.0. Therefore, the average flange stress equals the peak stress. By Article 1.7.206(B)(4), a uniform distribution of stresses in the ultimate condition is assumed with the bottom flange fully effective.



$$w_1 = \frac{\psi_1 b_1}{2}$$

$$w_2 = \frac{\psi_2 b_2}{2}$$

Total Effective Width

$$= b_{tot}(.6\psi_1 + .4\psi_2)$$

when $b_1 = .3b_{tot}$

$b_2 = .4b_{tot}$

FIGURE A-4. EFFECTIVE WIDTHS

Top flanges are discussed in Article 1.7.209 from which it is determined that the current AASHTO treatment of effective widths for concrete slabs is applicable.

The section as designed by the AASHTO code will be used as the starting point for the design by the proposed code. Therefore, the section properties shown in Table A-4 are appropriate. Note that the proposed code states that axial flange stresses should be computed at the mid-plane of the flange plate (for example, Article 1.7.208(D)(1)). Extreme fiber stresses have been used in the AASHTO design. Any differences are neglected for this example and extreme fiber stresses will be used.

A.3.2. Slenderness and Bracing Requirements

a) Top Flange

The steel top flange is treated the same as in the AASHTO Code, therefore, the checks of Section A.2.3.(a) are adequate.

b) Web

Article 1.7.212(C)(1) applies for webs with one longitudinal stiffener. There are two requirements.

1) $D/t_w \leq \frac{13.6}{\sqrt{F_y/E}}$

≤ 386 for A36 steel

2) $D'/t_w \leq \frac{2.7}{\sqrt{F_y/E}}$

≤ 77 for A36 steel

where D = clear depth of web

D' = depth of subpanel adjacent to compression flange $\geq 2D_c/5$

D_c = clear distance between neutral axis and compression flange

In the current AASHTO code, the value of D_c is computed using the composite section for live load. This procedure has also been used for the proposed code. However, this is only designer interpretation as the proposed code does not specifically address this point. Note that requirement 1) is the same as AASHTO. For the trial section

$$\begin{aligned} D' &= .2D \\ &= .2 \times 148.5 \\ &= 29.7 \text{ in.} \end{aligned}$$

$D'/t_w = 29.7/.4375 = 68 < 77$ O.K.

$D_c = 144 - 98.9 = 45.1$ in.

$2D_c/5 = 2 \times 45.1/5 = 18 < 29.7$ O.K.

Therefore, the web satisfies the proposed code requirements.

c) Bottom Flange

Article 1.7.208(G) contains specific slenderness limitations for bottom flanges in tension. Assume arrangement of longitudinal stiffeners is as shown in Figure A-3 with an 11/16 in. flange plate. The radius of gyration of the stiffener strut shown in Figure A-3(b) with an 11/16 in. flange is 1.60 in.

The plate slenderness, w/t, limitation of 120 is the same as used previously in the AASHTO design. Therefore, the plate slenderness is within the allowable. The slenderness ratio, L/r, of the stiffeners is limited to 120 in the proposed code. Using an unbraced length of 25 ft:

$$L/r = 25 \times 12/1.6 = 188$$

Therefore, the stiffener arrangement used for the AASHTO design is not adequate using the proposed code. To achieve an L/r ratio of about 120, the number of stiffeners must be increased to a total of ten and the stiffener size must be increased to a WT8X13.

A.3.3. Flange Design

The stiffeners added for slenderness reasons to the bottom flange are also effective in resisting moment. A 5/8 in. flange plate in conjunction with the 10-WT8X13 longitudinal stiffeners provides 263.4 $in.^2$ of flange area. This compares with the 260.5 $in.^2$ provided by an 11/16 in. flange with 4-WT7X11 stiffeners used for the AASHTO design. Therefore, a 5/8 in. flange plate will be used and the stresses will be assumed to be the same as shown in Table A-5, the slight change in area being negligible.

A.3.4. Web Design

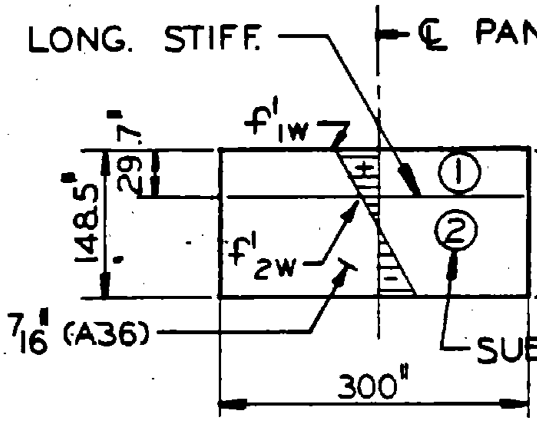
a) Panel Description

The factored moments and shears shown in Table A-6 will be used for the web design by the proposed code. Article 1.7.212 applies to transversely and longitudinally stiffened webs. The basic method is that covered in Article 1.7.211 with some modifications to accommodate the additional web panels.

Per Article 1.7.211(C), the moments and shears used in the web design are to be those at the center of the web panel under consideration. For this example, it will be assumed that section 1 shown in Figure A-1 corresponds to a panel centerline.

As regards design stresses, Article 1.7.211(C) refers to "the governing load-factored coincident shear and flexural or direct stresses" for use in web design. It is not clear exactly what this means. For purposes of this design example, the web is investigated for two cases: 1) maximum moment plus coincident shear; and 2) maximum shear plus coincident moment. Of course, there may be other coincident loading cases where neither the moment nor shear is maximum, but which nevertheless govern. No attempt has been made to identify these cases.

Since cross frames have been assumed at 25 ft. spaces, try this spacing for the panel length. Figure A-5 shows the panel and stresses required to compute the shear capacity.



TOTAL STRESSES

MOMENT		SHEAR	
Max. Mom.	Max. Shr.	Max. Mom.	Max. Shr.
35.5 ksi	31.6 ksi		
21.4	20.0	1.08 ksi	2.0 ksi
-34.8	-26.2		

FIGURE A-5. WEB PANEL - POSITIVE MOMENT DESIGN - PROPOSED

b) Critical Shear Buckling Stress, F_{vcr}, and Strength, V_B

F_{vcr} is a function of the applied loading and the following critical buckling stresses.

F^o_{vcr} = Due to shear stress acting alone

F^o_{bcr} = Due to bending acting along

F^o_{ccr} = Due to axial compression acting alone

An interaction equation is used to find the value of F_{vcr}, see Article 1.7.211(B)(4).

$$\left(\frac{F_{vcr}}{F^o_{vcr}}\right)^2 + \left(\frac{F_{bcr}}{F^o_{bcr}}\right)^2 + \left(\frac{F_{ccr}}{F^o_{ccr}}\right) = 1$$

F_{vcr}, F_{bcr}, and F_{ccr} are the individual shear, pure bending and pure compression stress components which when acting simultaneously cause buckling. F_{bcr} and F_{ccr} are related to F_{vcr} by the following equations.

$$F_{bcr} = \frac{1-R}{2}\mu F_{vcr} \quad \text{where } R = \frac{f'_{2w}}{f'_{1w}}$$

$$F_{ccr} = \frac{1+R}{2}\mu F_{vcr} \quad \text{where } \mu = \frac{f'_{1w}}{f_v}$$

f'_{1w} and f'_{2w} are as illustrated for subpanel (1) in Figure A-5 and f_v is the shear stress. If $R < -1$, the interaction equation reduces to

$$\left(\frac{F_{vcr}}{F^o_{vcr}}\right)^2 + \left(\frac{F_{bcr}}{F^o_{bcr}}\right)^2 = 1$$

with $F_{bcr} = \mu F_{vcr}$

A value of F_{vcr} is found for each subpanel. The minimum value of F_{vcr} so found is assumed to be the governing value for the entire web depth and is used to determine the buckling strength, V_B, of the web.

For subpanel (1), the values F^o_{vcr}, F^o_{bcr}, and F^o_{ccr} are determined as follows.

F^o_{vcr}:

$$\alpha' = d_o/D' = 300/29.7 = 10.1$$

$$K_v = 5 + 5/(\alpha')^2 = 5.049$$

$$\lambda_v = \frac{.8D'}{t_w}\sqrt{\frac{F_y}{E\ K_v}} = .85$$

From Table 1.7.211(B)(2) of proposed code,

$$F^o_{vcr} = \left[.58 - .357\ (\lambda_v - .58)^{1.18}\ F_y\right] = 18.1 \text{ ksi.}$$

F^o_{bcr}:

Since $\alpha' > 2/3$, by Article 1.7.211(B)(3) it is found that $K = 24$

and

$$\lambda = \frac{D'/t_w}{.95}\sqrt{\frac{F_y}{EK}} = .51 < .65$$

therefore, $F^o_{bcr} = F_y = 36$ ksi.

F^o_{ccr}:

Similarly, since $\alpha' > 1$, $K = 4$ and $\lambda = 1.26$

therefore,

$$F^o_{ccr} = \left[.072\ (\lambda - 5.62)^2 - .78\ F_y\right] = 21.2 \text{ ksi.}$$

For subpanel (1):

$R = 21.4/35.5 = .603$ Max. Mom. Case
$R = 20/31.6 = .633$ Max. Shr. Case

and $\mu = 35.5/1.08 = 32.9$ Max. Mom. Case
$\mu = 31.6/2.0 = 15.8$ Max. Shr. Case

All the terms for the interaction equation are now known or can be determined. It can be shown that

$F_{vcr} = 0.8$ ksi. Max. Mom. Case
$F_{vcr} = 1.62$ ksi. Max. Shr. Case

In a similar fashion to that used for subpanel (1), subpanel (2) can be solved as follows.

$F^{o}_{vcr} = 2.06$ ksi.

$F^{o}_{bcr} = 8.65$ ksi.

$R = -1.63$ Max. Mom. Case
$R = -1.31$ Max. Shr. Case
$\mu = 19.8$ Max. Mom. Case
$\mu = 10.0$ Max. Shr. Case

and $F_{vcr} = 0.43$ ksi. Max. Mom. Case
$F_{vcr} = 0.80$ ksi. Max. Shr. Case

As is seen, subpanel (2) governs for both cases and the shear buckling capacities are,

Maximum Moment Case: $V_B = Dt_w\ (F_{vcr})_{min} = 148.5 \times .4375 \times .43 = 28$ kips

Maximum Shear Case: $V_B = 52$ kips

c) Tension Field Stress, F_T, and Strength, V_T

Per Article 1.7.212(B)(5), the tension field strength is computed for the entire web panel disregarding the longitudinal stiffeners. The value of F_T is computed from Article 1.7.211(B)(5).

$$F_T = F_y - \sqrt{.25\ f_{2w}^2 + 3F_{vcr}^2}$$

where f_{2w} = axial web stress at opposite edge from maximum compression stress

F_{vcr} = taken as $(F_{vcr})_{min}$

For the two cases,

Maximum Moment: F_T = 18.6 ksi.
Maximum Shear: F_T = 22.8 ksi.

The value of V_T is found from Article 1.7.211(B)(1).

$$V_T = \frac{Dt_w F_T}{2\left(\sqrt{1+\alpha^2}+\alpha\right)}$$

where α is the aspect ratio for the entire panel, 300/148.5 = 2.02. Therefore, for

Maximum Moment: V_T = 141 kips

Maximum Shear: V_T = 173 kips

d) Ultimate Shear Capacity

The ultimate shear capacity, V_u, is defined as follows.

$$V_u = V_B + V_T$$

Therefore, for the two cases considered,

Maximum Moment: V_u = 169 kips $>$ 70 kips O.K.

Maximum Shear: V_u = 225 kips $>$ 129 kips O.K.

The maximum value of V_u is limited as shown below:

$$(V_u)_{max} = .58Dt_w\sqrt{F_y^2 - (2/3\ f_{av})^2}$$

where f_{av} = (35.5 + 34.8)/2 = 35.2 ksi.

therefore, $(V_u)_{max}$ = 1030 kips O.K.

e) Additional Flange Forces Due to Web Post-Buckling Behavior

The additional flange forces are computed using Article 1.7.211(E).

$$\Delta F_1 = \frac{V_M - \Sigma V_B}{V_M}\left[\left(f_{1R} - f_1\right) A_{fc} + 1/2\ V_M \cot\left(\frac{\theta_d}{2}\right)\right]$$

$$\Delta F_2 = \frac{V_M - \Sigma V_B}{V_M}\left[\left(f_{2R} - f_2\right) A_{ft} - 1/2\ V_M \cot\left(\frac{\theta_d}{2}\right)\right]$$

where ΔF_1 = additional force to compression flange

ΔF_2 = additional force to tension flange

V_M = total load factored shear force on box

ΣV_B = sum of buckling shear capacities of all webs

f_1, f_2 = stress in compression or tension flange from elastic analysis, fully participating web

f_{1R}, f_{2R} = stress in compression or tension flange from elastic analysis based on reduced moment of inertia, I_R, computed by removing the web in compression

A_{fc}, A_{ft} = compression or tension flange area

$\theta_d = \cot^{-1}(\alpha)$

Additional flange forces are computed for the maximum moment case only. It is not clear how to compute and apply the additional forces for a composite section. The procedure used here is to find the force $(f_R - f)A$ for each stage of analysis and sum them to obtain the total contribution. Conservatively, the additional forces have been assumed carried by the steel flanges only.

The reduced moments of inertia and the forces $(f_{1R} - f_1)A_{fc}$ are shown in Table A-7.

TABLE A-7. FORCES $(f_{1R} - f_1)A_{fc}$

Loading	I (in.^{4})	I_R (in.^{4})	I_R/I	f_{1R} (ksi.)	f_1 (ksi.)	A_{fc} (in.^{2})	$(f_{1R}-f_1)A_{fc}$ (kips)
NCDL	2,445,600	2,093,650	.86	31.5	27.1	144	634
SDL (n = 24)	3,570,700	3,401,500	.95	3.68	3.5	306	55
LL+I (n = 8)	4,867,000	4,813,500	.99	5.66	5.6	630	38
							727

The forces $(f_{2R} - f_2)\,A_{ft}$ are shown in Table A-8.

TABLE A-8. FORCES $(f_{2R} - f_2)A_{ft}$

Loading	f_{2R} (ksi.)	f_2 (ksi.)	A_{ft} (in.^{2})	$(f_{2R} - f_2)A_{ft}$ (kips)
NCDL	22.1	19.0	263	815
SDL (n = 24)	4.32	4.1	263	58
LL+I (n = 8)	12.1	12.0	263	26
				899

The total factored shear force, V_M, is

$$V_M = 1.3\ (150 + 29 - 5/3 \times 6) = 220 \text{ kips and}$$

$$\sum V_B = 4 \text{ webs} \times 28 = 112 \text{ kips}$$

$$\theta_d = \tan^{-1}\ (D/d_o) = 26.3°$$

Therefore,

$$\Delta F_1 = \frac{(220-112)}{220}\left[727 + .5 \times 220 \times \cot\left(\frac{26.3}{2}\right)\right] = 588 \text{ kips}$$

$$\Delta F_2 = \frac{(220-112)}{220}\left[899 - .5 \times 220 \times \cot\left(\frac{26.3}{2}\right)\right] = 210 \text{ kips}$$

The top flange will need to be increased. Try a 27 x 1½ in. plate. Approximate resulting stress:

$$f_T \cong (36.2 \times 144 + 588)/(4 \times 27 \times 1.5) = 35.8 \text{ ksi.} \quad \text{O.K.}$$

Bottom flange stress on original section:

$$f_B = (35.1 + 210/263) = 35.9 \text{ ksi.} \quad \text{O.K.}$$

Therefore, as a first approximation, the top flange will be increased to a 27 x 1½ in. plate. This design example assumes this plate as the final size disregarding the fact that a more rigorous solution requires a recycling through the moment analysis.

A.3.5. Web Stiffener Design

a) Longitudinal Stiffener

Article 1.7.213(D) applies to stiffeners in the compression zone of the web. The stiffeners must satisfy strength, rigidity, and stability requirements.

The minimum rigidity of the longitudinal stiffener, I_L, is

$$I_L = .09\ n\ m_L\ \gamma^*_{L(\sigma+\tau)}\ D t_w^3$$

where n = multiplier = 1 for one longitudinal stiffener
m_L = multiplier = 3 for webs with $D/t_w > 240$ and longitudinal stiffener at 0.2D

$\gamma^*_{L(\sigma+\tau)}$ = minimum relative rigidity coefficient of longitudinal stiffener

$$\gamma^*_{L(\sigma+\tau)} = \sqrt{\left[\gamma^*_{L\sigma}\left(\frac{f'_c}{F^o_{ccr}} + \frac{f'_b}{F^o_{bcr}}\right)\right]^2 + \left(\gamma^*_{L\tau}\ \frac{f'_v}{F^o_{vcr}}\right)^2}$$

where $\gamma^*_{L\sigma}$ and $\gamma^*_{L\tau}$ are determined from Fig. 1.7.213(B)

f'_c, f'_b, f'_v = smaller of f_c, f_b, f_v or F_{ccr}, F_{bcr}, F_{vcr}, respectively

f_c, f_b, f_v = actual pure compression, bending, and shear stresses in web subpanel. If axial stress is tension, disregard f'_c/F^o_{ccr} term.

and other terms are as previously defined.

All the stresses are to be computed from the subpanel with the lower value of F_{vcr}.

Substituting known values into the expression for I_L, yields:

$$I_L = .09 \times 1 \times 3 \times 148.5 \times .4375^3 \times \gamma^*_{L(\sigma+\tau)} = 3.36\ \gamma^*_{L(\sigma+\tau)}$$

Table A-9 shows the computation of $\gamma^*_{L(\sigma+\tau)}$ for the two cases of loading studied. Subpanel (2) is used to compute stresses. Note that for this panel the pure axial stress is tensile. A value of $\alpha = 2.0$ and R = -1.0 have been used to determine $\gamma^*_{L\sigma}$ and $\gamma^*_{L\tau}$. The values have been extrapolated from the available curves of the proposed code.

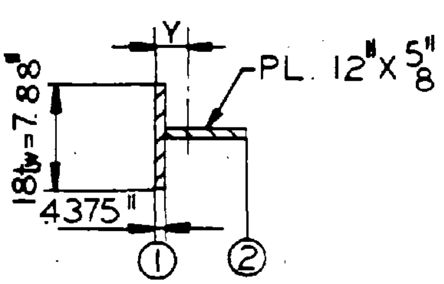
TABLE A-9. DETERMINATION OF $\gamma^*_{L(\sigma+\tau)}$

Loading Case	$\gamma^*_{L\sigma}$	$\gamma^*_{L\tau}$	f_b (ksi.)	F_{bcr} (ksi.)	F^o_{bcr} (ksi.)	f_v (ksi.)	F_{vcr} (ksi.)	F^o_{vcr} (ksi.)	$\gamma^*_{L(\sigma+\tau)}$
Max. Mom.	51	18	28.1	8.5	8.65	1.08	.43	2.06	50
Max. Shr.	51	18	23.1	8.0	8.65	2.0	.80	2.06	48

Therefore, the minimum value of I_L is

$$I_L = 3.36 \times 50 = 168 \text{ in.}^4$$

Trial stiffener section is 12 x 5/8 in. plate with properties as shown in Figure A-6.



A = 10.95 in.^{2}
Y = 4.48 in.
I = 181. in.^{4}
r = 4.07 in.
S_1 = 40.5 in.^{3}
S_2 = 22.8 in.^{3}

FIGURE A-6. LONGITUDINAL STIFFENER

As can be seen, the minimum moment of inertia requirement is met. To check strength use current AASHTO Article 1.7.69(A) and (B). From Article 1.7.213(D)(2)(a) of the proposed code, the design axial force is

$$F = 18 \times .4375^2 \times 21.4 = 74 \text{ kips}$$

and the design eccentricity is

$$e = 4.48 - .4375/2 + 300/500 = 4.86 \text{ in.}$$

Given the above applied force and eccentricity, the section can be shown to be adequate for strength.

The stability requirements of Article 1.7.213(C)(3) must also be checked. The maximum stress on the stiffener is approximately 15.8 ksi which is less than $.5F_y$ for A36 material. Therefore, the maximum value for C_s' is

$$C_s' = \frac{0.8}{\sqrt{F_y/E}} = 22.7$$

for the 12 x 5/8 in. plate:

$$C_s' = d/t_o = 12/.625 = 19.2 \qquad \text{O.K.}$$

b) Transverse Stiffener

As for the longitudinal stiffener, the transverse stiffener must satisfy strength, rigidity and stability requirements. Article 1.7.213(C) applies.

The minimum rigidity of the transverse stiffener, I_T, is

$$I_T = .09 m_T \, \gamma_T^* \, D t_w^3 \, \frac{f_v'}{F_{vcr}^o}$$

where m_T = multiplier = 1.5 for webs with $D/t_w \geq 150$

γ_T^* = minimum relative rigidity coefficient obtained from Figure 1.7.213(A) of proposed code or by an alternate method specified in Article 1.7.213(C)(2)(b).

Other terms are as previously defined or used. The code stipulates that the stresses f_v' and F_{vcr}^o are to be determined for the adjacent web panel or critical subpanel with the greater value of F_{vcr}. For this example, it will be assumed that the panel under consideration is the one with the greater value of F_{vcr}.

γ^*_T for an α equal to approximately 2.0 and the longitudinal stiffener at 0.2D is found to be about 3.0. Therefore,

$$I_T = .09 \times 1.5 \times 3.0 \times 148.5 \times (.4375)^3 \times .80/2.06 = 2.0 \text{ in.}^4$$

Values of f'_v and F^o_{vcr} have been taken from Table A-9 for the maximum shear case.

For strength considerations, the stiffener is treated as a column strut designed for the tension field vertical force, P_{VT}.

$$P_{VT} = \frac{F_T\, t_w\, d_o}{2}\left[1 - \frac{\alpha}{\sqrt{1+\alpha^2}}\right]\frac{V - V_B}{V_T}$$

with notation as previously defined. In addition, when longitudinal web stiffeners are present, the transverse stiffener column must be designed for an in-plane lateral force equal to 2% of the longitudinal stiffener capacity. Both P_{VT} and the lateral force are to be calculated in the panel that results in the larger value.

For the parameters previously determined:

$$P_{VT} = \frac{22.8 \times .4375 \times 300}{2}\left[1 - \frac{2.02}{\sqrt{1+2.02^2}}\right]\frac{129 - 52}{173} = 69 \text{ kips}$$

Note that the maximum shear case has been used.

The strength of the longitudinal stiffener shown in Figure A-6, can be shown to be approximately 120 kips. Two percent of this or 2.4 kips is applied to the stiffener column as shown in Figure A-7(a).

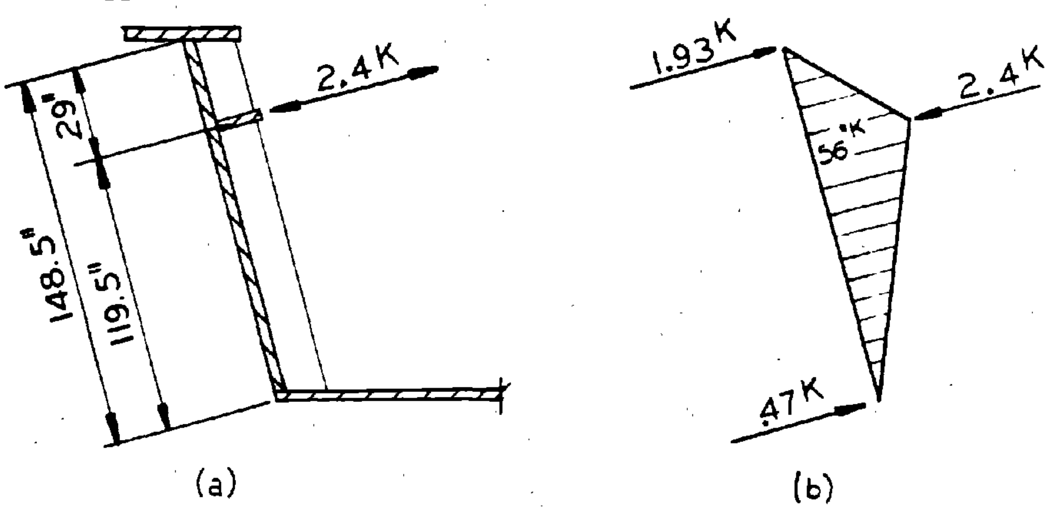


FIGURE A-7. LATERAL FORCE ON TRANSVERSE STIFFENER

The moment diagram on the stiffener due to the lateral force is shown in Figure A-7(b). The stiffener has conservatively been assumed to be pinned at top and bottom flanges.

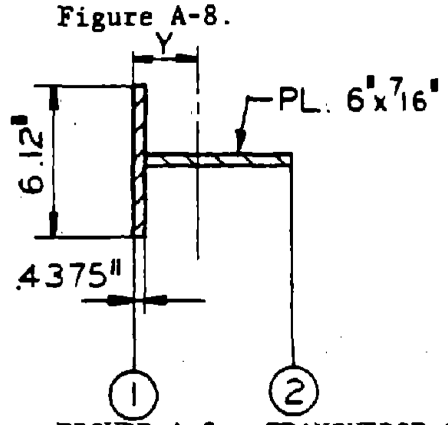
The effective width of the web acting with the transverse stiffener is given by

$$b_e = 18\left(1- \frac{V - V_B}{2V_T}\right) t_w = 18\left(1- \frac{129-52}{2x173}\right).4375 = 6.12 \text{ in.}$$

and the effective length of the stiffener column, L', is

$$L' = 0.7D = 0.7(148.5) = 104 \text{ in.}$$

Trial section is a 6 x 7/16 in. plate with properties as shown in Figure A-8.



A	= 5.31 in.^{2}
Y	= 1.81 in.
I	= 21.6 in.^{4}
r	= 2.02 in.
S_1	= 11.9 in.^{3}
S_2	= 4.67 in.^{3}

FIGURE A-8. TRANSVERSE STIFFENER

As can be seen, the rigidity requirement is met. It can be shown by AASHTO Article 1.7.69 that the stiffener is adequate for strength.

The resulting maximum stress on the stiffener can be shown to be 28.5 ksi. Therefore, the maximum value of C_s' allowed by the proposed code is

$$C_s' = \frac{.48}{\sqrt{F_y/E}} = 13.6 \text{ for A36 material}$$

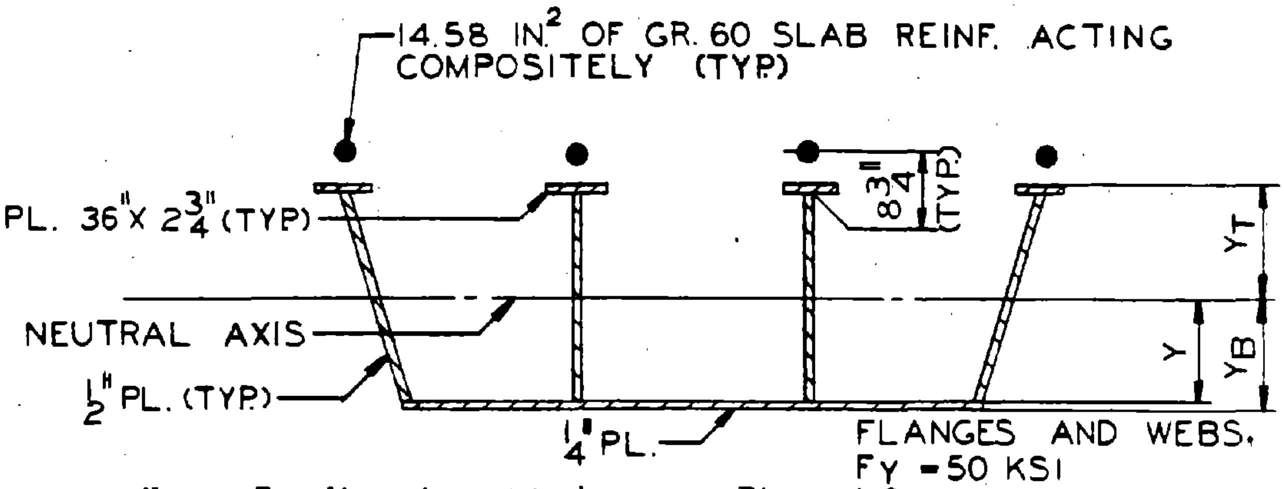
The actual value for the 6 x 7/16 in. plate is

$$C_s' = 6/.4375 = 13.7 \text{ Say O.K.}$$

A.4. DESIGN FOR MAXIMUM NEGATIVE MOMENT BY AASHTO SPECIFICATIONS

A.4.1. General Information

The main material of the box assumed at the interior support for the elastic analysis is shown in Figure A-9. The resulting moments and shears for this location are summarized in Table A-10. Note that the maximum live load moment and shear case coincide.



Note: For dimensions not shown, see Figure A-2.

FIGURE A-9. CROSS SECTION FOR NEGATIVE MOMENT DESIGN

TABLE A-10. MOMENTS AND SHEARS FOR NEGATIVE MOMENT DESIGN - FULL BOX - UNFACTORED LOADS

	Sign Convention	Non-Composite Dead Load	Super-imposed Dead Load	Live Load + Impact Max. Moment & Shear
Longit. Moment (ft.-kip)	+	-129,200	-27,600	-31,240
Torque Moment (ft.-kip)	+	-	-	-1680
Shear (kip)	+	-1843	-399	-428

Note: Torque moments and shears are those to left of joint.

A.4.2. Section Properties

The top flange and slab reinforcing are fully effective in resisting moments. The bottom flange effectiveness has been evaluated by extending Article 1.7.64(D) to the case of compression flanges. The bottom flange is found to be fully effective. Table A-11 summarizes the resulting section properties. The properties shown include 10-WT9X25 used to stiffen the 1-1/4 in. bottom flange.

TABLE A-11. SECTION PROPERTIES - NEGATIVE MOMENT DESIGN - AASHTO

Loading Application	Y (in.)	I (in.^{4})	Y_T (in.)	Y_B (in.)	S_T (in.^{3})	S_B (in.^{3})
NCDL	65.6	5,174,800	81.15	66.85	63,770	77,410
SDL+LL+I	69.6	5,597,400	77.15	70.85	72,550	79,000

A.4.3. Slenderness and Bracing Requirements

a) Top Flange

There are no specific slenderness limits for the top flange.

b) Web

$D_c = 69.6$ in.

$D/2 = 148.5/2 = 74.25$ in. $> D_c$

Therefore, Article 1.7.59(F) applies for webs stiffened transversely and longitudinally.

Allowable: $D/t = 73{,}000/\sqrt{50{,}000} = 326$

Actual D/t with 1/2 in. web:

$D/t = 148.5/.5 = 297$ O.K.

c) Bottom Flange

Figure A-10 shows the arrangement of longitudinal flange stiffeners used.

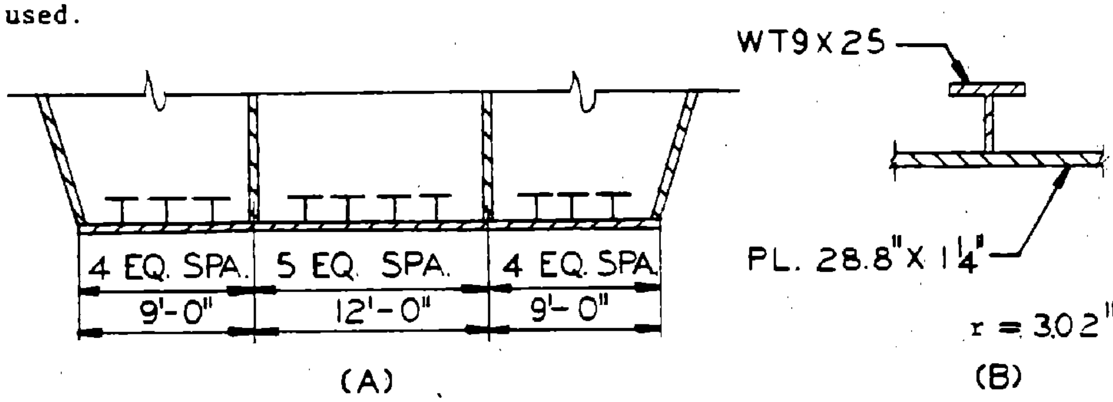


FIGURE A-10. BOTTOM FLANGE STIFFENERS - NEGATIVE MOMENT DESIGN - AASHTO

A modified form of Article 1.7.49(D)(4) and Article 1.7.51 is used to compute the critical buckling stresses for the compression flanges. There is no specific limit on L/r of the stiffener column. The plate width to thickness ratio, w/t, is not to exceed 60.

$$\therefore \quad w/t = 28.8/1.25 = 23 < 60 \qquad \text{O.K.}$$

The outstanding elements of the flange stiffeners are not to exceed

$$\frac{b'}{t'} = \frac{2600}{\sqrt{F_y}} = \frac{2600}{\sqrt{50{,}000}} = 11.6$$

For the WT9X25:

$$b'/t' \cong 7.495/(2 \times .570) = 6.6 < 11.6 \qquad \text{O.K.}$$

A.4.4. Flange Design

a) Strength of Flanges

The top (tension) flange can be designed to the yield point of the steel, 50 ksi. The strength of the bottom (compression) flange will be governed by buckling and is determined from an extension of Article 1.7.49 or from Article 1.7.51. The minimum strength so determined will govern. The strength determined from Article 1.7.49 consists of one of three cases depending on the w/t ratio of the flange

1) $F_{cr} = F_y$ when $w/t \leq \dfrac{3070\sqrt{k_1}}{\sqrt{F_y}}$

It can be shown that

$$k_1 = \frac{(1 + (a/b)^2\,)^2 + \;((n+1)\,(EI_i/bD))}{(n+1)^2(a/b)^2\;(1 + (n+1)\,(A_i/bt))} \qquad \text{for } a/b \leq 3.$$

where the notation is the same as in AASHTO except,

I_i = moment of inertia of one stiffener including an effective width of plate about an axis parallel to the flange

D = plate rigidity
= $Et^3/12(1 - \gamma^2)$

A_i = area of one stiffener

2) $F_{cr} = .592\, F_y \left[1 + .687 \sin (c\pi/2)\right]$

$$\text{where } c = \frac{6650\sqrt{k_1} \quad \frac{w}{t}\sqrt{F_y}}{3580\sqrt{k_1}}$$

$$\text{and when } \frac{3070\sqrt{k_1}}{\sqrt{F_y}} < \frac{w}{t} \leq \frac{6650\sqrt{k_1}}{\sqrt{F_y}} \qquad \text{but not to exceed 60}$$

$$3)\quad F_{cr} = 26.2 \times 10^6\ (t/w)^2\ k_1 \qquad \text{when } 60 > w/t > \frac{6650\sqrt{k_1}}{\sqrt{F_y}}$$

The strength of the flange can be determined from Article 1.7.51 by rearranging the equation for (L/r) max to solve for F_{cr} and assuming a factor of safety of 1/.55:

$$F_{cr} = 1/1485\left[1500\ F_y - \left(.001\ \frac{L}{r}\right)^2 F_y^{\ 2}\right]$$

For the flange shown in Figure A-10, the results of applying Article 1.7.49 as described above are summarized in Table A-12.

TABLE A-12. CRITICAL STRESS BY EXTENDED AASHTO 1.7.49

w (in.)	t (in.)	a (in.)	b (in.)	n	I_i (in.^{4})	A_i (in.^{2})	k_1	c	F_{cr} (ksi.)
28.8	1.25	240	144	4	396	7.33	1.09	.481	43.5

Using an L of 240 in. results in a value of F_{cr} of 39.9 ksi. from the use of Article 1.7.51.

Therefore, the strength of the flange shown in Figure A-10 is determined to be 39.9 ksi.

b) Stresses Due to Moment

Using the section properties found in Table A-11 and the moments from Table A-10, the stresses are found. These are summarized in Table A-13.

TABLE A-13. STRESS RESULTS - NEGATIVE MOMENT DESIGN - AASHTO

Location	NCDL (ksi.)	SDL (ksi.)	LL+I (ksi.)	Total (ksi.)
Top Flg.	31.6	5.9	11.2	48.7
Bott. Flg.	26.0	5.4	10.3	41.7

As is seen, the top flange section is adequate. The bottom flange stress is about 5 percent over the capacity. Since this stress occurs at the support where the flange will be supported transversely, buckling will not control and the bottom flange stress can be allowed to go up to the yield point. At the first panel quarter point (5 ft from the support), the stress has reduced to 39.3 ksi. Therefore, the bottom flange is adequate.

A.4.5. Web Design

The web is checked in a manner similar to that shown in Section A.2.5. Table A-14 is a summary of the factored moments and shears as computed using the values from Table A-10.

TABLE A-14. FACTORED MOMENTS AND SHEARS - NEGATIVE MOMENT DESIGN - AASHTO

Loading Case	Total Mom. -Full Box (ft.-kip)	Shear -Ext. Web (kips)
Max. Mom. or Max. Shr.	-271,530	-1044

Assume a transverse stiffener spacing, d_o, of 80 in. It can be shown that

$$C = 0.272$$
$$V_u = 1786 \text{ kips}$$

Since the ratio V/V_u is less than 0.6, no reduction in moment capacity is required.

A.4.6. Web Stiffener Design

a) Longitudinal Stiffener

Assume use of A36 steel. Requirements are the same as stated in Section A.2.6.(a) except for d_o = 80 in. and t_w = .5 in.

$$I \geq 10.5 \text{ in.}^4$$

$$r \geq .66 \text{ in.}$$

Try plate 5 x 3/8 in. with 9 in. of web acting with it.

$$b'/t = 5/.375 = 13.33 < 13.7 \qquad \text{O.K.}$$

$$I \cong 5^3 \times .375/3 = 15.6 \text{ in.}^4 > 10.5 \text{ in.}^4 \qquad \text{O.K.}$$

$$A = 5 \times .375 + 9 \times .5 = 6.375 \text{ in.}^2$$

$$r = \sqrt{15.6/6.375} = 1.56 \text{ in.} > .66 \text{ in. O.K.}$$

Therefore, plate 5 x 3/8 in. is adequate as longitudinal stiffener.

b) Transverse Stiffener

Assume use of A36 steel. The requirements are summarized below.

$$b'/t \leq 13.7$$

$$A \geq 6.4 \text{ in.}^2$$

$$I \geq 35 \text{ in.}^4$$

$$S_t \geq 1.93 \text{ in.}^3$$

Try plate 9 x 7/8 in.:

$b'/t = 9/.875 = 10.3 < 13.7$ O.K.

$A = 9 \times 7/8 = 7.9 \text{ in.}^2 > 6.4 \text{ in.}^2$ O.K.

$I = .875 \times (9 + .25)^3/3 = 231 \text{ in.}^4 > 35 \text{ in.}^4$ O.K.

$S_t = 231/9.25 = 25 \text{ in.}^3$ O.K.

Therefore, plate 9 x 7/8 in. stiffener from A36 material is adequate.

A.5. DESIGN FOR MAXIMUM NEGATIVE MOMENT BY THE PROPOSED SPECIFICATIONS

A.5.1. Shear Lag Effect

Article 1.7.204 is used to determine the effective width of the bottom flange. Refer to Figure A-4 for an illustration of terms. The bottom flange as designed by the ASSHTO code is used as a starting point for this design. Using an L of 190 ft and including the area of the WT9X25 stiffeners, Figure 1.7.204 is used to find the values of ψ_1 and ψ_2. The distance 190 ft corresponds to the distance between points of NCDL contraflexure in spans 1 and 2. The resulting values of ψ_1 and ψ_2 are

$$\psi_1 = .86$$

$$\psi_2 = .83$$

Therefore, the effective width of the bottom flange equals .85 of the actual width.

Per Article 1.7.206(B)(4) shear lag may be neglected if the peak stress, f_{max}, does not exceed the average stress, f_{avg}, by more than 20%.

Therefore, if $\left.\begin{array}{l} f_{max} \leq 1.2\ f_{avg} \\ \text{or } f_{avg} \geq .83\ f_{max} \end{array}\right\}$ then neglect shear lag.

From the distribution of stresses given in Article 1.7.204(A)(3), it can be shown that

$$f_{avg} = \psi f_{max}$$

Therefore, if $\psi > .83$ shear lag can be neglected and a uniform distribution of stresses can be assumed. This is seen to be the case for this example.

The top flange is fully effective.

A.5.2. Design Forces

Note that the proposed code requires that the forces for compression flange design be those at 0.4 of the panel length from the higher stressed panel end (see Article 1.7.206(C)). Similarly, for stiffened webs, the design forces are calculated midway between the transverse stiffeners. Therefore, the moments and shears shown in Table A-10 are not appropriate for design using the proposed code. Table A-15 summarizes moments and shears to be used for compression flange and web design. A panel length of 10 ft. was assumed for the compression flange. Transverse stiffener spacing was assumed to be 5 ft.

TABLE A-15. MOMENTS AND SHEARS - FULL BOX - UNFACTORED - NEGATIVE MOMENT DESIGN - PROPOSED SPECIFICATION.

		Non-Composite Dead Load	Superimposed Dead Load	Live Load + Impact
COMPRESSION FLANGE	Longit. Moment (ft.-kip)	-122,870	-26,200	-30,020
	Torque Moment (ft.-kip)	-	-	-1642
	Shear (kip)	-1801	-391	-420
WEB	Longit. Moment (ft.-kip)	-125,240	-26,730	-30,480
	Torque Moment (ft.-kip)	-	-	-1656
	Shear (kip)	-1817	-394	-423

A.5.3. Slenderness and Bracing Requirements

a) Section Used and Top Flange

The cross section shown in Figure A-9 is used as the basis for the design by the proposed code. The web thickness will be reduced to 7/16 in. by the use of multiple longitudinal stiffeners. The resulting properties are shown in Table A-16. There are no specific slenderness requirements for the top flange.

TABLE A-16. SECTION PROPERTIES - SECTION OF FIGURE A-9 WITH 7/16 IN. WEBS

Loading Application	Y (in.)	I (in.^{4})	Y_T (in.)	Y_B (in.)	S_T (in.^{3})	S_B (in.^{3})
NCDL	65.5	5,111,800	81.25	66.75	62,915	76,580
SDL+LL+I	69.6	5,535,080	77.15	70.85	71,740	78,120

b) Web

Article 1.7.212(C)(2) applies to webs with two or more lines of longitudinal stiffeners. The requirements are

$$\frac{D'_n}{t_w} \leq \frac{8.1}{\sqrt{F_y/E}} \frac{\eta_n D}{D_c}$$

$$\leq 195 \frac{\eta_n D}{D_c} \qquad \text{(for } F_y = 50 \text{ ksi.)}$$

and $\quad D'_1 \leq \frac{2D_c}{5}$

where $\quad D'_n$ = depth of web subpanel n

$\eta_n D$ = distance between compression flange and stiffener n

D'_1 = depth of subpanel adjacent to the compression flange

and other terms are as previously defined.

For this example, two longitudinal stiffeners were assumed, one at 0.2D, the other at 0.4D. Therefore,

$D'_1 = D'_2 = .2 \times 148.5 = 29.7$ in.

$D'_3 = 89.1$ in.

For subpanel 1:

$D'_n/t_w = 29.7/.4375 = 67.9$

$195\,\eta_n D/D_c = 195 \times 29.7/69.6 = 83.2 > 67.9$ O.K.

$2D_c/5 = 2 \times 69.6/5 = 27.8$ in. $>$ 29.7 in. say O.K.

Similarly, subpanel 2 and 3 are within the limits.

c) Bottom Flange

Article 1.7.206(D) contains the slenderness limits for the plate and attached stiffeners. Reference is made to Article 1.7.205(E) for the plate. The plate width-to-thickness ratio, w/t, is limited to a value of 60. This is the same value specified by AASHTO, therefore, the plate is within limits.

Article 1.7.207 is referred to for the longitudinal stiffeners. For a factored compression stress in excess of one-half the yield stress, the effective slenderness coefficient, C_s, is limited to a maximum value of

$$C_s = \frac{.40}{\sqrt{F_y/E}} = 9.6$$

For the tee stiffener:

$$C_s = \frac{d}{1.35\ t_o + .56\ r_y} + w/12t$$

where $\quad d$ = stiffener depth

t_o = stem thickness

r_y = stiffener radius of gyration about axis perpendicular to flange plate

w = width of plate between stiffeners

t = flange plate thickness

For the WT9X25 stiffener, C_s = 8.3. Article 1.7.207 also contains a limit for the width-to-thickness ratio of outstanding elements of a stiffener. This limit is the same as the current AASHTO. Therefore, the WT9X25 stiffener is adequate.

A.5.4. Flange Design

a) Strength of Flanges

The top flange can be designed to the yield point of the steel. The bottom flange strength is governed by buckling. Article 1.7.206 applies to stiffened bottom flanges in compression. The ultimate strength, F_u, is found from Figure 1.7.206(A). Two parameters, λ_{PL} and λ_{COL}, must be evaluated.

$$\lambda_{PL} = \frac{w/t}{1.9}\sqrt{\frac{F_y}{E}}$$

$$\lambda_{COL} = \frac{1}{\pi}\sqrt{\frac{F_y}{E}}\left(\frac{L}{r}\right)$$

where L = spacing of the transverse flange stiffeners

r = radius of gyration of one longitudinal stiffener including width of flange plate, w.

Other notation is as previously defined. For the flange shown in Figure A-10, with L = 120 in. and F_y = 50 ksi.

$$\lambda_{PL} = \frac{28.8/1.25}{1.9}\sqrt{\frac{50}{29{,}000}} = .50$$

$$\lambda_{COL} = \frac{1}{\pi}\sqrt{\frac{50}{29{,}000}}\left(\frac{120}{3.02}\right) = .53$$

Therefore, F_u/F_y = .87 from Figure 1.7.206(A) and F_u = 43.5 ksi.

b) Stresses Due to Moment

Using the section properties found in Table A-16 and the moments from Table A-10 or A-15, the stresses can be determined. The top flange stress is found at the centerline of support. The compression flange stress is at 0.4 of the first compression panel. The stresses are summarized in Table A-17.

TABLE A-17. STRESS RESULTS - NEGATIVE MOMENT DESIGN - PROPOSED

Location	NCDL (ksi.)	SDL (ksi.)	LL+I (ksi.)	Total (ksi.)
Top Flg.	32.0	6.0	11.3	49.3
Bott. Flg.	25.0	5.2	10.0	40.2

Therefore, the section is adequate. The effect of combined axial compressive stress and shear was found to be negligible.

A.5.5. Web Design

a) Panel Description

As mentioned previously, a spacing of 5 ft. has been assumed for the transverse stiffeners near the support. Figure A-11 shows the panel and stresses required to compute the shear capacity. The total factored moment and shear at the centerline of panel are summarized in Table A-18.

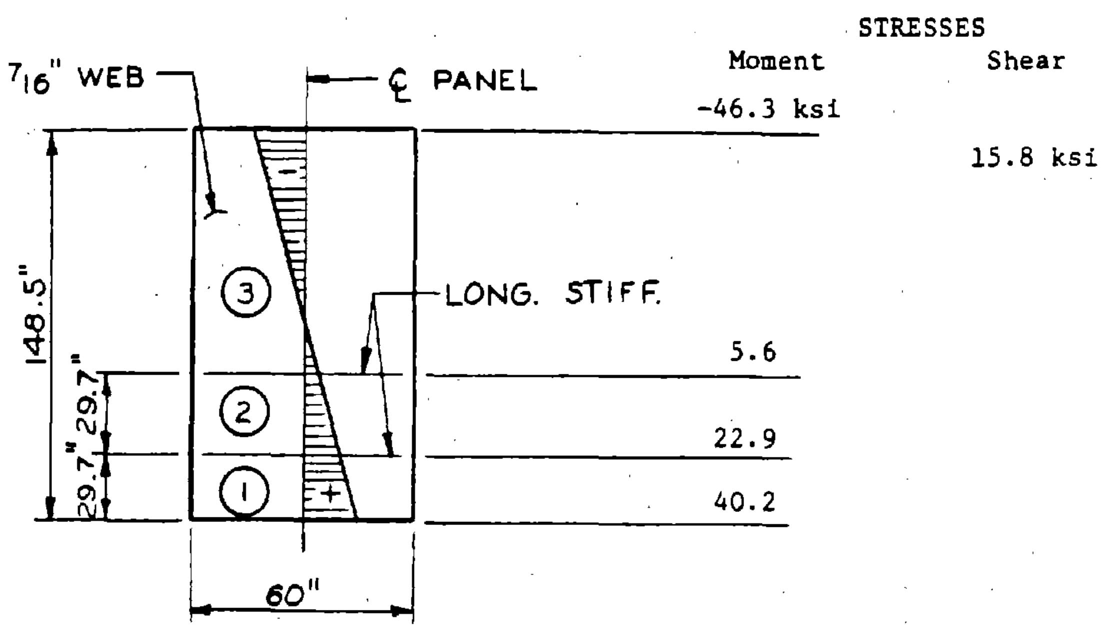


FIGURE A-11. WEB PANEL - NEGATIVE MOMENT DESIGN - PROPOSED SPECIFICATIONS

TABLE A-18. FACTORED MOMENTS AND SHEARS - NEGATIVE MOMENT DESIGN - PROPOSED

Loading Case	Total Mom. -Full Box (ft.-kip)	Shear -Ext. Web (kip)
Max. Mom. or Shr.	-263,700	-1029

b) Critical Shear Buckling Stress, F_{vcr}, and Strength, V_B

The computation of F_{vcr} for the three subpanels is similar to that illustrated in Section A.3.4.(b) of this design example and the details are not included here. A summary of the results for each panel is shown in Table A-19.

TABLE A-19. SHEAR BUCKLING STRESS - NEGATIVE MOMENT DESIGN - PROPOSED SPECIFICATIONS

	R	μ	F^{o}_{vcr} (ksi.)	F^{o}_{ccr} (ksi.)	F^{o}_{bcr} (ksi.)	F_{vcr} (ksi.)
Subpanel (1)	0.57	2.54	24.3	22.7	50.0	9.5
Subpanel (2)	0.24	1.45	24.3	22.7	50.0	15.0
Subpanel (3)	-8.26	0.35	10.2	-	15.1	9.9

From Table A-19, it is seen that subpanel (1) governs for F_{vcr}. Therefore,

$$V_B = 148.5 \times .4375 \times 9.5 = 617 \text{ kips}$$

c) Tension Field Stress, F_T, and Strength, V_T

From Article 1.7.211(B)(5)

$$F_T = 50 - \sqrt{.25(46.3)^2 + 3(9.5)^2} = 21.6 \text{ ksi.}$$

and from Article 1.7.211(B)(1)

$$V_T = \frac{148.5 \times .4375 \times 21.6}{2\left(\sqrt{1 + (60/148.5)^2} + 60/148.5\right)} = 473 \text{ kips}$$

d) Ultimate Shear Capacity

Knowing V_B and V_T results in

$$V_u = 617 + 473 = 1090 \text{ kips} > 1029 \text{ kips} \qquad \text{O.K.}$$

The maximum value for V_u is found from Article 1.7.211(B)(1).

$$(V_u)_{max} = 1538 \text{ kips} > 1029 \text{ kips} \qquad \text{O.K.}$$

e) Additional Flange Forces Due to Web Post-Buckling Behavior

In a similar fashion to that illustrated in Section A.3.4.(e), the additional flange forces can be found as follows.

$$\Delta F_1 \text{ (Compression Flange)} = 1139 \text{ kips}$$

$$\Delta F_2 \text{ (Tension Flange)} = -674 \text{ kips}$$

The forces have been computed at the location where the flange stresses were checked for bending.

Check the stress in the bottom flange including the effect of the additional force.

$$f_B \cong 40.2 + 1139/523.3 = 42.4 \text{ ksi.}$$

This total stress is less than the 43.5 ksi. ultimate strength computed in Section A.5.4.(a). Therefore, the bottom flange is adequate.

Some savings can be achieved in the top flange due to the reduction in total force. Try plate 35 x 2-3/4 in.

$$f_T \cong (49.3 \times 396 - 674)/(4 \times 35 \times 2.75) = 49.0 \text{ ksi.}$$

A.5.6. Web Stiffener Design

The web stiffener design is similar to that covered in Section A.3.5. and the details are not shown here. The required longitudinal stiffeners are,

at 0.2D: plate 6 x ½ in. (A36)
at 0.4D: plate 5 x 3/8 in. (A36)

and the transverse stiffener is

plate 10 x 7/8 in. (F_y = 50 ksi.)

APPENDIX B
AREAS FOR POSSIBLE IMPROVEMENT AND CLARIFICATION OF THE PROPOSED SPECIFICATION

This appendix lists and discusses certain areas of the proposed specification that could benefit from some minor rewording, additional comments, etc. These are not presented to criticize but rather to ensure that the proposed specification is fully understood by designers and is applied in the way the authors intend.

The comments are listed in order based on the section numbering of the proposed specification.

1.7.204(A)(3)

The term f_{max} is used in this section to denote the bending longitudinal stress at the flange-web junction. This term is also used in other areas of the code, e.g., Figure 1.7.205(B), Figure 1.7.206(B), Section 1.7.213(C)(3). The meaning is, however, different for each usage. While this is not always avoidable in a code of this size, it is objectionable since it leads to possible confusion. If it is not practical to use different subscripts for the different stresses referred to, possibly the term f_{max} could be included in the notation of Section 1.7.53 and defined in general terms.

1.7.204(B)(1)

This section specifies that the effective width of transverse deck floorbeams shall be computed by the rules of Section 1.7.204(A). This requirement seems to conflict with Section 1.7.209(B) which refers the designer to AASHTO Article 1.7.51 for the design of orthotropic decks. An obvious question raised by this discrepancy is whether the effective width of deck plate acting with the longitudinal girder is to be based on 1.7.204 or 1.7.51(C). Some additional comments on these points would be helpful.

Figure 1.7.204(d)

This figure is used to determine Ψ (the effective width coefficient) based on span-to-width ratios and the position within the span. The curves should be continued for span-to-width ratios between 0 and 5. Also, a larger figure with more divisions indicated on the axes would be helpful. Typically, the ordinates and abcissas of the figures in the proposed specification are drawn to scales which do not fit an engineer's scale. Hence, the use of these figures is more cumbersome than necessary.

1.7.205(D), 1.7.206(C), 1.7.208(D)

These sections discuss the design forces to be used in flange design. There are many references to the "governing stresses." It has been interpreted that the governing stresses arise from that case which causes the maximum axial stress in the flange. Therefore, the "governing" shear stresses may or may not be the absolute maximum values. Is it intended that other coinciding loading cases are to be investigated in an effort to find the true governing case? Or, alternately, could the maximum values of axial and shear stresses be treated as one case even though in reality they may arise from different cases?

1.7.206(B)(5)

This article discusses compression flanges subject to a linearly varying stress across their width. This situation could arise, for example, from a lateral load on the box. It is not clear whether this case is to be investigated for a non-uniform stress distribution due to shear lag. Or is such an investigation confined to vertical loads only?

Figure 1.7.206(A)

This figure is used to determine the strength of stiffened plates in compression. For manual computations the presentation is adequate and easy to use. However, if the design of such elements is to be computerized in the future, a mathematical formulation of these strengths will probably be necessary.

1.7.207

This section pertains to the longitudinal stiffeners of compression flanges. It has been interpreted that the stress f_{max} is to include the effect of the additional force from tension field action, ΔF. It is not clear where the stress is to be computed, whether in the flange plate or at the free edge of the stiffener. The cross-section used for computing this stress has been assumed to be the same as that used for design of the flange panel.

The limits for the effective-slenderness coefficient, C_s, are discontinuous at 0.5 F_y. This seems somewhat unfortunate and, if practical, a transition in this area would be more logical. Also, a general mathematical formula to determine C_s for sections other than plates, tees, and angles might be useful.

1.7.209(C)(1)(a)

To avoid confusion the first paragraph of this section could be changed to read:

> "..., and the flange stresses produced by the factored superimposed dead load and the factored service live load acting on the composite girder,..."

1.7.209(C)(2)

It has been interpreted that the AASHTO effective width from Article 1.7.48(C) for concrete composite flanges is applicable. Therefore, on the designs with composite flanges the section properties have been based on a fully effective steel flange (based on 1.7.206(B)(4)) and a partially effective concrete flange based on the AASHTO effective widths. Possibly some clarification is in order regarding this point.

1.7.210(C), 1.7.211(C)

These sections discuss the design forces to be used in web panel design. As discussed above regarding flange design, the reference to "governing...stresses" is somewhat troubling. Does the maximum shear case with its coinciding moment constitute the governing case as implied in the definition of f_{1w} in Section 1.7.211(B)(4)? Should other coinciding cases be investigated? Would the use of maximum moment and shear values be too conservative for design? Some further discussion on this subject seems warranted.

1.7.210(D)

This section deals with slenderness limitations for unstiffened webs. The limitations should be reversed. See 1.7.211(D).

1.7.210(D), 1.7.211(D), 1.7.212(C)

All sections deal with the slenderness limitations for webs. D_c is defined as the clear distance between neutral axis and compression flange. For composite designs the neutral axis used was that for the composite section for live loads as per AASHTO Article 1.7.61(b). The definition of D_c in the proposed specification should be revised to include composite sections.

1.7.211(B)(4), 1.7.212(B)(3)

These sections cover the determination of the critical shear buckling stress, F_{vcr}. One question arises concerning the stresses f_{1w} and f_{2w} (or f'_{1w} and f'_{2w}). Are these stresses to be computed on the basis of fully effective flanges regardless of whether or not a non-uniform longitudinal stress distribution due to shear lag has been considered in the flange design?

1.7.211(E)

This section covers the additional flange forces due to the postbuckling behavior of the web. Some clarification is required as regards composite flanges. Is a ΔF component to be computed for each stage of analysis (non-composite, live load, etc.) or is only one ΔF to be computed for each flange? If only one ΔF is calculated, is it applied to the composite flange or the steel flange only?

1.7.211(E), 1.7.213(C)(1)(b)

The horizontal and vertical components of the tension field force are dealt with in these sections. These forces are transferred to the flanges and transverse stiffeners. The proposed specification has not discussed the manner in which this transfer takes place. Are there special design considerations that should be specified regarding this topic?

1.7.212(B)(5)

This section deals with the tension field strength of longitudinally stiffened webs. It has been interpreted that the value of F_{vcr} used to compute the tension field stress is $F_{vcr\ min}$ used to determine the ultimate shear capacity of the web. It could be implied from this section that a new value of F_{vcr} should be computed "with horizontal stiffeners disregarded." This should be clarified.

1.7.212(C)(1) and (2)

These sections cover slenderness limitations for webs with longitudinal stiffeners. One apparent contradiction could be discussed in the commentary. The depth of subpanel adjacent to the compression flange is to be greater than or equal to $2D_c/5$ for the case of one longitudinal stiffener but less than or equal to $2D_c/5$ for two or more lines of longitudinal stiffeners.

1.7.212(D)(1)

This section pertains to the additional flange forces when the webs are longitudinally stiffened. The last part of the paragraph should probably read:

> "..., the reduced moment of inertia, I_R, shall be obtained by removing those portions of the longitudinal stiffeners including appropriate effective widths of the web plate in compression."

1.7.213(C)(2)(a)

This subsection deals with the required rigidity of transverse stiffeners. To avoid misapplication of this provision, the last paragraph should probably be changed to read:

> "...the greater value of F_{vcr} or $F_{vcr\ min}$."

1.7.213(C)(2)(b)

This subsection contains an alternate method to determine the minimum relative rigidity coefficient for transverse stiffeners. In computing the thickness t_e the elastic buckling capacity under shear acting alone (F^{o}_{vcr}) is required for the longitudinally stiffened web.

Is this the minimum F°_{vcr} of all subpanels or does the subpanel with the minimum critical buckling shear stress (F_{vcr}) govern? The former interpretation has been used for all designs.

1.7.213(C)(3)

This section covers the torsional stability of web stiffeners. Generally, the above comments on Section 1.7.207 apply. It is not clear where on the stiffener the stress f_{max} is to be calculated. The stress on one-sided stiffeners in particular varies considerably across the stiffener section. The free edge of the stiffener is often in tension.

1.7.213(D)(3)

This section specifies rigidity requirements for longitudinal stiffeners. The stress f_c and f_b have been interpreted as being the pure axial and pure bending stresses, respectively, which together make up the true state of axial stress in the web subpanel. The use of the word "actual" in the specification seems a little confusing. Possibly some rewording is necessary.

In the last sentence of this section the word "adjacent" could be added to avoid confusion:

> "All above stresses shall be calculated in the adjacent subpanel with the lower value of F_{vcr}."

Figure 1.7.213(A)

This figure is used to determine the relative rigidity coefficient for transverse web stiffeners. It is unfortunate that the web panel illustrated with this figure shows only one longitudinal stiffener. It has been interpreted that for webs with multiple longitudinal stiffeners the selection of the rigidity coefficient is based on the longitudinal stiffener with the largest η value.

For computerized design, mathematical expressions for the relative rigidity coefficients will be required.

Figure 1.7.213(B)

This figure is used to determine the relative rigidity coefficient for longitudinal web stiffeners. It appears that too much information has been compressed on this one figure as it is somewhat confusing to use. A possible solution would be multiple figures, one for each value of R. Also, curves for additional values of α would be useful. The curves also limit the value of η to no less than 0.2. In reality, values of η as low as 0.1 would be practical for deep webs and the curves should be extended to at least this point.

Mathematical expressions for the rigidity coefficients will be required for computerized design.