Design of a Rolled Beam Bridge by New AASHTO Guide Specification for Compact Braced Sections

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The following example illustrates the design of a three span composite continuous beam highway bridge (83′–104′–83′) by Autostress procedures. The bridge will carry Route 427 over Seeley Creek near Elmira, New York. The bridge was designed using rolled beams without cover plates and the slab was considered non-composite in the negative moment regions. The design used the proposed "Guide Specification for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections" of March 1, 1985. The guide specification contains suggested Autostress design procedures submitted to AASHTO for consideration. ASTM A588 steel has been used in the design. A general elevation and transverse section of the bridge are shown in Figs. 1 and 2.

General Design Criteria

Load: HS 20-44
Structural steel: ASTM A588, with F_y = 50,000 psi
Concrete: f'_c = 3,000 psi, modular ratio n = 9
Slab reinforcing steel:
ASTM A615 Gr. 60 with F_y = 60,000 psi
Loading conditions:

Case 1. Weight of girder and slab supported by the steel girder alone.

Case 2. Superimposed dead load (railings and future wearing surface) supported by the composite section with the increased modular ratio $3n = 3 \times 9 = 27$.

Case 3. Live load plus impact supported by the composite section with the modular ratio n = 9.

Stress cycles for fatigue: 500,000 cycles of truck loading
100,000 cycles of lane loading

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Design Loads

In Autostress design, the same load levels are used as in the Load Factor method. The Load Factor method proportions members for multiples of the design loads. Members are required to meet certain criteria for three theoretical levels: (1) Maximum Design Load, (2) Overload and (3) Service Load. Maximum Design Load and Overload are based on multiples of the service loads. Service loads are defined as the same loads as used in working stress design.

The moments, shears or forces to be carried by the steel members were computed from the following formulas for the three loading levels.

Service load: $D + (L + I)$
Overload: $D + 2.08\ (L + I)$
Maximum design load: $1.30\ [D + 2.08\ (L + I)]$

where D = dead load
L = live load
I = impact load

New York State design specifications modify AASHTO by increasing the multiplier for live load and impact from 5/3 to 2.08 for the overload and maximum design load conditions.

Loads, Shears and Moments

The analysis is based on the assumption of a constant moment of inertia throughout the length of the girder. Five W30 × 191 rolled beams spaced at 8′-6″ will be assumed. A 7 in. design slab thickness with a 1-½ in. wearing surface is used.

Dead load carried by steel:

Slab:	8.5/12 × 8.5′ × .15 kcf =	.903 k/ft
Slab haunch:	(2 × 15)/144 × .15 kcf =	.031 k/ft
Stay-in-place forms (foam filled):	8.5 × .004 ksf =	.034 k/ft
W30 × 191 Beam:		.191 k/ft
Diaphragms and laterals:	Assumed:	.040 k/ft
	Dead load carried by steel, per girder =	1.199 k/ft

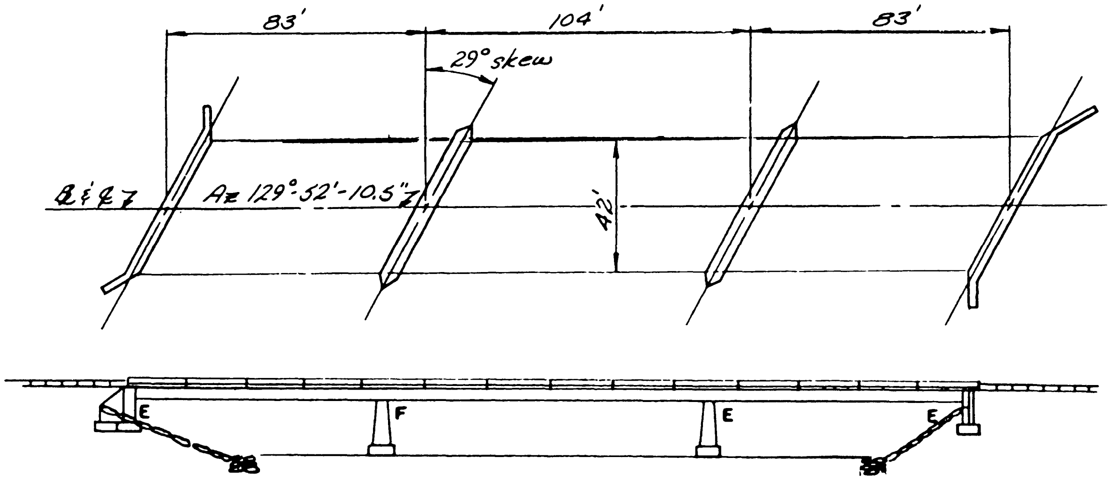


Fig. 1. General elevation

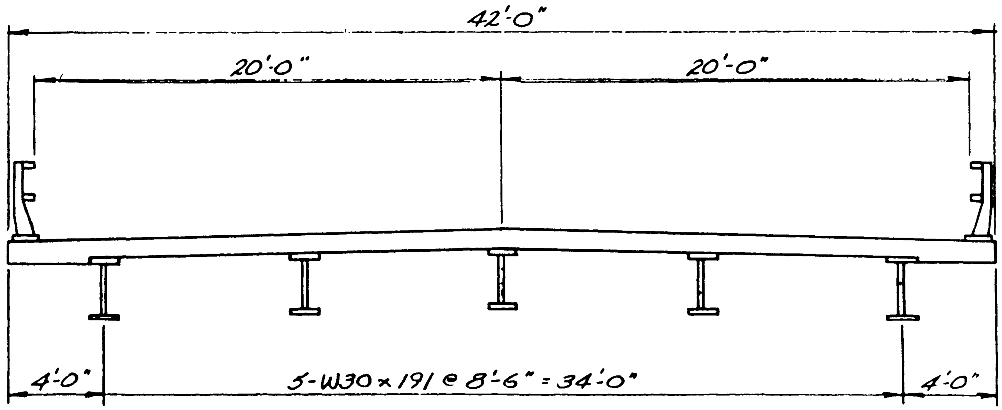


Fig. 2. Transverse section

Dead load carried by composite section:

Railings:	.03 k/ft × 2/5 beams	=	.012 k/ft
Future wearing surface:	.02 ksf × 40′/5 beams	=	.160 k/ft
Composite dead load carried, per girder		=	.172 k/ft

Live load:

Live load distribution = $S/5.5 = 8.5/5.5 = 1.55$ wheels $= 0.773$ of a lane load

Maximum moment and shear curves for service loads are shown in Figs. 3 and 4.

Check Compact Section Requirements

W30 × 191 section—Actual width to thickness ratio of the projecting compression flange element. (bottom flange in the negative moment regions) is:

$$\frac{b'}{t} = \frac{(15.04 - .71)/2}{1.185} = 6.05$$

where b' is the width of the projecting flange element, t is the flange thickness.

Allowable width to thickness ratio is:

$$\frac{b'}{t} = \frac{2,055}{\sqrt{50,000}} = 9.2$$

$9.2 > 6.05$ **o.k.**

$$\frac{6.05}{9.2} = 66\%$$

Actual web thickness ratio:

$$\frac{D}{t_w} = \frac{30.68 - 2(1.185)}{0.71} = 39.9$$

where D is the clear distance between the flanges, t_w is the web thickness.
The section is non-composite in negative moment regions so no adjustment to D has to be made.

Allowable web thickness ratio is:

$$\frac{D}{t_w} = \frac{19,230}{\sqrt{50,000}} = 86.0$$

$86.0 > 39.9$ **o.k.**

$$\frac{39.9}{86} = 46\%$$

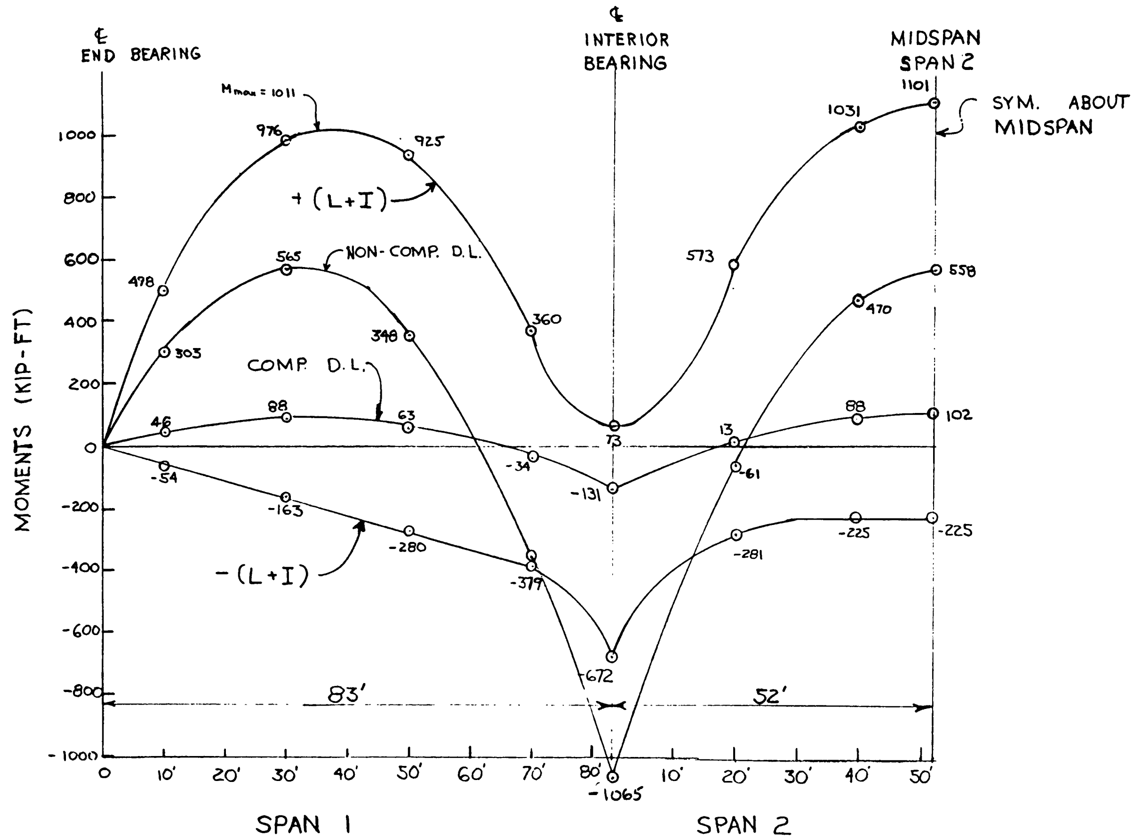


Fig. 3. Maximum moment curves

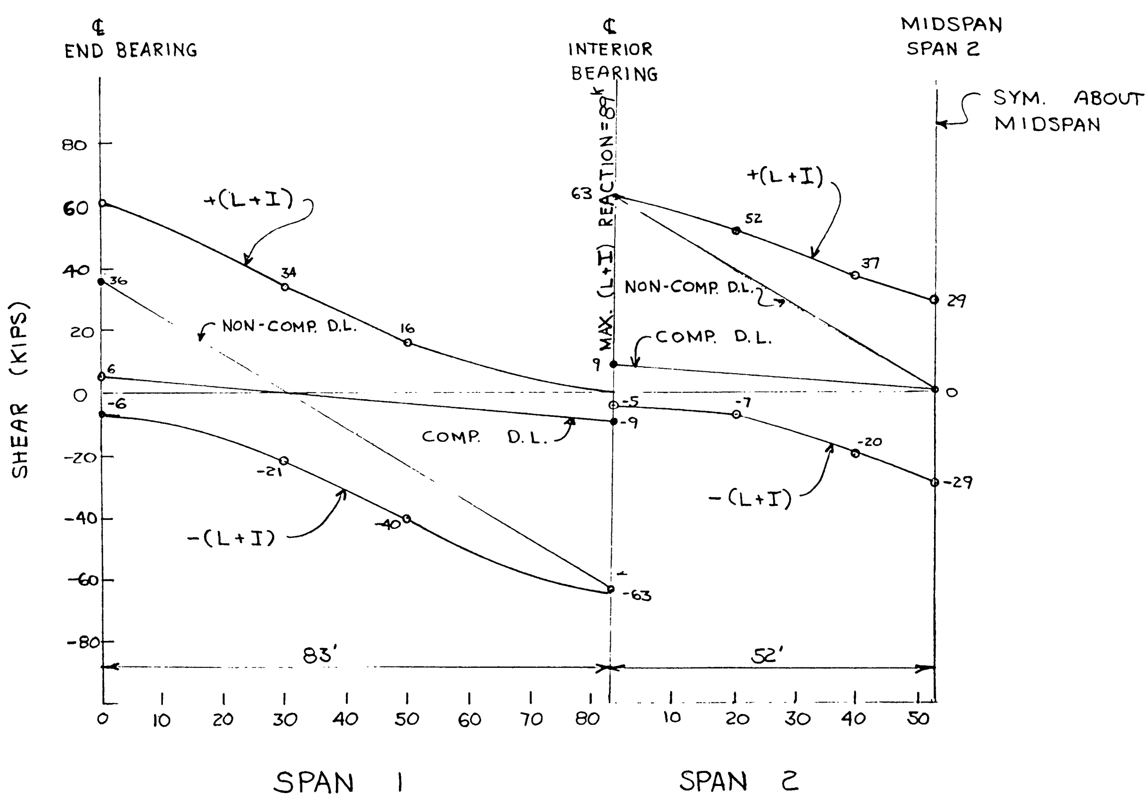


Fig. 4. Maximum shear curves

Since the compact section ratios do not exceed 75% of the allowable limits, there is no need to check the interaction equation.

Diaphragms or cross frames must be placed at the supports and at intermediate points not to exceed 25 ft (AASHTO 10.20.1). In positive moment areas the compression flange is braced by the concrete slab. In the negative moment regions the required bracing distance will be determined in calculations which follow later:

Maximum shear allowed for a compact section is:

$$V = 0.55\, F_y d t_w = 0.55(50)(30.68)(0.71) = 599\text{k}$$

Actual maximum shear at maximum design load is:

$$V = 1.3(63 + 9 + 2.08(63)) = 264\text{k}$$

$$264\text{k} < 599\text{k} \quad \textbf{o.k.}$$

The W30 × 191 section qualifies as a braced compact composite section.

Compute Section Properties

Negative Moment Section

Non-composite W30 × 191

$A = 56.1 \text{ in.}^2$

$I_x = 9{,}170 \text{ in.}^4$

$S_x = 598 \text{ in.}^3$

$Z = 673 \text{ in.}^3$

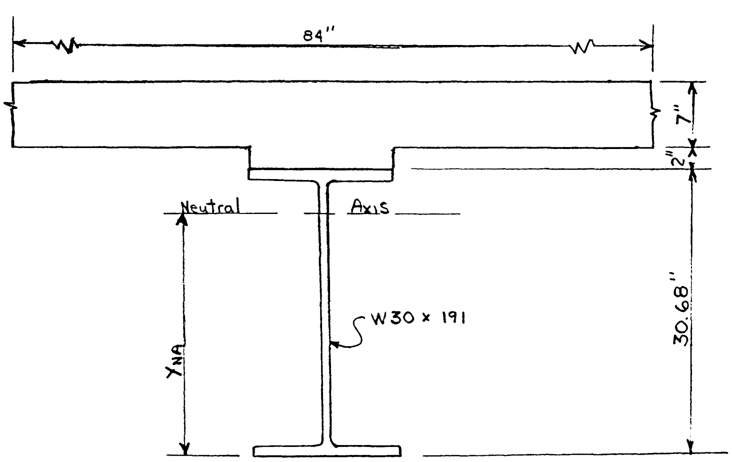
Compute yield moment

$$M_y = \frac{F_y S_x}{12} = \frac{50(598)}{12} = 2{,}492 \text{ k/ft}$$

Compute plastic moment

$$M_p = \frac{F_y Z}{12} = \frac{50(673)}{12} = 2{,}804 \text{ k/ft}$$

Positive Moment Section



Elastic Properties

Composite section, $n = 9$
Transformed slab area $= (84 \times 7)/9 = 65.33 \text{ in.}^2$
Transformed haunch area $= (15.04 \times 2)/9$
$= 3.34 \text{ in.}^2$

Material	A	y	Ay	d	Ad^2	I_o	I
W30 × 191	56.1	15.34	860.57	11.35	7227	9170	16397
Slab	65.33	36.18	2363.64	9.49	5884	267	6151
Haunch	3.34	31.68	105.81	4.99	83	1	84
	124.77		3,330.02		I_{NA}	=	22,632 in.⁴

$$y_{NA} = \frac{3{,}330.02}{124.77} = 26.69 \text{ in.}$$

$d_{\text{top of slab}} = 12.99$ in. $\quad d_{\text{top of steel}} = 3.99$ in.

$$S_{\text{top of slab}} = \frac{22{,}632}{12.99} = 1{,}742.3 \text{ in.}^3$$

$$S_{\text{top of steel}} = \frac{22{,}632}{3.99} = 5{,}672.2 \text{ in.}^3$$

$$S_{\text{bottom of steel}} = \frac{22{,}632}{26.69} = 848.0 \text{ in.}^3$$

Compute yield moment, M_y:

$$M_y = \frac{F_y\, S_{\text{bot. of steel}}}{12} = \frac{50(848)}{12} = 3{,}533.3 \text{ k/ft}$$

Composite section, $n = 27$
Transformed slab area $= (84 \times 7)/27 = 21.78 \text{ in.}^2$
Transformed haunch area $= (15.04 \times 2)/27$
$= 1.11 \text{ in.}^2$

Material	A	y	Ay	d	Ad^2	I_o	I
W30 × 191	56.1	15.34	860.57	5.98	2006	9170	11176
Slab	21.78	36.18	788.00	14.86	4809	89	4898
Haunch	1.11	31.68	35.16	10.36	119	—	119
	78.99		1,683.73				16,193 in.⁴

$$y_{NA} = \frac{1{,}683.73}{78.99} = 21.32 \text{ in.}$$

$d_{\text{top of slab}} = 18.36$ in. $\quad d_{\text{top of steel}} = 9.36$ in.

$$S_{\text{top of slab}} = \frac{16{,}193}{18.36} = 882.0 \text{ in.}^3$$

$$S_{\text{top of steel}} = \frac{16{,}193}{9.36} = 1{,}730.0 \text{ in.}^3$$

$$S_{\text{bottom of steel}} = \frac{16{,}193}{21.32} = 759.5 \text{ in.}^3$$

Compute yield moment

$$M_y = \frac{F_y S_{\text{bot. of steel}}}{12} = \frac{50(759.5)}{12} = 3{,}164.6 \text{ k/ft}$$

Plastic Properties

The section has been found to be a braced compact, composite section in the positive moment area. The maximum strength of the section is determined by the fully plastic moment capacity of the section. This is found by determining the compressive force in the slab, locating the neutral axis and calculating the total moment about the neutral axis of all forces acting on the section. If adequate shear connectors are provided the compressive force in the slab is the smaller value of either the fully plastic force developed by the steel section or the compressive strength of the slab.

Capacity of the concrete slab: $f'_c = 3{,}000$ psi, longitudinal reinforcing steel − 5 # 5 bars, $A_s = 1.55 \text{ in.}^2$, $F_y = 60{,}000$ psi.

$C_1 = .85 f'_c b t_s + A_s F_y + .85 f'_c$ (haunch area)

$C_1 = .85(3)(84)(7) + 1.55(60) + .85(3)(15.04)(2)$

$C_1 = 1{,}669$ kips (controls)

Plastic force developed by the steel section:

$C_2 = AF_y$

$C_2 = 56.1(50)$

$C_2 = 2{,}805$ kips

The neutral axis of the plastic section is in the steel section and probably in the flange. Let y be the distance from the top of steel to the neutral axis.

Balance forces:

$1{,}669 + y\,(15.04)(50) = 2{,}805 - y\,(15.04)(50)$

$y = .76$ in. from top of steel section

Compute the plastic moment capacity:

Bott. flange 15.04(1.185)(29.33)(50)	= 26,140
Web 28.31(.71)(14.58)(50)	= 14,650
Top flange (tens.) 15.04(1.185 − .76)(.21)(50)	= 67
Top flange (comp.) 15.04(.76)(.38)(50)	= 217
Haunch 77(1.76)	= 136
Slab 1,499(3.5 + 2 + .76)	= 9,384
Slab reinforcement 93(6.26)	= 582
	51,176 k/in.
Plastic moment capacity, positive moment, M_p	= 4,265 k/ft

Calculation of Automoments and Inelastic Rotations

Elastic service load moments:

	Max. Neg. Moment (k/ft)	Max. Pos. Moment (k/ft)
Non-Comp. DL	1,065	558
Comp. DL	131	102
$L + I$	672	1,101
	1,868 k/ft	1,761 k/ft

Because of unshored construction find automoment and plastic deformation of the steel beam at the piers due to the non-composite dead load.

Ratio M (non-composite dead load)/plastic moment capacity (negative moment)

$$\frac{M}{M_{PN}} = \frac{1{,}065}{2{,}804} = 0.38$$

The inelastic rotation curve (Fig. 5) shows no plastic deformation for $M/M_{PN} < 0.6$ for non-composite sections. Non-composite dead load produces no plastic deformation in this section.

The overload negative moment at the piers is

$$D + 2.08\,(L + I) = 1{,}065 + 131 + 2.08(672) = 2{,}594 \text{ k/ft}$$

Check steel stress at overload:

$$f_b = \frac{2{,}594(12)}{598} = 52.1 \text{ ksi}$$

Steel will yield at overload.

Determine location of Point A on Fig. 5. Point A is determined by ratio, M/M_{PN}.

$$\frac{M}{M_{PN}} = \frac{2{,}594}{2{,}804} = .925$$

Point B on Fig. 5 is determined by the total rotation at the pier due to the elastic overload moment assuming there are free hinges at the piers and the rest of the structure is elastic.

Computer equivalent uniform load to produce a negative moment of 2,594 k/ft at the piers.

From AISC *Moments, Shears and Reactions for Continuous Highway Bridges*, interpolating between Tables 3.2 and 3.3 for a span ratio of 1:1.25:1, a uniform load will produce a moment $= -0.1291\,wL^2$ at the piers. Setting this equal to the overload moment:

$$-0.1291 wL^2 = -2{,}594$$

$$w = \frac{2{,}594}{0.1291(83)^2} = 2.917 \text{ k/ft} = .243 \text{ k/in.}$$

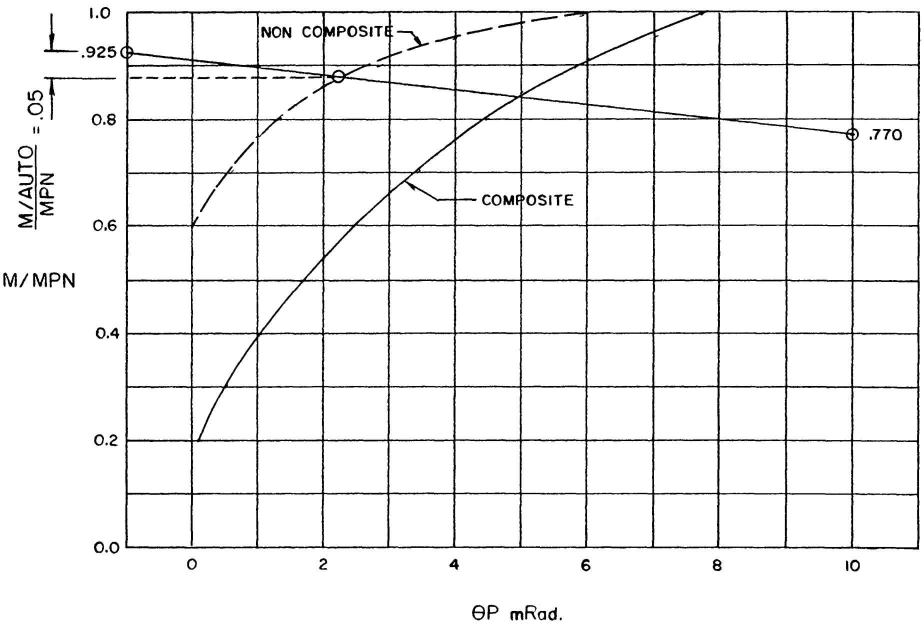
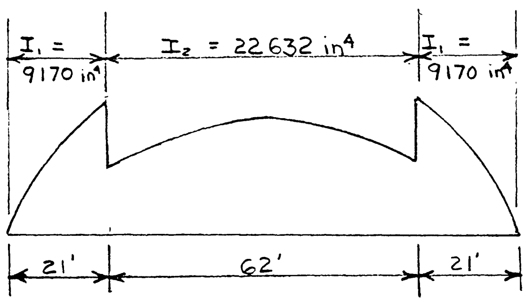


Fig. 5. Inelastic rotation

The change in slope between two points on the elastic curve is equal to the area of the M/EI diagram between these points. The M/EI diagram for the 104′ span considering the non-composite portions of the span is shown below:



w = .243 k/in
l = 1248"
I_1 = 9170 in^4
I_2 = 22,632 in^4

The equation for moment at any point, x distance from the left support, on a simple span uniformly loaded is $wx/2\ (\ell - x)$

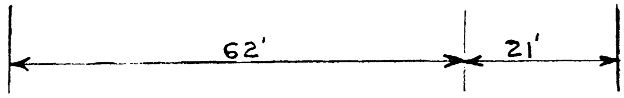
The end rotation for the 104-ft span is equal to half the area under the M/EI diagram. This is calculated by integrating the equation for moment to determine the area under the curve.

$$\theta = \int_0^{252} \frac{wx}{2}(\ell - x)/EI_{1dx} + \int_0^{624} \frac{wx}{2}(\ell - x)/EI_{2dx}$$

$$\theta = \frac{wlx^2}{4EI_1}\bigg|_0^{252} - \frac{wx^3}{6EI_1}\bigg|_0^{252} + \frac{wlx^2}{4EI_2}\bigg|_{252}^{624} - \frac{wx^3}{6EI_2}\bigg|_{252}^{624}$$

$\theta = 39.1 \times 10^{-3}$ radians for 104-ft span

The rotation for the 83-ft span must now be determined and added to that calculated for the 104-ft span. Again a free hinge is assumed at the pier. The M/EI diagram for the 83-ft span is shown below.



The total area under the M/EI diagram was integrated and found to be .0373.

The area under curve A was integrated and found to be .0305.

By conjugate beam theory the rotation at the interior support will equal the "shear" in the M/EI diagram at that point. Find the reaction, R, of the conjugate beam.

Shear due to curve A = .0305/2 = .0151

Area between curve A and the total M/EI diagram

$$= 0.0373 - 0.0305 = 0.0068$$

The centroid of this area is located at approximately $2/3 \times 252 = 168$ in. from the interior support.

$$\text{Shear due to this area} = \frac{996 - 168}{966}(0.0068) = .0056$$

Rotation for 83-ft span = 0.0151 + 0.0056 = 0.0207
= 20.7×10^{-3} radians

The total rotation at the pier due to the elastic overload moment assuming there are free hinges at the piers is the sum of the rotation for the 83-ft. and 104-ft spans.

Total rotation = 39.1 + 20.7 = 59.8 mrads

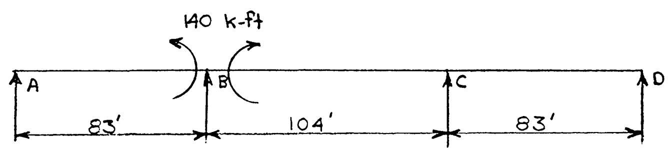
This rotation determines the location of point B on Fig. 5. A rotation of 59.8 mrads is off the chart so the slope of the line from point A to point B is used to determine the value of M/M_{PN} at a rotation of 10 mrads.

$$x = \frac{59.8 - 10}{59.8}(.925) = .77$$

The A-B line is plotted on Fig. 5 and its intersection with the non-composite curve is located.

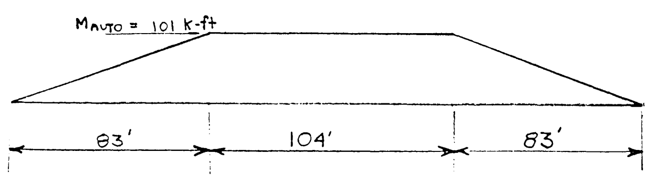
From this intersection the value of $M_{\text{AUTO}/M_{PN}}$ is determined from the chart to be 0.05.

$$M_{\text{AUTO}} = M_{PN} \times .05$$
$$= 2{,}804 \times .05$$
$$= 140 \text{ k/ft}$$



This is the value of the automoment due to plastic rotation at a single pier. However the effect of plastic rotation at both piers must be evaluated. This will be done by applying the automoments to one pier at a time and then using the principle of superposition to obtain the results.

The three-moment equation will be used to determine the moment at C due to the 140 k/ft automoment applied at B. The beam to the right of B is shown below.



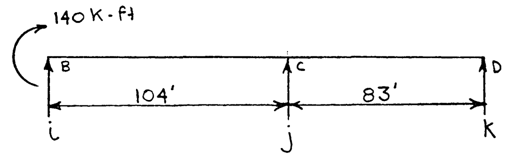
The three-moment equation is:

$$M_i L_i + 2M_j(L_i + L_j) + M_k L_j = \frac{W_i L_i^3}{4} - \frac{W_j L_j^3}{4}$$

$$140(104) + 2M_j(104 + 83) + 0(83) = 0 - 0$$
$$14{,}560 + 374M_j = 0$$
$$M_j = -38.9 \text{ k/ft.}$$

By superposition, M_{AUTO} due to plastic rotations at both piers = 140 − 39 = 101 k/ft.

The moment diagram due to the automoments is shown below:



The automoments result in a decrease in the maximum negative moment at the piers and an increase in the maximum positive moment.

Check Stresses at Overload

The stresses at mid-span of the center span will be computed for the overload condition.

Stresses due to dead-load moment on non-composite section:

$$f_s\,(\text{top}) = -\frac{558(12)}{598} = -11.2 \text{ ksi}$$

$$f_s\,(\text{bottom}) = \frac{558(12)}{598} = 11.2 \text{ ksi}$$

$$f_c = 0$$

Stresses due to dead-load moment on composite section, n = 27:

$$f_s\,(\text{top}) = \frac{-102(12)(9.36)}{16{,}193} = -0.7 \text{ ksi}$$

$$f_s\,(\text{bottom}) = \frac{102(12)(21.32)}{16{,}193} = 1.6 \text{ ksi}$$

$$f_c = \frac{102(12)(18.36)}{16{,}193(27)} = -0.05 \text{ ksi}$$

Stresses due to overload live load on composite section, n = 9:

Overload live load + imp. moment = 2.08 × 1,101 = 2,290 k/ft

$$f_s\,(\text{top}) = \frac{-2{,}290(12)(3.98)}{22{,}632} = -4.8 \text{ ksi}$$

$$f_s\,(\text{bottom}) = \frac{2{,}290(12)(26.70)}{22{,}632} = 32.4 \text{ ksi}$$

$$f_c = \frac{-2{,}290(12)(12.98)}{22{,}632(9)} = -1.75 \text{ ksi}$$

Stresses due to automoments on composite section, n = 27:

$$f_s\,(\text{top}) = \frac{-101(12)(9.36)}{16{,}193} = -0.7 \text{ ksi}$$

$$f_s\,(\text{bottom}) = \frac{101(12)(21.32)}{16{,}193} = 1.6 \text{ ksi}$$

$$f_c = \frac{-101(12)(18.36)}{16{,}193(27)} = -0.05 \text{ ksi}$$

Summary of stresses:

	Bott. Steel	Top Steel	Top Concrete
Non-Comp. *DL*	11.2	−11.2	0
Comp *DL*	1.6	− 0.7	− .05
Overload *LL* + *I*	32.4	− 4.8	−1.75
Automoment	1.6	− 0.7	− .05
Total	46.8 ksi	−17.4 ksi	−1.85 ksi

Allowable steel stress at overload = $0.95\ F_y$ = 47.5 ksi

46.8 ksi < 47.5 ksi **o.k.**

W30 × 191 section satisfies overload criteria.

Check maximum design load

Maximum design load (positive moment) at midspan of center span:

$$= 1.3[(558 + 102) + 2.08(1101)] = 3{,}835 \text{ k/ft}$$

Maximum design load (negative moment) at interior supports:

$$= 1.3[(1{,}065 + 131) + 2.08(672)] = 3{,}372 \text{ k/ft}$$

The plastic moment capacity for negative moment at the interior supports was previously computed to be 2,804 k/ft.

The difference between the maximum design load negative moment and the plastic moment capacity is:

$$3{,}372 - 2{,}804 = 568 \text{ k/ft}$$

This excess will be redistributed to the positive moment areas in the same proportion as the automoments previously computed. The controlling location is at the midpoint of the center span. The maximum design load at the midpoint of the center span after the redistribution of the negative moments is:

$$3{,}835 + \frac{101}{140}(568) = 4{,}245 \text{ k/ft}$$

The plastic moment capacity of the section for positive moment was computed to be 4,265 k/ft.

4,245 k/ft < 4,265 k/ft **o.k.**

A failure mechanism will not form, so a W30 × 191 section is adequate.

Lateral Bracing Requirements

The lateral bracing requirements will be designed in accordance with AASHTO 10.48.1.1. The compact section requirements have already been checked.

AASHTO requirements that apply in negative moment regions are:

$$\frac{l_b}{r_y} = \frac{12{,}000}{\sqrt{F_y}} \text{ when } M_2 < 0.7M_1 \text{ (steep moment gradient)}$$

$$Max\ L_b = \frac{12{,}000}{\sqrt{50{,}000}}(3.46) = 186'' = \underline{15.5'}$$

OR

$$\frac{l_b}{r_y} = \frac{12{,}000}{\sqrt{F_y}} \text{ when } M_2 \geq 0.7M_1$$

$$\text{Max } L_b = \frac{7{,}000}{\sqrt{50{,}000}}(3.46) = 108'' = \underline{9.0'}$$

At the plastic hinge at the interior supports the current AISC requirements for lateral bracing will be used:

$$\frac{l_{cr}}{r_y} = \frac{1{,}375}{F_y} \text{ when } -0.5 \geq \frac{M}{M_p} > -1.0$$

Where M = lesser of the moments at the ends of the unbraced segment.

M/M_P = end moment ratio, positive when the segment is bent in reverse curvature and negative when bent in single curvature.

$$l_{cr} = \frac{1{,}375(3.46)}{50} = 95'' = 7.9'$$

Diaphragms must also be placed at the supports and intermediate diaphragms must be provided at intervals not to exceed 25 ft. In addition diagonal lateral bracing for the bottom flange was placed near the interior supports. The framing plan is shown in Fig. 6.

Check Fatigue Stresses

Base metal adjacent to stud shear connectors and the base metal in the beam web at the toe of bearing stiffeners or diaphragm connection plates are AASHTO Category C details. The average daily truck traffic (ADTT) for this structure is less than 2,500. For this ADTT the structure must be checked for 500,000 cycles of truck loading and 100,000 cycles of lane loading.

From AASHTO Table 10.3.1A allowable fatigue stress ranges for Category C details are 32 ksi for 100,000 cycles and 19 ksi for 500,000 cycles.

Check section at midpoint of center span:

$LL + I$ max. pos. service load moment = 1,101 k/ft (truck)
$LL + I$ max. neg. service load moment = $\underline{-225}$ k/ft (truck)
Moment range = 1,326 k/ft

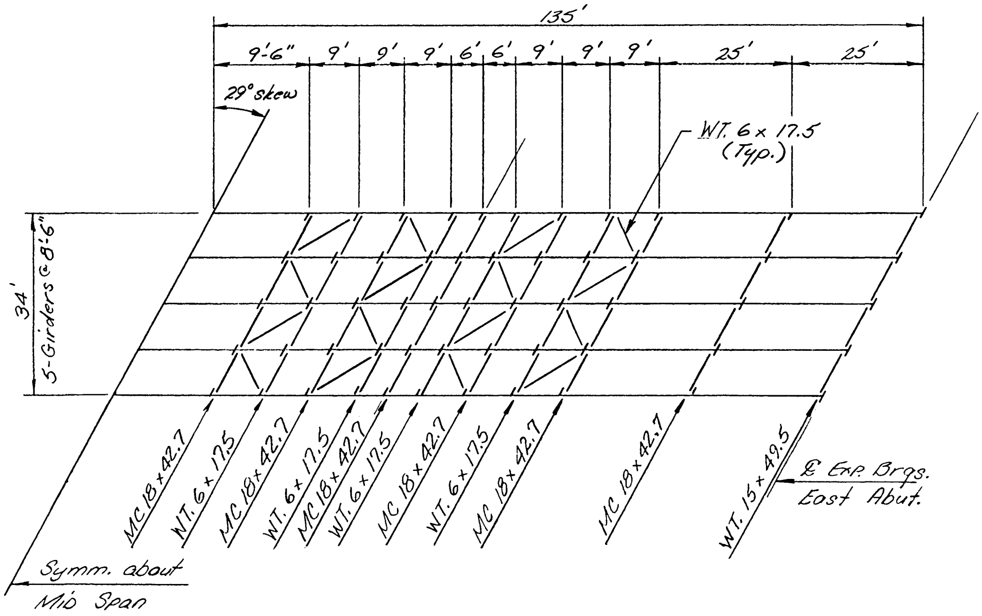


Fig. 6. Framing plan

Stress range at top of bottom flange

$$\frac{1{,}326(12)(26.70 - 1.185)}{22{,}632} = 18.0 \text{ ksi}$$

$$18.0 \text{ ksi} < 19.0 \text{ ksi} \quad \textbf{o.k.}$$

Check section at point of dead load contraflexure in center span (stud shear connectors welded to top flange):

$LL + I$ max. pos. service load moment = 625 k/ft (truck)
$LL + I$ max. neg. service load moment = -272 k/ft (truck)
Moment range = 899 k/ft

Stress range at top of top flange (non-composite section)

$$\frac{899(12)}{598} = 18.0 \text{ ksi}$$

$$18.0 \text{ ksi} < 19.0 \text{ ksi} \quad \textbf{o.k.}$$

Check section at interior support:

$LL + I$ max. pos. service load moment = 73 k/ft (truck)
$LL + I$ max. neg. service load moment = -672 k/ft (lane)
Moment range = 745 k/ft

Stress range at top of web (non-composite section)

$$= \frac{745(12)(15.34 - 1.185)}{9{,}170}$$

$$= 13.8 \text{ ksi}$$

Allowable stress range = 32.0 ksi (lane loading)

$$13.8 \text{ ksi} < 32.0 \text{ ksi} \quad \textbf{o.k.}$$

Section is **o.k.** for fatigue.

Check deflections

Live load deflections were computed using a continuous girder analysis computer program. AASHTO 10.6.2. limits live load plus impact deflection to preferably less than $\ell/800$ of the span. $\ell/800$ of the center span is 1.56 in. Live load plus impact deflections were computed considering all five beams to equally share the live-load deflections and to be non-composite in negative moment regions where no shear connectors are provided.

$LL + I$ deflection for 3 traffic lanes (design condition) = 1.79 in.
$LL + I$ deflection for 2 traffic lanes = 1.33 in.

If the girders are considered composite for deflection in the negative moment regions the following deflections are computed:

$LL + I$ deflection for 3 traffic lanes = 1.36 in.
$LL + I$ deflection for 2 traffic lanes = 1.01 in.

The 1.79-in. $LL + I$ deflection for 3 traffic lanes, considering the beam to be non-composite in negative moment regions, is 15% over the $L/800$ value of 1.56 in. This is considered allowable because the beams will pick up some composite action in the negative moment regions even though there are no shear connectors and it is unlikely that 3 traffic lanes will be loaded to the maximum simultaneously since the structure carries a two lane highway.

Check for Uplift at abutments

The possibility of uplift at the abutments when the center span is loaded must be investigated. An influence line for

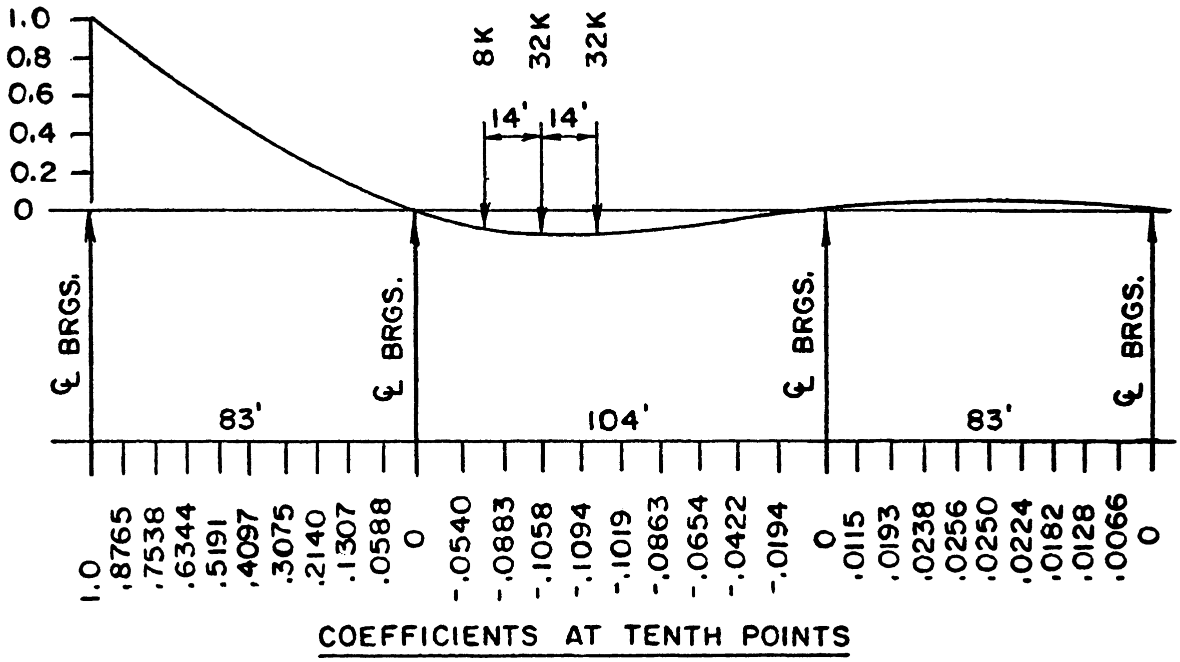


Fig. 7. Influence line for abutment reaction

the abutment reaction is shown in Fig. 7. Possible uplift at the abutments is checked rather than possible uplift at the interior supports because of the much higher dead load reactions at the interior supports.

First trial: Place end axle of truck at $0.4L$ of center span.

Lane reaction
$= 32(-0.1094) + 32(-0.0995) + 8(-0.0637)$
$= -7.19\text{k}$

Second trial: Place center axle of truck at $.4L$ of center span.

Lane reaction
$= 32(-0.0963) + 32(-0.1094) + 8(-0.0995)$
$= -7.38\text{k}$ **controls**

$$\text{Impact} = \frac{50}{L + 125} = \frac{50}{104 + 125} = 0.218$$

$$\text{Distribution factor} = \frac{S}{5.5} = \frac{8.5}{5.5} = 1.55$$

$LL + I$ uplift reaction $= 7.38(1.218)(1/2)(1.55) = 6.97\text{k}$

Dead load reaction at abutment—from Fig. 4 $= 36\text{k}$

$36\text{k} > 6.97\text{k}$ **o.k.—No uplift.**

Check flange stability prior to concrete deck placement

AASHTO 10.50c requires that the ratio of the projecting top compression flange width to thickness shall not exceed

$$\frac{b'}{t} = \frac{2{,}200}{\sqrt{1.3 f_{dl}}}$$

where f_{dl} is the top flange compressive stress due to non-composite dead load. At the section of maximum positive moment.

$$f_{dl} = \frac{558(12)}{598} = 11.2 \text{ ksi}$$

$$\frac{2{,}200}{\sqrt{1.3(11{,}200)}} = 18.2$$

$$\frac{b'}{t} \text{ of W30} \times 191 \text{ section} = 6.05$$

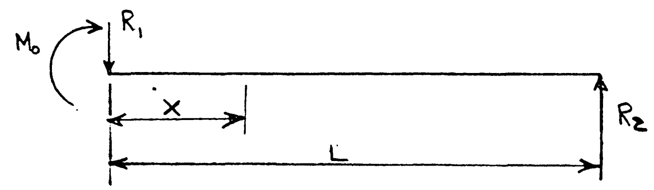
$6.05 < 18.2$ **o.k.**

Camber Due to Automoment

The deflections produced due to the automoments at the interior supports must be calculated so that this camber can be fabricated into the beams. The automoments are considered long term loading that act on the composite, $n = 27$, section. The deflections will be computed here for the span midpoints.

End spans:

From *Design of Welded Structures* by Blodgett #8.1-10



$$\Delta_x = \frac{M_0}{6EI}\left(3x^2 - \frac{X^3}{L} - 2Lx\right)$$

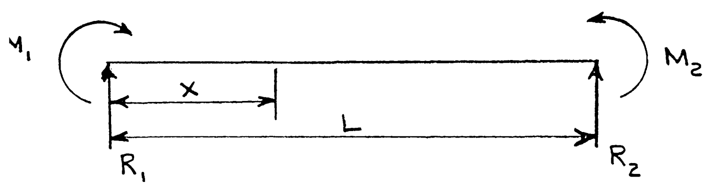
At $x = .5L = 498''$

$$\Delta_x = \frac{101(12)}{(6)(29{,}000)(16{,}193)}\left(3(498)^2 - \frac{498^3}{996} - 2(996)(498)\right)$$

$\Delta_x = 0.160$ in. $= 0.01$ ft camber to be provided for auto-moments

Center span:

From *Design of Welded Structures* by Blodgett #8.1-11



$$\Delta_x = \frac{x(L-x)}{6EIL}[M_1(2L-x) + M_2(L+x)]$$

$$\Delta_x = \frac{624(1{,}248 - 624)}{6(29{,}000)(16{,}193)(1{,}248)}[101(12)(2(1{,}248) - 624) + 101(12)(1{,}248 + 624)]$$

$\Delta_x = 0.502$ in. $= 0.04$ ft camber to be provided for auto-moments

Check for wind loads

AASHTO 3.15 and 10.21.2 specify a horizontal wind force of 50 lbs. per square foot on the exposed area of the superstructure. Half of this force is to be applied to each flange. The total force shall not be less than 300 lbs. per foot. The following stress combinations will be investigated.

$$1.3\,(F_{DL} + F_w)$$

$$1.3\,(F_{DL} + 1.25\,F_{LL+I} = .3F_w)$$

where F_{DL} is the total service load dead load stress in the bottom flange

F_{L+I} is the service load stress due to live load plus impact in the bottom flange.

F_w is the wind load stress in the bottom flange.

The wind acting on the live load is neglected because it is assumed that it will be taken by the deck slab.

Check wind load stresses at maximum positive moment section

The section to be checked is at mid-span of the center span.

Exposed superstructure area:

Beam 30/12	$= 2.5'$
Deck & haunch $\frac{8.5+2}{12}$	$= .875$
Railing (2-rail) $(2 \times .25)$	$= .5$
	$3.875'$

Wind load $= 3.875' \times 50$ psf $= 194$ lbs./ft.

Use 300 lbs./ft minimum. The wind load carried by the bottom flange is 150 lbs. per foot. Wind load stresses are computed in accordance with AASHTO 10.20.21.

$M_{cb} = .08WSd^2$

where W = Wind Load (lb./ft)

Sd = diaphragm spacing (ft)

$M_{cb} = 0.08(150)(25)^2 = 7{,}500$ ft/lb.

$$F_{cb} = \frac{72\,M_{cb}}{t_f b_f^2} = \frac{72(7{,}500)}{1.185(15.04)^2}$$

$F_{cb} = 2{,}015$ psi

$R = [0.2272L - 11]\,S_d^{-2/3}$ when no bottom lateral bracing is provided.

$R = [0.2272(104) - 11]\,25^{-2/3}$

$R = 1.48$

$F_w = RF_{cb}$

$F_w = 1.48\,(2{,}015) = 2{,}960$ psi

$$F_{DL} = \frac{558(12)}{598} + \frac{102(12)(21.32)}{16{,}193} + \frac{101(12)(21.32)}{16{,}193}$$

$F_{DL} = 11.2 + 1.6 + 1.6 = 14.4$ ksi

$$F_{LL+I} = \frac{1{,}101(12)(26.70)}{22{,}632} = 15.6 \text{ ksi}$$

Checking the combined stresses:

$1.3\,(F_{DL} + F_w) = 1.3\,(14.4 + 2.96) = 22.6$ ksi

22.6 ksi < 50 ksi **o.k.**

$$1.3\,(F_{DL} + 1.25F_{LL+I} + 0.3F_w)$$

$$1.3\,(14.4 + 1.25(15.6) + 0.3\,(2.96) = 45.2 \text{ ksi}$$

$$45.2 \text{ ksi} < 50 \text{ ksi} \quad \textbf{o.k.}$$

Check wind load stresses at the pier

Use wind load of 150 lbs. per foot on botton flange. Bottom flange is in compression. Diaphragm spacing near pier is 6 feet.

$$M_{cb} = 0.08(150)(6)^2 = 432 \text{ ft/lb.}$$

$$F_{cb} = \frac{72(432)}{1.185(15.04)^2} = 116 \text{ psi}$$

$R = [0.059L - 0.64]\,S_d^{-1/2}$ when bottom lateral bracing is provided.

$$R = [0.059(104) - 0.64](6)^{-1/2}$$

$$R = 2.24$$

$$F_w = RF_{cb} = 2.24(116) = 260 \text{ psi}$$

This is a negligible wind load stress so it is not necessary to check further.

Design of Shear Connectors

The range of horizontal shear is given by AASHTO 10.38.5.1.1

$$S_r = \frac{V_r Q}{I}$$

where S_r = range of horizontal shear, in kips per in. at the junction of the slab and girder.

V_r = range of shear due to live load and impact in kips.

Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area.

I = moment of inertia of the transformed composite girder.

From Fig. 4 the live load plus impact shear ranges at the following locations are obtained:

Centerline bearings at abutments	= 66k	**controls**
Dead load contraflexure—83′ spans	= 56k	
Dead load contraflexure—104′ span	= 58k	

$$S_r = \frac{66(636)}{22{,}632} = 1.85 \text{ k/in.}$$

¾ in. dia., 4 in. long welded studs will be used.

The allowable range of shear for 500,000 cycles of loading = $10{,}600(.75)^2$ = 5,960 pounds per connector.

Assuming a double row of connectors the required pitch is:

$$\frac{2(5.96)}{1.85} = 6.44 \text{ in.}$$

Check shear connectors for ultimate strength:

Ultimate strength of one connector,

$$S_u = 0.4d^2\sqrt{f'_c E_c}$$

$$= 0.4(0.75)^2\sqrt{3{,}000(3{,}320{,}000)}$$

$$= 22{,}500 \text{ lbs.}$$

Number of connectors required between point of maximum moment and end support,

$$N_1 = \frac{P}{\phi S_u}$$

where P is smaller of

$$P_1 = A_s F_y = 56.1(50) = 2{,}805\text{k}$$

$$P_2 = .85f'_c bc = .85(3)(84)(7) = 1{,}499\text{k} \quad \textbf{controls}$$

$$N_1 = \frac{1{,}499}{.85(225)} = 78.4 \text{ connectors in 28 ft required.}$$

$$\text{Required pitch} = \frac{28(12)}{78.4(2)} = 8.5 \text{ in. for double row}$$

Fatigue controls for shear connectors. Use double row at 6.44 in. maximum pitch.

Additional connectors required at points of dead load contraflexure: (AASHTO 10.38.5.1.3)

$$N_c = A_r^s f_r / Z_r$$

where A_r^s = total area of longitudinal slab reinforcing steel for each beam over interior support. #6 bars at 6″ spacing over 84 in. effective slab width = 6.6 sq. in.

$$f_r = 10{,}000 \text{ psi}$$

$$N_c = 6.6(10{,}000)/5{,}960 = 11.0 \text{ connectors}$$

These must be placed within a distance equal to one-third the effective slab width.

Bearing Stiffeners

Check if bearing stiffeners are required at the abutments. Service load reactions are:

Non-composite dead load	36k
Composite dead load	6k
Live load plus impact	60k
Total reaction =	102k

Shear stress in web $= \dfrac{102}{(30.68)(.71)} = 4.68$ ksi

Allowable shear stress in web $= 17.0$ ksi

$$\frac{4.68}{17} = 27.5\%$$

AASHTO 10.33.2 provides that bearing stiffeners are not required if shear stresses do not exceed 75% of the allowable. Bearing stiffeners will not be provided at the abutments.

Bearing stiffeners at interior supports:

The autostress design procedures require that bearing stiffeners be placed at the locations of rotating plastic hinges.

Service load reactions at the interior supports:

Non-composite dead load	126k
Composite dead load	18k
Live load plus impact	89k
Total reaction =	233k

Minimum thickness of bearing stiffeners (AASHTO 10.34.6.1)

$$= \frac{b'}{12}\sqrt{\frac{F_y}{33{,}000}}$$

$$= \frac{7}{12}\sqrt{\frac{50{,}000}{33{,}000}} = .718$$

Use ¾-in. thickness

Try 7 × ¾ bearing stiffeners

Check bearing stress of stiffeners on beam flanges.

Actual bearing stress $= \dfrac{233}{2(7-1)(.75)} = 25.9$ ksi

Allowable bearing stress $= 40$ ksi

40 ksi > 25.9 ksi **o.k.**

Check capacity of stiffeners and web section as a column.

Area = 12.78(0.71) + 2(0.75)(7) = 19.58 in.^{2}

$$I_x = 2(7)(0.75)\left(\frac{0.71}{2} + 3.5\cos 29°\right)^2 = 122.8 \text{ in.}$$

$$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{122.8}{19.58}} = 2.50$$

$$\frac{kl}{r} = \frac{(1)(30.68 - 2(1.185))}{2.50} = 11.3$$

$$\text{Allowable compressive stress} = 23{,}580 - 1.03\left(\frac{kl}{r}\right)^2$$

$$= 23{,}580 - 1.03(11.3)^2$$

$$= 23{,}450 \text{ psi}$$

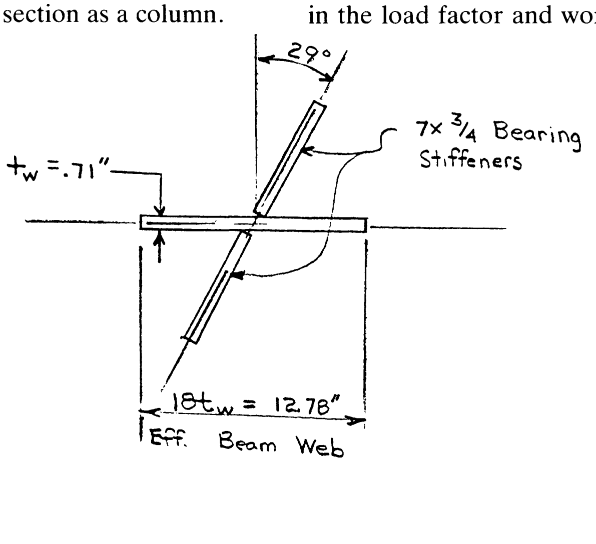
$$\text{Actual stress} = \frac{233{,}000}{1{,}958} = 11{,}900 \text{ psi}$$

11,900 psi < 23,450 psi **o.k.**

Check capacity of 5⁄16-in. continuous fillet welds connecting bearing stiffeners to beam webs.

= 0.3125(0.27)(70 ksi)(0.707)(4)(27) = 451k

451k > 233k **o.k.**



Summary

The autostress design was compared to rolled beam designs using conventional working stress and load factor designs. All of the comparable designs do not use cover plates. Comparison is as follows:

Design Type	Beam Section	Steel Weight
Working Stress	W33 × 241	364,000 lb.
Load Factor	W30 × 211	324,000 lb.
Autostress	W30 × 191	312,600 lb.

The autostress design is a 14% savings over the working stress design and a 3.5% savings over the load factor design. The autostress design contains approximately 11,500 pounds of lateral and intermediate bracing that is not used in the load factor and working stress designs.