Example 3.2
Design of a Composite Interior Steel Stringer

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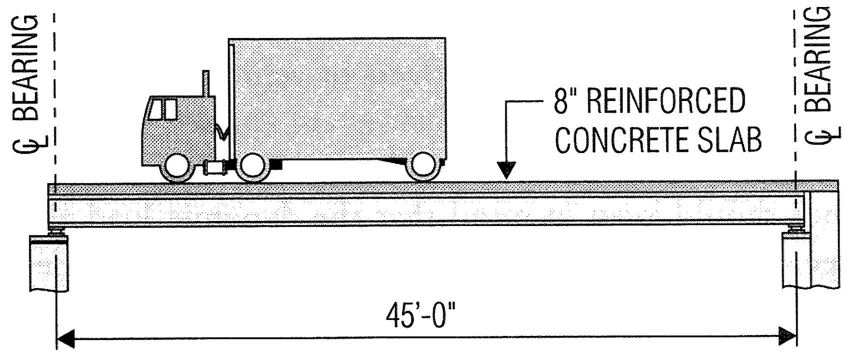
PROBLEM: Design the interior stringer for the bridge cross section given in Design Example 3.1. The bridge elevation is shown below.



GIVEN:

All cross-sectional properties of Design Example 3.1.
Span length of 45 ft centerline to centerline of bearings.
Average haunch depth of 2 in.
Unshored construction.
Account for 25 psf future wearing surface.
Span is simply supported.
Overpass is a major highway with ADTT of 3000.

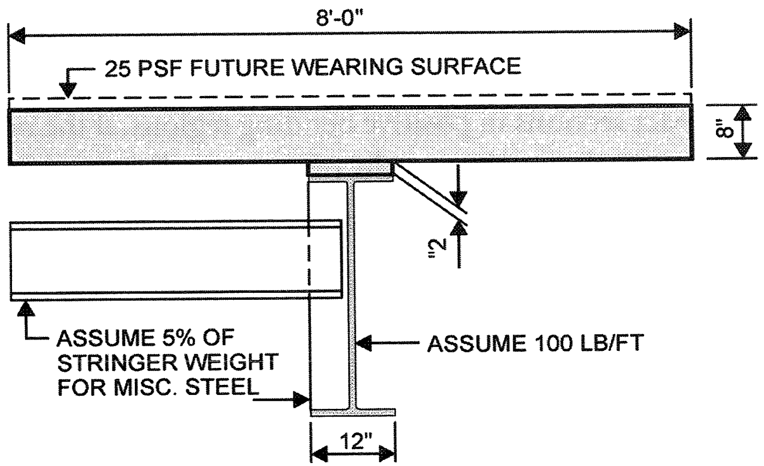
STEP 1: Compute the Effective Flange Width

The effective flange width is defined as
The least of:

$1/4 \times$ span length $= (0.25)(45.00)$	$= 11.25$ ft
Center-to-center between stringers	$= 8.00$ ft
$12 \times$ min. slab thickness $+ 1/2$ flange width	
$= [(12)(7.5) + (0.5)(9.0)]/12$	$= 7.88$ ft

$$\Rightarrow b_{eff} = 7.88 \text{ ft}$$

- See 3.10.2 @P165 for Unshored Construction.
- ADTT: Averaged Daily Truck Traffic
- See 3.10.3 @P165 for Effective Flange Width.



STEP 2: Compute the dead load on noncomposite section.

The dead load is composed of the following items:

$$DC_{slab} = (b)(\text{slab thickness})(w_{conc}) = (8.0)(8.0/12)(0.150) = 0.800\text{ k/ft}$$

$$DC_{haunch} = (\text{haunch width})(\text{haunch thickness})(w_{conc}) = (1.0)(2.0/12)(0.150) = 0.025\text{ k/ft}$$

$$DC_{steel} = (\text{assumed stringer weight}) + (\text{misc. steel}) = 0.100 + (5\%)(0.100) = 0.105\text{ k/ft}$$

$$\Rightarrow DC_1 = 0.930\text{ k/ft}$$

$$M_{DC1} = \frac{wL^2}{8} = \frac{(0.930)(45.0^2)}{8} = 235.41\text{ k-ft}$$

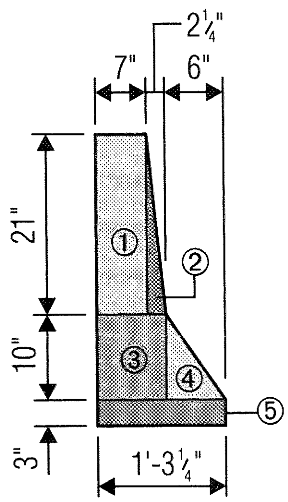
$$\Rightarrow M_{DC1} = 235.41\text{ k-ft}$$

$$V_{DC1} = \frac{wL}{2} = \frac{(0.930)(45.0)}{2} = 20.93\text{ k-ft}$$

$$\Rightarrow V_{DC1} = 20.93\text{ k}$$

- Girder weight is assumed at this moment, and will be updated by exact value.

STEP 3: Compute the dead load on composite section.



Calculate weight of parapet by first computing the area of its cross section:

$$A_1 = (7.0)(21.0) = 147.00\text{ in}^2$$
$$A_2 = \tfrac{1}{2}(2.25)(21.0) = 23.63\text{ in}^2$$
$$A_3 = (9.25)(10.0) = 92.50\text{ in}^2$$
$$A_4 = \tfrac{1}{2}(6.0)(10.0) = 30.00\text{ in}^2$$
$$A_5 = (15.25)(3.0) = 45.75\text{ in}^2$$
$$A_P = 338.88\text{ in}^2$$

$$w_P = (338.88)(1/12^2)(0.150) = 0.353\text{ k/ft}$$

There are two parapets on the bridge that are distributed over all six stringers:

$$w = 2\text{ parapets}\cdot\frac{w_p}{\text{no. stringers}} = 2\cdot\frac{0.353\text{ k/ft}}{6\text{ stringers}} = 0.118\text{ k/ft}$$

$$M_{DC2} = \frac{wL^2}{8} = \frac{(0.118)(45.0^2)}{8} = 29.87\text{ k-ft}$$

$$\Rightarrow M_{DC2} = 29.87\text{ k-ft}$$

$$V_{DC2} = \frac{wL}{2} = \frac{(0.118)(45.0)}{2} = 2.66\text{ k}$$

$$\Rightarrow V_{DC2} = 2.66\text{ k}$$

- Superimposed dead loads are those loads placed on the bridge after the deck has cured.

STEP 4: Compute wearing surface on composite section:

$$DW = \frac{\text{(width of roadway)(future wearing surface)}}{\text{number of stringers}}$$

$$= \frac{(44\text{ ft})(0.025\text{ k/ft}^2)}{6\text{ stringers}} = 0.183\text{ k/ft}$$

$$M_{DW} = \frac{wL^2}{8} = \frac{(0.183)(45.0)^2}{8} = 46.32\text{ k-ft}$$

$$\Rightarrow M_{DW} = 46.32\text{ k/ft}$$

$$V_{DW} = \frac{wL}{2} = \frac{(0.183)(45.0)}{2} = 4.12\text{ k}$$

$$\Rightarrow V_{DW} = 4.12\text{ k}$$

STEP 5: Compute live load distribution factors.

We first assume the stringer size W24 × 76:

$A = 22.4\text{ in}^2$, $I = 2100\text{ in}^4$, $d = 23.92\text{ in}$

The distance between the center of gravity of the stringer and the center of gravity of the deck:

$$e_g = \frac{7.5}{2} + 2.0 + \frac{23.92}{2} = 17.71\text{ in}$$

For 4.5 ksi concrete, $n = 8$.

$$K_g = n(I + Ae_g^2) = (8)[2100 + (22.4)(17.71)^2] = 73{,}005\text{ in}^4$$

Live load distribution factor for moment—strength limit state:

$$DF_m = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

$$= 0.075 + \left(\frac{8.0}{9.5}\right)^{0.6}\left(\frac{8.0}{45.0}\right)^{0.2}\left(\frac{73{,}005}{12.0\cdot 45.0\cdot 7.5^3}\right)^{0.1}$$

$$\Rightarrow DF_m = 0.645$$

- For a simply supported bridge, a range of 1/20 to 1/27 of the span length is normally used as the depth of the stringer.
- See 3.8@P132 for Distribution Factor.
- See 3.10.4@P167 for modular ratio *n*.

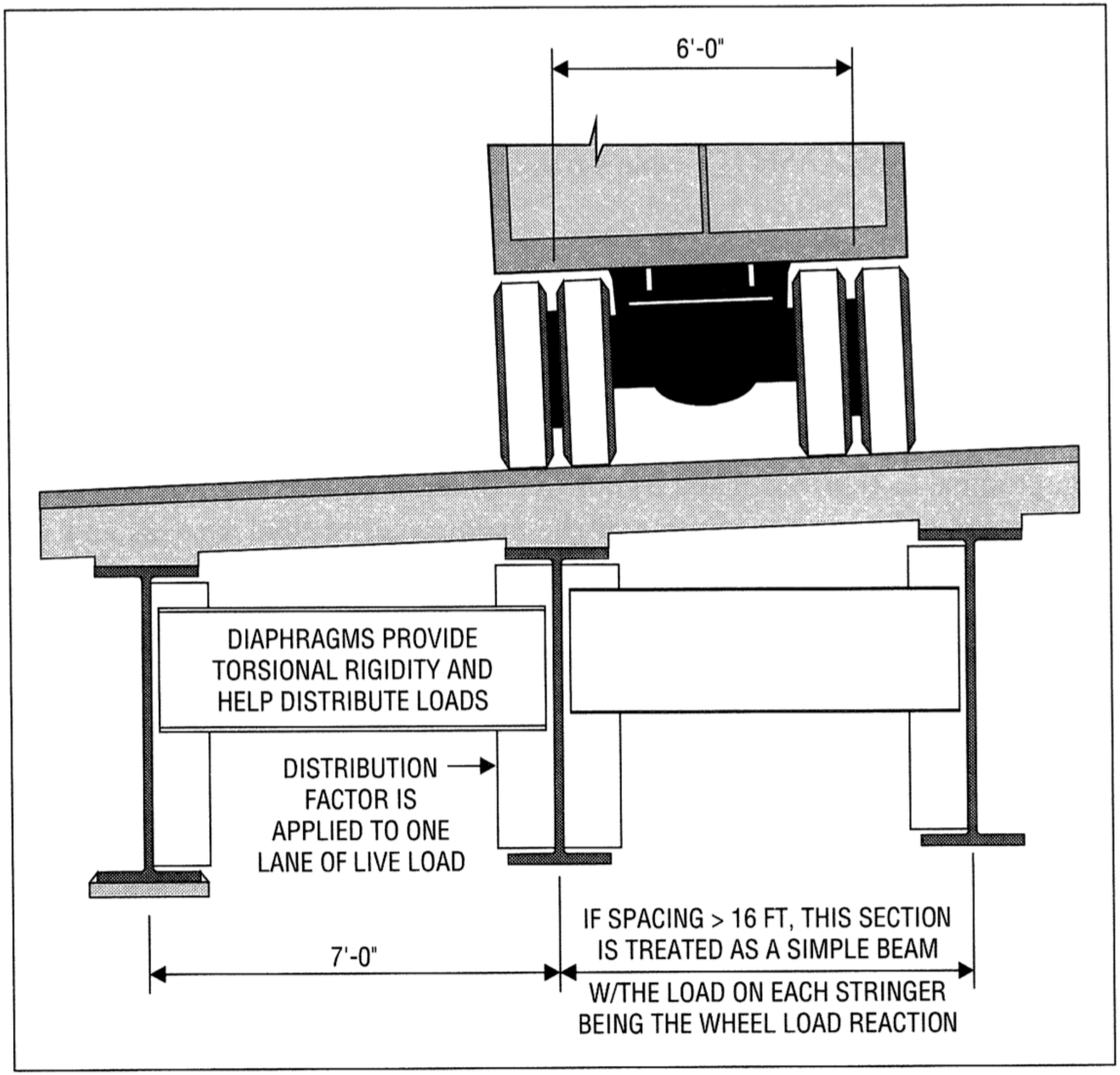


Figure 3.21 Wheel load distribution for a slab-on-stringer bridge.

TRUCK LOADS ARE resisted by many girders, but the loads are not shared by the girders equally. The maximum percentage of a truck load one girder may take, which is called live load distribution factor, is a function of many factors such as girder spacing, span length, deck thickness, girder sectional properties, etc (see Figure 3.21). Note that the live load distribution factors for bending moment and for shear force are slightly different.

If one design lane is loaded,

$$0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \qquad \text{(Eq. 3.32)}$$

If two or more design lanes are loaded,

$$0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \qquad \text{(Eq. 3.33)}$$

The live load distribution factor for shear in interior stringers is as follows:

If one design lane is loaded,

$$0.36+\frac{S}{25} \qquad \text{(Eq. 3.34)}$$

If two or more design lanes are loaded,

$$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0} \qquad \text{(Eq. 3.35)}$$

To apply these equations, the minimum number of stringers is four, and the bridge has to meet the following conditions:

$$3.5 \le S \le 16.0$$

$$4.5 \le t_s \le 12.0$$

$$20 \le L \le 240$$

$$10{,}000 \le K_g \le 7{,}000{,}000$$

where S = spacing of stringers, ft
L = span length, ft
K_g = longitudinal stiffness parameter of stringer, in^4
t_s = depth of concrete slab, in

$$K_g = n\left(I + Ae_g^2\right) \qquad \text{(Eq. 3.36)}$$

n = ratio of modulus of elasticity between stringer material and concrete deck
I = moment of inertia of stringer, in^4
A = section area of stringer, in^2
e_g = distance between centers of gravity of stringer and deck, in

STEP 5: Compute live load distribution factor (*continued*).

Live load distribution factor for shear—strength limit state:

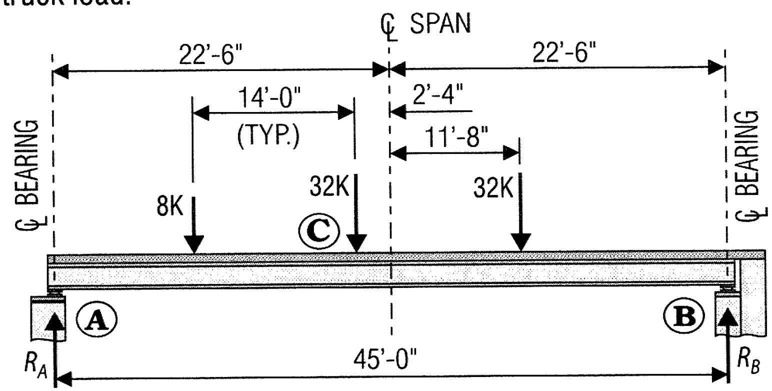
$$DF_s = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 = 0.2 + \frac{8}{12} - \left(\frac{8}{35}\right)^2 = 0.814$$

$\Rightarrow DF_s = 0.814$

STEP 6: Compute live load moment and shear.

For short-span bridges, both HL-93 truck and tandem load should be checked to determine the worst case.

For truck load:



First, solve for the reactions by summing moments about point *A*:

$$\sum M_A = 0$$

$$(8\,\text{k} \cdot 6.167\text{ ft}) + (32\,\text{k} \cdot 20.167\text{ ft}) + (32\,\text{k} \cdot 34.167\text{ ft}) - (R_B \cdot 45\text{ ft}) = 0$$

$$R_B = \frac{1788\text{ ft} \cdot \text{k}}{45\text{ ft}} = 39.734\,\text{k} \quad \text{so} \quad R_A = 72\,\text{k} - 39.734\,\text{k} = 32.266\,\text{k}$$

Now, compute the maximum live load moment:

$$M_{LL} = M_{MAX} = (R_A \cdot 20.167\text{ ft}) - (8\,\text{k} \cdot 14\text{ ft}) = 538.70\text{ k-ft}$$

- Homework#3.1 for the maximum live load moment.

STEP 6: Compute live moment and shear (*continued*).

The maximum shear is at support:

$$V_{LL} = 32 + \frac{(32)(31) + (8)(17)}{45} = 57.06 \text{ k}$$

Apply the same principle, we can get the maximum moment and shear for tandem load:

$M_{LL} = 513.60 \text{ k-ft} \qquad V_{LL} = 47.78 \text{ k}$

Therefore, truck load governs for both moment and shear:

$$\Rightarrow M_{LL} = 538.70 \text{ k-ft}$$

$$\Rightarrow V_{LL} = 57.06 \text{ k}$$

For design lane load:

$$M_L = \frac{wL^2}{8} = \frac{(0.64)(45.0)^2}{8} = 162.00 \text{ k-ft}$$

$$V_L = \frac{wL}{2} = \frac{(0.64)(45.0)}{2} = 14.40 \text{ k}$$

The dynamic allowance (applies only to truck or tandem load)

IM = 0.33

The live and impact applied to an interior stringer:

$$M_{LL+I} = (0.645)(1.33 \times 538.70 + 162.00) = 566.61 \text{ k-ft}$$

$$V_{LL+I} = (0.814)(1.33 \times 57.06 + 14.40) = 73.50 \text{ k}$$

$$\Rightarrow M_{LL+I} = 566.61 \text{ k-ft}$$

$$\Rightarrow V_{LL+I} = 73.50 \text{ k}$$

- Confirm the maximum moment and shear for tandem load by yourself.
- See slide #11 for design lane load in PPT for Design Loads and Combinations.
- See slide #13 for dynamic load allowance in PPT for Design Loads and Combinations.

STEP 7: Compute factored moment and shear.

The following load combinations will be used:

Strength I: $1.25DC + 1.50DW + 1.75(LL + IM)$

$$M_u = 1.25(\underbrace{235.41}_{\text{DC1}} + \underbrace{29.87}_{\text{DC2}}) + (1.50)(\underbrace{46.32}_{\text{DW}}) + (1.75)(566.61)$$

$$= 1392.6 \text{ k-ft}$$

$$V_u = 1.25(20.93 + 2.66) + (1.50)(4.12) + (1.75)(73.50)$$

$$= 164.29 \text{ k}$$

$\Rightarrow M_u = 1392.6$ k-ft

$\Rightarrow V_u = 164.29$ k

STEP 8: Choose a preliminary section.

We try section W24 × 76, grade 50 ksi steel, and ignore reinforcement in the deck.

First calculate the compression block height and plastic moment capacity for the composite section:

$$a = \frac{AF_y}{0.85 f'_c b_{\text{eff}}} = \frac{(22.4)(50.0)}{(0.85)(4.5)(7.88)(12)} = 3.10 \text{ in}$$

$$M_p = AF_y\left(d - \frac{a}{2}\right) = (22.4)(50.0)\left(\frac{23.92}{2} + 2.0 + 7.5 - \frac{3.1}{2}\right)\left(\frac{1}{12}\right)$$

$$= 1858.3 \text{ k-ft}$$

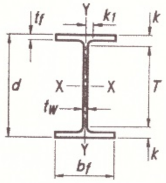


Table 1–1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Workable Gage
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W24×370[h]	109	28.0	28	1.52	1 1/2	3/4	13.7	13 5/8	2.72	2 3/4	3.22	3 5/8	1 9/16	20 3/4	5 1/2
×335[h]	98.4	27.5	27 1/2	1.38	1 3/8	11/16	13.5	13 1/2	2.48	2 1/2	2.98	3 3/8	1 1/2		
×306[h]	89.8	27.1	27 1/8	1.26	1 1/4	5/8	13.4	13 3/8	2.28	2 1/4	2.78	3 3/16	1 7/16		
x279[h]	82.0	26.7	26 3/4	1.16	1 3/16	5/8	13.3	13 1/4	2.09	2 1/16	2.59	3	1 7/16		
×250	73.5	26.3	26 3/8	1.04	1 1/16	9/16	13.2	13 1/8	1.89	1 7/8	2.39	2 13/16	1 3/8		
×229	67.2	26.0	26	0.960	15/16	1/2	13.1	13 1/8	1.73	1 3/4	2.23	2 5/8	1 5/16		
×207	60.7	25.7	25 3/4	0.870	7/8	7/16	13.0	13	1.57	1 9/16	2.07	2 1/2	1 1/4		
×192	56.3	25.5	25 1/2	0.810	13/16	7/16	13.0	13	1.46	1 7/16	1.96	2 3/8	1 1/4		
×176	51.7	25.2	25 1/4	0.750	3/4	3/8	12.9	12 7/8	1.34	1 5/16	1.84	2 1/4	1 3/16		
×162	47.7	25.0	25	0.705	11/16	3/8	13.0	13	1.22	1 1/4	1.72	2 1/8	1 3/16		
×146	43.0	24.7	24 3/4	0.650	5/8	5/16	12.9	12 7/8	1.09	1 1/16	1.59	2	1 1/8		
×131	38.5	24.5	24 1/2	0.605	5/8	5/16	12.9	12 7/8	0.960	15/16	1.46	1 7/8	1 1/8		
×117[c]	34.4	24.3	24 1/4	0.550	9/16	5/16	12.8	12 3/4	0.850	7/8	1.35	1 3/4	1 1/8		
×104[c]	30.6	24.1	24	0.500	1/2	1/4	12.8	12 3/4	0.750	3/4	1.25	1 5/8	1 1/16		
W24×103[c]	30.3	24.5	24 1/2	0.550	9/16	5/16	9.00	9	0.980	1	1.48	1 7/8	1 1/8	20 3/4	5 1/2
×94[c]	27.7	24.3	24 1/4	0.515	1/2	1/4	9.07	9 1/8	0.875	7/8	1.38	1 3/4	1 1/16		
×84[c]	24.7	24.1	24 1/8	0.470	1/2	1/4	9.02	9	0.770	3/4	1.27	1 11/16	1 1/16		
×76[c]	22.4	23.9	23 7/8	0.440	7/16	1/4	8.99	9	0.680	11/16	1.18	1 9/16	1 1/16		
×68[c]	20.1	23.7	23 3/4	0.415	7/16	1/4	8.97	9	0.585	9/16	1.09	1 1/2	1 1/16		
W24×62[c]	18.2	23.7	23 3/4	0.430	7/16	1/4	7.04	7	0.590	9/16	1.09	1 1/2	1 1/16	20 3/4	3 1/2[g]
×55[c,v]	16.2	23.6	23 5/8	0.395	3/8	3/16	7.01	7	0.505	1/2	1.01	1 7/16	1	20 3/4	3 1/2[g]
W21×201	59.2	23.0	23	0.910	15/16	1/2	12.6	12 5/8	1.63	1 5/8	2.13	2 1/2	1 5/16	18	5 1/2
×182	53.6	22.7	22 3/4	0.830	13/16	7/16	12.5	12 1/2	1.48	1 1/2	1.98	2 3/8	1 1/4		
×166	48.8	22.5	22 1/2	0.750	3/4	3/8	12.4	12 3/8	1.36	1 3/8	1.86	2 1/4	1 3/16		
×147	43.2	22.1	22	0.720	3/4	3/8	12.5	12 1/2	1.15	1 1/8	1.65	2	1 3/16		
×132	38.8	21.8	21 7/8	0.650	5/8	5/16	12.4	12 1/2	1.04	1 1/16	1.54	1 15/16	1 1/8		
×122	35.9	21.7	21 5/8	0.600	5/8	5/16	12.4	12 3/8	0.960	15/16	1.46	1 13/16	1 1/8		
×111	32.7	21.5	21 1/2	0.550	9/16	5/16	12.3	12 3/8	0.875	7/8	1.38	1 3/4	1 1/8		
×101[c]	29.8	21.4	21 3/8	0.500	1/2	1/4	12.3	12 1/4	0.800	13/16	1.30	1 11/16	1 1/16		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi.

Table 1–1 (continued)
W Shapes
Properties

W24 – W21

Nom-inal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	r_{ts}	h_o	Torsional Properties J	Torsional Properties C_w
lb/ft			in.⁴	in.³	in.	in.³	in.⁴	in.³	in.	in.³	in.	in.	in.⁴	in.⁶
370	2.51	14.2	13400	957	11.1	1130	1160	170	3.27	267	3.92	25.3	201	186000
335	2.73	15.6	11900	864	11.0	1020	1030	152	3.23	238	3.86	25.0	152	161000
306	2.94	17.1	10700	789	10.9	922	919	137	3.20	214	3.81	24.9	117	142000
279	3.18	18.6	9600	718	10.8	835	823	124	3.17	193	3.76	24.6	90.5	125000
250	3.49	20.7	8490	644	10.7	744	724	110	3.14	171	3.71	24.5	66.6	108000
229	3.79	22.5	7650	588	10.7	675	651	99.4	3.11	154	3.67	24.3	51.3	96100
207	4.14	24.8	6820	531	10.6	606	578	88.8	3.08	137	3.62	24.1	38.3	84100
192	4.43	26.6	6260	491	10.5	559	530	81.8	3.07	126	3.60	24.0	30.8	76300
176	4.81	28.7	5680	450	10.5	511	479	74.3	3.04	115	3.57	23.9	23.9	68400
162	5.31	30.6	5170	414	10.4	468	443	68.4	3.05	105	3.57	23.8	18.5	62600
146	5.92	33.2	4580	371	10.3	418	391	60.5	3.01	93.2	3.53	23.7	13.4	54600
131	6.70	35.6	4020	329	10.2	370	340	53.0	2.97	81.5	3.49	23.5	9.50	47100
117	7.53	39.2	3540	291	10.1	327	297	46.5	2.94	71.4	3.46	23.4	6.72	40800
104	8.50	43.1	3100	258	10.1	289	259	40.7	2.91	62.4	3.42	23.3	4.72	35200
103	4.59	39.2	3000	245	10.0	280	119	26.5	1.99	41.5	2.40	23.6	7.07	16600
94	5.18	41.9	2700	222	9.87	254	109	24.0	1.98	37.5	2.40	23.4	5.26	15000
84	5.86	45.9	2370	196	9.79	224	94.4	20.9	1.95	32.6	2.37	23.3	3.70	12800
76	6.61	49.0	2100	176	9.69	200	82.5	18.4	1.92	28.6	2.34	23.2	2.68	11100
68	7.66	52.0	1830	154	9.55	177	70.4	15.7	1.87	24.5	2.30	23.1	1.87	9430
62	5.97	50.1	1550	131	9.23	153	34.5	9.80	1.38	15.7	1.75	23.2	1.71	4620
55	6.94	54.6	1350	114	9.11	134	29.1	8.30	1.34	13.3	1.71	23.1	1.18	3870
201	3.86	20.6	5310	461	9.47	530	542	86.1	3.02	133	3.55	21.4	40.9	62000
182	4.22	22.6	4730	417	9.40	476	483	77.2	3.00	119	3.51	21.2	30.7	54400
166	4.57	25.0	4280	380	9.36	432	435	70.0	2.99	108	3.48	21.1	23.6	48500
147	5.44	26.1	3630	329	9.17	373	376	60.1	2.95	92.6	3.45	20.9	15.4	41100
132	6.01	28.9	3220	295	9.12	333	333	53.5	2.93	82.3	3.42	20.8	11.3	36000
122	6.45	31.3	2960	273	9.09	307	305	49.2	2.92	75.6	3.40	20.7	8.98	32700
111	7.05	34.1	2670	249	9.05	279	274	44.5	2.90	68.2	3.37	20.6	6.83	29200
101	7.68	37.5	2420	227	9.02	253	248	40.3	2.89	61.7	3.35	20.6	5.21	26200

STEP 8: Choose a preliminary section (*continued*).

- The yield stress of steel is 50 ksi < 70 ksi.
- Web $\frac{D}{t_w} = \frac{21}{0.44} = 47.7 \le 150$

The plastic neutral axis is within the concrete slab, so Equation 3.74 is automatically satisfied.

Therefore, the section is a compact section.

$D_p = a = 3.10$ in

$D_t = 7.5 + 2.0 + 23.92 = 33.42$ in

So $D_p < 0.1D_t$

$M_n = M_p = 1858.3$ k-ft

Since the stringers are not subjected to lateral bending or torsion, $f_l = 0$.

$$M_u + \frac{1}{3} f_l \cdot S_{xt} = 1392.6 \text{ k-ft} \le \phi_f M_n = 1858.3 \text{ k-ft} \quad ✔$$

Check the ductility requirement:

$$\frac{D_p}{D_t} = \frac{3.10}{33.42} = 0.09 \le 0.42 \quad ✔$$

Check shear capacity:

k = 5 for unstiffened stringer.

$$1.12\sqrt{\frac{KE}{F_y}} = 1.12\sqrt{\frac{(5)(29{,}000)}{50}} = 60.3 > \frac{D}{t_w} = \frac{23.92}{0.44} = 54.4$$

So, $C = 1.0$.

$$V_n = CV_p = C(0.58\,F_y D t_w) = (1.0)(0.58)(50.0)(23.92)(0.44)$$

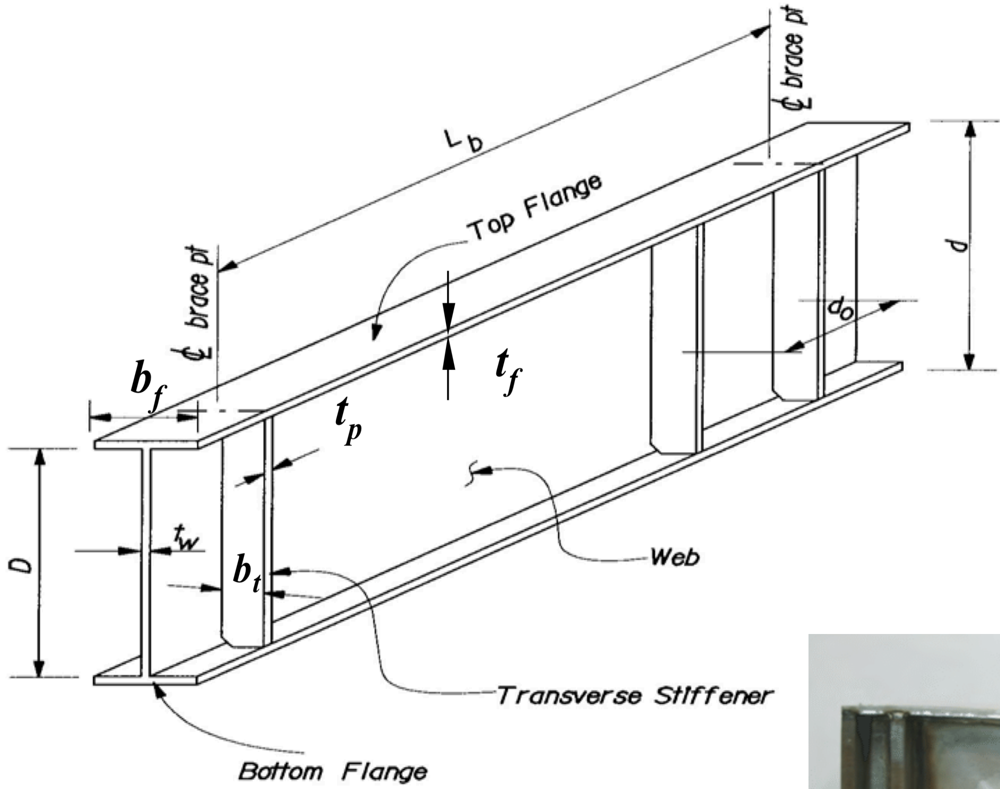
$$= 305.22 \text{ k}$$

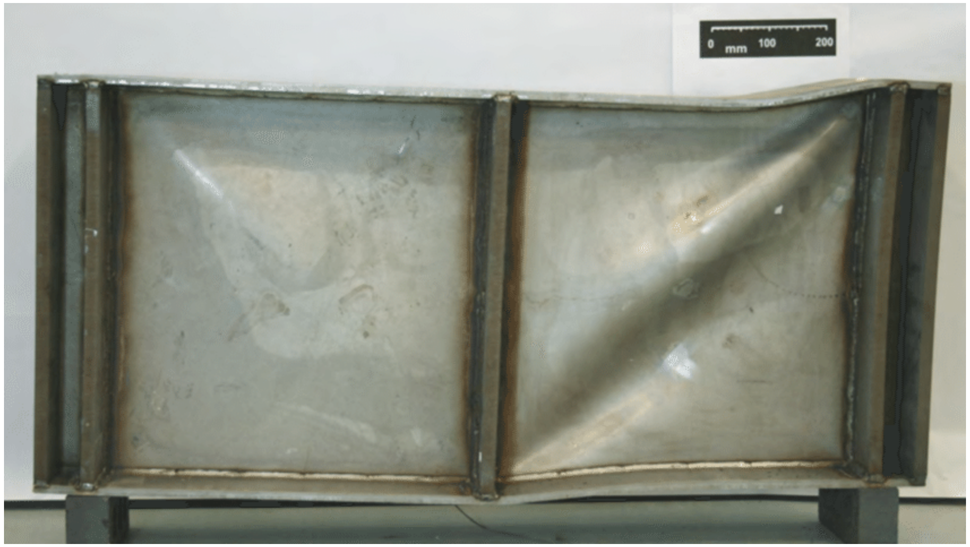
$$\phi_v V_n = 1.0 \cdot 305.22 = 305.2 \text{ k} \ge V_u = 164.29 \text{ k} \quad ✔$$

Therefore, the preliminary section meets strength requirements.

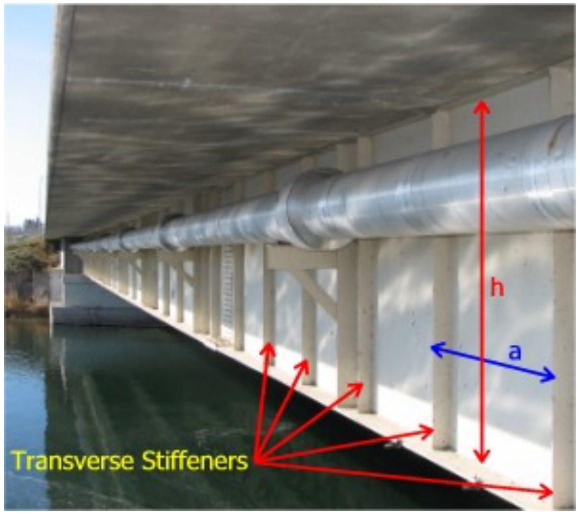
- See lecture note P99-104 and P108 for the flexural and shear strengths of composite slab-on-stringer deck.

Transverse Stiffeners and Shear-Dominant Failure mode of Web





Vertical Stiffeners, Longitudinal Stiffeners, and Bearing Stiffeners



Transverse Web Stiffener

Bearing Stiffener

Jacking Stiffeners

STEP 9: Compute composite section moment of inertia.

For superimposed dead loads, use $3n$ (= 24):

$$b_f = \frac{b_{\text{eff}}}{3n} = \frac{7.88 \cdot 12}{3 \cdot 8} = 3.94 \text{ in} \qquad I_{\text{deck}} = \frac{(3.94 \text{ in})(7.5 \text{ in})^3}{12} = 138.5 \text{ in}^4$$

Element	A, in^2	Y, in	AY, in^3	AY^2, in^4	I_o, in^4
W 24 × 76	22.4	11.96	267.90	3,204.1	2,100.0
Slab ($3n$)	29.55	29.67	876.75	26,013.1	138.5
Totals	51.95		1,144.65	29,217.2	2,238.5

$$I_z = \sum I_o + \sum AY^2 = 2238.5 \text{ in}^4 + 29{,}217.2 \text{ in}^4 = 31{,}456 \text{ in}^4$$

$$Y' = \frac{\sum AY}{\sum A} = \frac{1144.65 \text{ in}^3}{51.95 \text{ in}^2} = 22.03 \text{ in}$$

$$I = I_z - \left(\sum A\right)(Y')^2 = 31{,}456 \text{ in}^4 - (51.95 \text{ in}^2)(22.3 \text{ in})^2$$

$$= 6243.3 \text{ in}^4$$

$$S = \frac{I}{y'} = \frac{6243.3 \text{ in}^4}{22.03 \text{ in}} = 283.40 \text{ in}^3$$

For live loads, use n (= 8):

$$b_f = \frac{b_{\text{eff}}}{n} = \frac{7.88 \cdot 12}{8} = 11.82 \text{ in} \qquad I_{\text{deck}} = \frac{(11.82 \text{ in})(7.5 \text{ in})^3}{12} = 415.5 \text{ in}^4$$

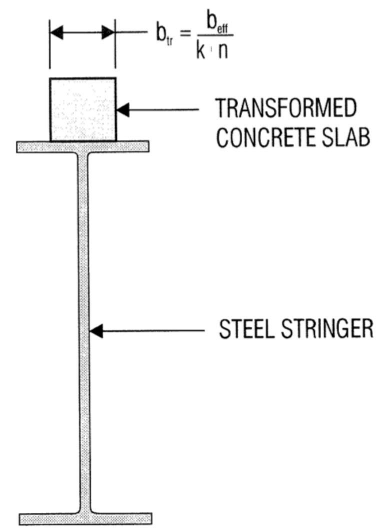
Element	A, in^2	Y, in	AY, in^3	AY^2, in^4	I_o, in^4
W 24 × 76	22.4	11.96	267.90	3,204.1	2,100.0
Slab (n=8)	88.65	29.67	2,630.25	78,039.4	415.5
Totals	111.05		2,898.15	81,243.5	2,515.5

AASHTO STANDARD

6.10.1.1.1b MODULAR RATIO

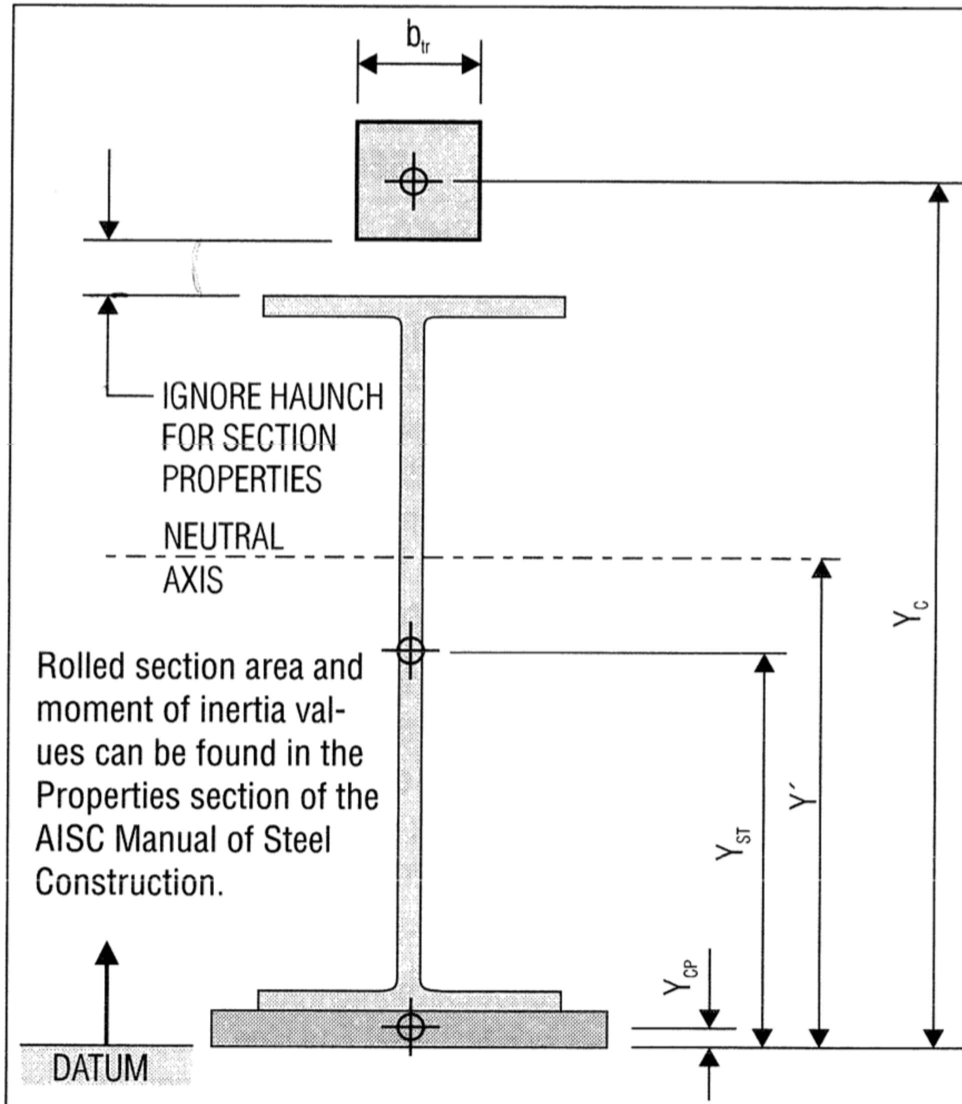
AASHTO offers a table of modular ratio values to be used for various strengths of concrete in the design of composite members.

Compressive Strength, f'_c	n
2.4–2.9 ksi	10
2.9–3.6 ksi	9
3.6–4.6 ksi	8
4.6–6.0 ksi	7
6.0 ksi or greater	6



TRANSFORMED SECTION

For k, see 3.10.5@P168.



STEP 1: Compute the area of each element in the section.
STEP 2: Compute distance to each element centroid.
STEP 3: Compute AY and AY^2 for each element.
STEP 4: Compute moment if inertia for each element (e.g., $I = bh^3/12$ for a rectangle).
STEP 5: Sum columns 2, 4, 5, and 6.
STEP 6: Solve the bottom three equations.

ELEMENT	**A** (in^2)	**Y** (in)	**AY** (in^3)	**AY**^{2} (in^4)	I_o (in^4)
COVER PLATE					
STRINGER					
SLAB					
Σ: TOTAL	2		4	5	6

Equation #1	Equation #2	Equation #3
$I_z = \sum I_o + \sum AY^2$	$Y' = \dfrac{\sum AY}{\sum A}$	$I = I_z + \left(\sum A\right)(Y')^2$

Figure 3.33 Method for calculating moment of inertia for a composite section.

STEP 9: Composite section moment of inertia (*continued*).

$$I_z = \sum I_o + \sum AY^2 = 2515.5 \text{ in}^4 + 81{,}243.5 \text{ in}^4 = 83{,}759 \text{ in}^4$$

$$Y' = \frac{\sum AY}{\sum A} = \frac{2898.15 \text{ in}^4}{111.05 \text{ in}^2} = 26.10 \text{ in}$$

$$I = I_z - \left(\sum A\right)(Y')^2 = 83{,}759 \text{ in}^4 - (111.05 \text{ in}^2)(26.10 \text{ in})^2$$

$$= 8110.6 \text{ in}^4$$

$$S = \frac{I}{y'} = \frac{8110.6 \text{ in}^4}{26.10 \text{ in}} = 310.75 \text{ in}^3$$

STEP 10: Check service limit state.

Service II load combination is used to check steel stress under the factored service loads:

$$f_{DL} = \frac{M_{DC1}}{S} = \frac{235.41 \cdot 12}{176.0} = 16.05 \text{ ksi}$$

$$f_{SDL} = \frac{M_{DC2} + M_{DW}}{S} = \frac{(29.87 + 46.32)(12)}{283.40} = 3.23 \text{ ksi}$$

$$f_{LL+I} = \frac{M_{LL+I}}{S} = \frac{566.61 \cdot 12}{310.75} = 21.88 \text{ ksi}$$

Total factored stress at bottom flange at service II load combination:

$$f_{bot} = 1.0(16.05 \text{ ksi} + 3.23 \text{ ksi}) + 1.3(21.88 \text{ ksi})$$

$$= 47.72 \text{ ksi} < f_y = 50.0 \text{ ksi} \quad ✔$$

STEP 11: Compute shear range in the stringer.

For fatigue limit state, only one truck is loaded.
Live load distribution factor for shear:

$$DF = \left(0.36 + \frac{S}{25.0}\right) = \left(0.36 + \frac{8.0}{25.0}\right) = 0.68$$

Live load impact for fatigue: $IM = 0.15$ √ 비교
Love load factor for fatigue limit state: $LF = 0.75$ – 표 3.4.1

$[x = 0]$ Positive shear:

$$+V_{x=0} = LF \cdot DF(1+IM)\left(32 + 32 \cdot \frac{15}{45} + 8 \cdot \frac{1}{45}\right)$$

$$= 0.75 \cdot (0.68)(1.15)(42.844) = 25.13 \text{ k}$$

$[x = 0]$ Negative shear:

$$-V_{x=0} = 0$$

$[x = 0]$ Total shear range

$$V_{x=0} = +V_{x=0} - (-V_{x=0}) = 25.13 - 0 = 25.13 \text{ k} \qquad \Rightarrow V_{x=0} = 25.13 \text{ k}$$

$[x = 4.5 \text{ ft}]$ Positive shear:

$$+V_{x=4.5} = LF \cdot DF(1+IM)\left(32 \cdot \frac{40.5}{45} + 32 \cdot \frac{10.5}{45}\right)$$

$$= 0.75(0.68)(1.15)(36.267) = 21.27 \text{ k}$$

$[x = 4.5 \text{ ft}]$ Negative shear:

$$-V_{x=4.5} = LF \cdot DF(1+IM)\left(8 \cdot \frac{-4.5}{45}\right)$$

$$= 0.75(0.68)(1.15)(-0.800) = -0.47 \text{ k}$$

- See slide #19 for the fatigue load in PPT for Design Loads and Combinations.
- See slide #13 for the impact factor for fatigue load in PPT for Design Loads and Combinations.
- See slide #21 for the impact factor for fatigue load in PPT for Design Loads and Combinations.
- The spacing of the two 32 kip axle loads should be 30ft.

STEP 11: Compute shear range in the stringer (*continued*).

[$x = 4.5$] Total shear range:

$$V_{x=4.5} = +V_{x=4.5} - (-V_{x=4.5}) = 21.27 - (-0.47) = 21.74 \text{ k}$$

$$\Rightarrow V_{x=4.5} = 21.74 \text{ k}$$

[$x = 9.0$] Positive shear:

$$+V_{x=9.0} = LF \cdot DF(1+IM)\left(32 \cdot \frac{36}{45} + 32 \cdot \frac{6}{45}\right)$$

$$= (0.75)(0.68)(1.15)(29.867) = 17.52 \text{ k}$$

$$-V_{x=9.0} = LF \cdot DF(1+IM)\left(32 \cdot \frac{-9}{45} + 8 \cdot \frac{22}{45}\right)$$

$$= (0.75)(0.68)(1.15)(-2.489) = -1.46 \text{ k}$$

[$x = 9.0$] Total shear range:

$$V_{x=9.0} = +V_{x=9.0} - (-V_{x=9.0}) = 17.52 - (-1.46) = 18.98 \text{ k}$$

$$\Rightarrow V_{x=9.0} = 18.98 \text{ k}$$

[$x = 22.5$ ft] Positive shear:

$$+V_{x=22.5} = LF \cdot DF(1+IM)\left(32 \cdot \frac{22.5}{45} + 8 \cdot \frac{8.5}{45}\right)$$

$$= (0.75)(0.68)(1.15)(17.511) = 10.27 \text{ k}$$

[$x = 22.5$ ft] Negative shear:

$$-V_{x=22.5} = LF \cdot DF(1+IM)\left(32 \cdot \frac{-22.5}{45} + 8 \cdot \frac{8.5}{45}\right)$$

$$= (0.75)(0.68)(1.15)(-14.489) = -8.50 \text{ k}$$

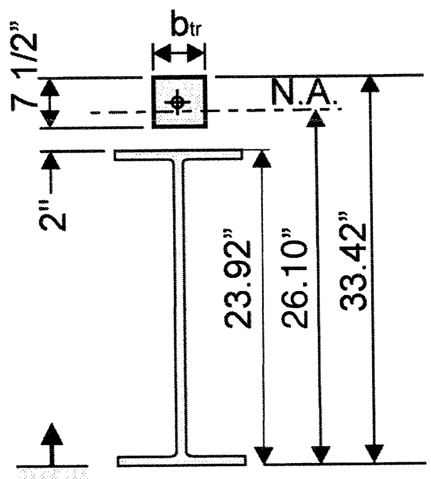
STEP 11: Compute shear range in the stringer (*continued*).

$[x = 22.5]$ Total shear range:

$$V_{x=22.5} = +V_{x=22.5} - (-V_{x=22.5}) = 10.27 - (-8.50) = 18.77\text{ k}$$

$$\Rightarrow V_{x=22.5} = 18.77\text{ k}$$

STEP 12: Shear studs based on fatigue criteria.



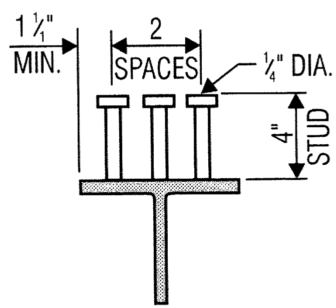
Since we have a constant cross-section use, I and Q for all locations:

$$I_x = 8110.6 \text{ (step 9)}$$

$$Q = (\text{area of steel stringer}) \times (\text{distance N.A. to stringer C.G.}) = (22.40)(26.10 - 11.96) = 316.74\text{ in}^3$$

$$S_r = \frac{V_r Q}{I} \quad \text{and} \quad p = \frac{(\#\text{ studs})(Z_r)}{S_r}$$

Major highway w/ADTT of 3000.



Use ¾ in diameter studs.

$$s \geq (4)(d) = (4)(0.75\text{ in}) = 3.0\text{ in}$$

$$b_f = 8.99\text{ in (flange width)}$$

$$s = \frac{b_f - (2 \cdot \text{edge distance})}{2\text{ spaces}}$$

$$= \frac{8.99\text{ in} - (2 \cdot 1.375\text{ in})}{2\text{ spaces}}$$

$$= 3.12\text{ in} > 3.0\text{ in} \quad ✔$$

- See pp.109-111 in the Lecture Note for the design of shear connectors.

STEP 12: Shear stud based on fatigue criteria (*continued*).

$$N = 365 \cdot 75 \cdot n \cdot p \cdot (\text{ADTT}) = 365 \cdot 75 \cdot 1.0 \cdot 0.85 \cdot 3000$$

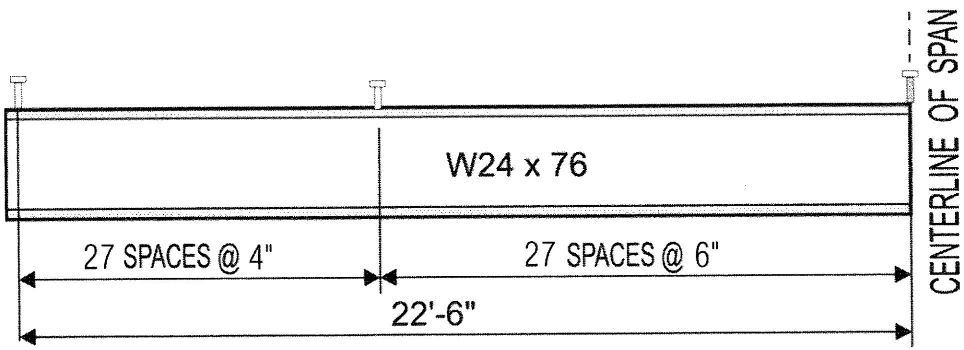
$$= 69.8 \times 10^6 \qquad (p = 0.85 \text{ for two-lane traffic})$$

$$\alpha = 34.5 - 4.28 \log N = 34.5 - 4.28 \log (69.8 \times 10^6) = 0.928$$

$$Z_r = \alpha d^2 \geq \frac{5.5 d^2}{2}$$

$$\text{So } Z_r = \frac{(5.5)(0.75)^2}{2} = 1.55\ k \qquad p = \frac{3Z_r}{S_r}$$

Point, ft	V_r, k	Q, in³	I, in⁴	S_r, k/in	p, in
$x = 0.0$	33.47	316.74	8111	0.981	4.74
$x = 4.5$	29.72	316.74	8111	0.849	5.48
$x = 9.0$	26.49	316.74	8111	0.741	6.27
$x = 22.5$	24.97	316.74	8111	0.733	6.34



Shear Stud Spacing

A WAY TO LAY OUT shear connectors is presented in Figure 3.35. The calculated spacing at the various beam locations is plotted as a curve passed through the points. Obviously, the more points taken, the more accurate the plot. In Design Example 3.2, four such points were taken. Since, for a simple span, shear is greatest at the supports, the greatest variation is seen here. When selecting the break point, the designer must take care that the specified spacing falls below the plotted curve. This will ensure the most conservative result. Our breakpoint is at $x = 9.0$ ft from the support. At this point, a 6.0 in spacing falls below the curve, so we are OK. The total number of studs is

$$3(28 \text{ studs} + 53 \text{ studs} + 28 \text{ studs})$$

since we have three studs per row, for a total of 327 studs.

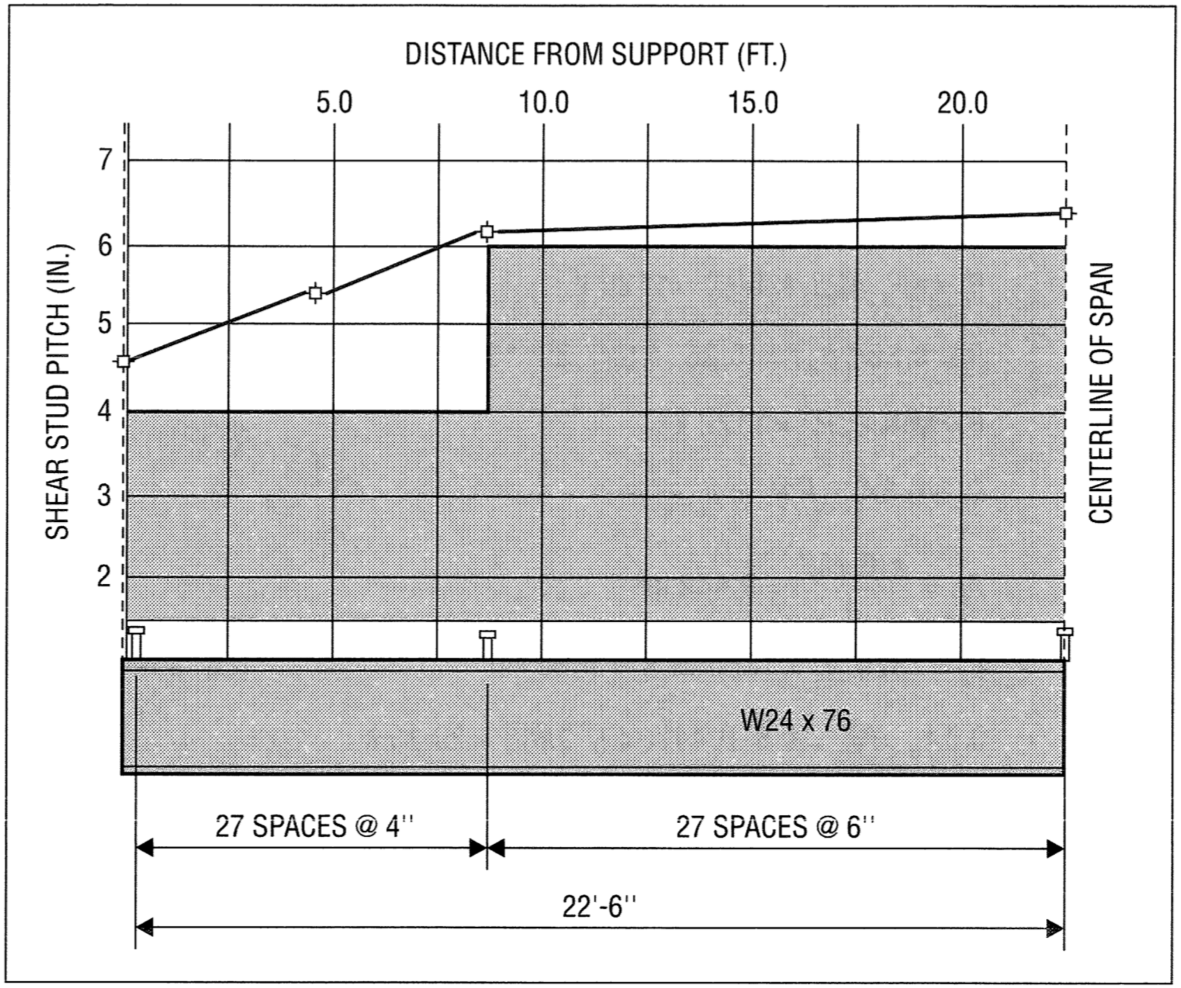


Figure 3.35 Shear connector layout for the beam presented in Design Example 3.2.

STEP 13: Check shear studs based on strength limit state.

$$P_1 = A_s F_y = 22.4 \cdot 50.0 = 1120 \text{ k}$$

$$P_2 = 0.85 f_c' \, b_{eff} t = (0.85 \cdot 4.5)(7.88 \cdot 12)(7.5) = 2713 \text{ k}$$

$$P = \text{MIN}(P_1, P_2) = 1120 \text{ k}$$

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \le A_{sc} F_u$$

7/21 D 36.2

$$A_{sc} = \frac{\pi (0.75)^2}{4} = 0.442 \text{ in}^2$$

$$E_c = 1820\sqrt{f_c'} = 1820\sqrt{4.5} = 3860 \text{ ksi} \qquad \text{(Eq. 3.66)}$$

$$0.5 A_{sc} \sqrt{f_c' E_c} = 0.5 \cdot 0.442 \sqrt{4.5 \cdot 3860} = 29.13 \text{ k}$$

$$A_{sc} F_u = 0.442 \cdot 70.0 = 30.94 \text{ k}$$

Therefore,

$$Q_n = 29.13 \text{ k}$$

$$\text{No. studs required} = \frac{P}{\phi_{sc} Q_n} = \frac{1120}{0.85 \cdot 29.13} = 45$$

26

$$\text{No. studs provided} = 3(27 + 36) = 159 \quad ✔$$

- E_s= 29,000 ksi (200GPa).
- For normal weight concrete (approximately 0.145 kcf) E_c= 1,820$\sqrt{f_c'}$
- If the actual unit weight of concrete used is known, E_c= $33{,}000 w^{1.5}\sqrt{f_c'}$
- The ultimate strength may also be obtained form Table 3.6@P197.

THANK YOU

for your attention!