Strengthening of Existing Composite Beams Using LRFD Procedures

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INTRODUCTION

Many times, the load carrying capacity of an existing composite steel floor beam needs to be increased beyond its original design capacity due to a change in occupancy use classification or the addition of a localized heavy load. For example, this situation occurs frequently in an office building that was initially designed using a live load of 50 pounds per square foot where a tenant wants to locate a library that requires a live load of 150 pounds per square foot. Or perhaps a tenant would like to place a very heavy object in a certain location that is beyond the capacity of the existing floor framing.

One of the many advantages of a steel framed structure is the relative ease and economy of the addition of reinforcing to the existing members to increase their load carrying capacity. There are several techniques available to the designer for adding reinforcement to an existing steel beam. Perhaps the most often used solution is to weld a steel shape to the bottom of the beam. This paper deals with the addition of either a flat steel plate or a WT section to the bottom of the beam to moderately increase its load carrying capacity. For a major increase in capacity, the method described in this paper could easily be extended to other reinforcing shapes.

This paper describes a procedure for the rapid direct solution of the required amount of steel reinforcement to be added to a composite beam to resist a given bending moment. The solution is obtained using the design aids provided and brief hand calculations. The procedure is based on Load and Resistance Factor Design (LRFD) Theory and is applicable to composite beams designed by either Allowable Stress Design (ASD) or LRFD and with either solid concrete slabs or slabs on metal deck.

THEORY

For this discussion, simply supported composite sections subjected only to positive bending, i.e. the top in compression and the bottom in tension, are considered. The nominal moment resisting capacity of a composite steel beam and concrete slab system based on LRFD Theory is derived assuming a plastic stress distribution throughout the cross-section. Thus, each steel element of the section is assumed to be stressed to the yield point of the material either in tension or compression, depending on which side of the Plastic Neutral Axis (PNA) it is on, and the concrete flange is always on the compression side of the section. The compressive force carried by the concrete slab is the smallest of the following:[1]

a) the sum of the products of the steel areas and their respective yield stresses,
b) 85 percent of the product of the specified concrete compressive strength and the area of the concrete (the Whitney stress block),
c) the sum of the strengths of the shear connectors between the point of maximum moment and the point of zero moment to either side.

One of the first two conditions a) or b) above will govern in the case of a fully composite beam and condition c) will govern in the case of a partially composite beam.[4] Since the total steel area is increasing with the addition of reinforcing, the first condition will always result in the largest of the three quantities. Thus, the smaller of conditions b) and c) will need to be determined to use in the analysis.

In determining the nominal moment capacity of a given composite beam, it is first necessary to find the location of the PNA. Using a plastic stress distribution, the PNA is located such that the compressive forces above it are equal to the tensile forces below it. The PNA can be located either in the concrete slab, in the top flange of the steel beam, or in the web of the steel beam, depending on many factors such as the number of shear connectors between the slab and beam, the size of the beam, the concrete slab parameters, and the size of the added bottom reinforcement.

To simplify the analysis, the PNA will be assumed to be located somewhere in the web of the steel beam. This assumption is valid in most cases since the PNA of an unreinforced composite beam is almost always located in the concrete slab or in the top flange of the steel beam. The addition of reinforcing to the bottom of the steel beam only serves to lower the PNA so chances are it will be located in the web. This assumption will need to be verified after a reinforcing section size is determined.

Figure 1 shows the plastic stress distribution for a compos-

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ite beam subjected to positive bending that is reinforced by the addition of a WT section to the bottom. The PNA is shown to be located in the web.

Note that the exact location of the PNA from the top flange of the beam, x, is unknown and is dependent on two variables, namely T_r, the tensile force in the reinforcing, and z, the distance from the bottom of the steel beam to the centroid of the reinforcing. The distance from the centroid of the reinforcing to the bottom of the steel beam, z, can be estimated accurately, depending on what shape of reinforcing is chosen. Thus, there remains two unknowns in determining the nominal moment capacity of the section in Figure 1; the distance from the top of the beam to the PNA, x, and the required force in the reinforcing, T_r. Since it is desired to solve directly for the amount of steel reinforcing required to resist a given factored moment, two equations can be written with two unknowns, T_r and x, and solved simultaneously for T_r.

The first equation is obtained by summing the horizontal forces acting on the cross-section. Referring to Figure 1:

$$C_c + C_{sf} + C_{sw} - T_{sw} - T_{sf} - T_r = 0 \qquad (1)$$

Note that C_{sf} and T_{sf}, the compressive force in the top flange and the tensile force in the bottom flange, respectively, are equal and opposite so that they cancel each other leaving:

$$C_c + C_{sw} = T_{sw} + T_r \qquad (2)$$

where:

$$C_{sw} = (x - t_f)t_w F_{yb} \qquad (3)$$

$$T_{sw} = (d - x - t_f)t_w F_{yb} \qquad (4)$$

After substituting Equations 3 and 4 into Equation 2 and simplifying, the following expression for x is obtained in terms of T_r:

$$x = d/2 + (T_r - C_c)/(2t_w F_{yb}) \qquad (5)$$

In order for this and subsequent derivations to be valid, the assumption that the PNA falls within the beam web must be verified with the following equation:

$$t_f \le x \le d - t_f \qquad (6)$$

The second equation is obtained by summing the moments about the bottom of the steel beam. Referring again to Figure 1:

$$M_n + C_c[d + h - a/2] + C_{sf}[d - t_f/2]$$
$$+ C_{sw}[d - t_f - (x - t_f)/2]$$
$$- T_{sw}[(d - x - t_f)/2 + t_f] - T_{sf}[t_f/2] + T_r[z] = 0 \qquad (7)$$

where:

$$C_{sf} = T_{sf} = b_f t_f F_{yb} \qquad (8)$$

After substituting Equations 3, 4 and 8 into Equation 7 and simplifying, the following expression is obtained:

$$M_n = C_c[d + h - a/2] + T_r z \; + t_w F_{yb}[2dx - x^2] + C \qquad (9)$$

where

$$C = b_f t_f F_{yb}[d - t_f] + t_w F_{yb}[t_f^2 - dt_f - d^2/2] \qquad (9a)$$

The constant C is dependent only on the cross-sectional dimensions of the bare steel beam and is given in Table 1.

Now, the expression for x in Equation 5 is substituted into Equation 9 and the following expression for the required tensile force in the reinforcing is obtained:

$$T_r = A[2z + d] + C_c$$
$$\pm 2A[z^2 + dz + B + (yC_c - M_u/\phi)/A]^{1/2} \qquad (10)$$

where:

$$A = t_w F_{yb} \qquad (10a)$$

$$B = d^2/2 + dt_f[b_f/t_w - 1] + t_f^2[1 - b_f/t_w] \qquad (10b)$$

$$C_c = 0.85 f_c' A_c \text{ or SUM } Q_n \text{ (smaller)} \qquad (10c)$$

$$y = z + d + h - a/2 \qquad (10d)$$

The constants A and B in Equations 10a and 10b, respectively, can be tabulated for any steel beam and yield strength and are given in Table 1. C_c in Equation 10c is a function of the concrete slab parameters. So, using Equation 10, the required tensile force in the reinforcing, T_r, can be computed, given the original steel beam size, number and size of shear connectors, slab thickness and strength, the height of the metal deck, and an assumed dimension for z.

Since plastic stress distributions are being considered, the

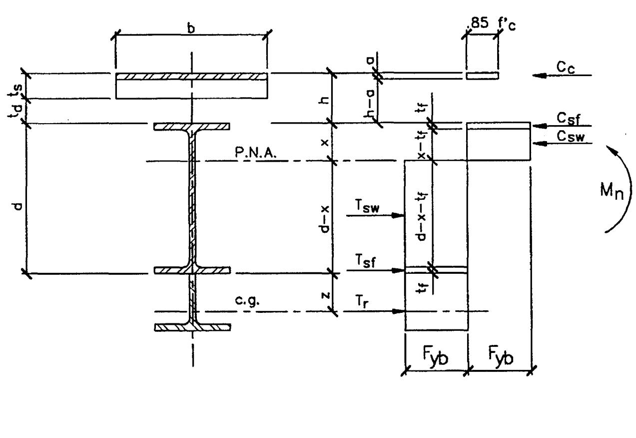

Fig. 1. Plastic stress distribution for a reinforced composite steel beam.

force in the reinforcing is related to the area of steel by the following equation:

$$A_{sr} = T_r / F_{yr} \tag{11}$$

where the subscript, r, refers to the reinforcing.

It was mentioned earlier that the distance z can be estimated fairly accurately. The two types of reinforcing considered in this paper are a flat plate and a WT section, with the web attached to the bottom flange of the beam.

When using a bottom reinforcing plate, it is recommended that the plate be wider than the bottom flange of the beam to avoid overhead welding. For a flat reinforcing bottom plate, the z distance is equal to exactly one half the plate thickness. In most cases it is accurate enough to assume that a 1-in. plate will be used. Thus, $z_{(plate)} = 0.5$ in.

For most WT sections, the distance from the free edge of the web to the centroid of the section is from about $.65d_T$ to about $.80d_T$, where d_T is the overall depth of the WT section. It is usually accurate enough to use 75 percent of the depth of the WT, or $z_{(WT)} = 0.75d_T$.

A step by step procedure, using the above analysis, to determine the required amount of reinforcing steel to be added to the bottom of an existing composite beam is as follows:

Step 1. Determine the existing beam parameters:

a. Steel beam size and specified yield strength.
b. Slab and deck thicknesses.
c. Specified concrete strength.
d. Number and size of shear connectors.

Step 2. Determine the maximum required factored moment and its location.

Step 3. Determine the smaller of:

a. C_c = SUM Q_n (between point of maximum moment and point of zero moment to either side)
b. $C_c = 0.85f_c't_sb$

where

b = Span Length/4 or Beam Spacing (smaller).

Step 4. Choose type of reinforcing and estimate distance z.

Step 5. Calculate $y = z + d + h - a/2$

where

$a = C_c / (0.85f_c'b)$

Step 6. Find constants A and B from Table 1.

Step 7. Calculate T_r using Equation 10.

Step 8. Calculate A_{sr} from Equation 11 and choose actual size of reinforcement.

Step 9. Calculate x from Equation 5 and make sure it satisfies Equation 6.

Step 10. Calculate actual moment capacity from Equation 9. Make sure that $\phi M_n \geq M_u$.

OTHER CONSIDERATIONS

Cut-Off Point

It is usually not necessary or practical to extend the reinforcing the full length of the beam. Usually, the designer chooses to stop the reinforcing at the point either side of the maximum moment where it is no longer required. To determine the locations of the cut-off points, the designer needs to know the location of the maximum moment that the section is being reinforced for, as well as the nominal moment capacity of the original composite steel beam.

Similar to other types of composite beams, the reinforcing may be terminated at a distance L beyond where it is theoretically required. The distance L is the distance that is required to fully develop the reinforcing section in tension. This is illustrated in the example problem.

Welds

The reinforcing section chosen must be adequately attached to the bottom of the existing beam to make the entire section act together. Field welds are typically chosen and are described here, although field bolting is also a possibility.

Since the tensile capacity of the reinforcing section is resisted by the welds at the cut-off points, and compression stability is not an issue, the designer needs to only consider a weld spacing that satisfies the requirements for built-up tension members given in the LRFD specifications. Generally, weld spacing should not exceed 24 times the thickness of the thinner element nor 12 inches. Also, the spacing should limit the slenderness ratio of the reinforcing section to less than 300.

Beam Shear, End Connections, and Column Strength

Since a beam is being reinforced to be able to carry more load, there is a possibility that the original end connection design is not capable of carrying the new end reaction. In addition, the shear in the beam web may exceed its capacity, especially if the beam is coped. The designer, of course, must check these items.

The capacity of the existing connection must be determined by analysis or by AISC Tables using the actual connection parameters. If the existing capacity is not satisfactory, then there are a number of choices available to the designer to strengthen the connection. Some of these may include a seat angle or additional welds or bolts, depending on the conditions. The designer must be aware of the limitations on the load sharing of bolts and welds.

There may also be a possibility that the existing column and foundation may be overloaded due to the increased carrying capacity of the floor beams, especially in low-rise buildings. The designer must check these and strengthen them if found deficient. This is usually not a consideration, however, where the floor of a building is only locally reinforced.

Deflections

To accurately compute the deflection of the reinforced section would be a very tedious manual chore. In order to do so, two different moments of inertia, one for the reinforced portion and one for the unreinforced portion, would need to be determined elastically. An easier, approximate procedure is described here.

The "Lower Bound Elastic Moment of Inertia" tables are found in the LRFD *Manual* will be used. It will be assumed that the moment of inertia for the reinforced section will dominate the deflection characteristics of the beam and, thus, the variation along the span need not be considered. The moment of inertia contributions for the composite beam and the reinforcing section will simply be summed.

From the LRFD *Manual*, the lower bound moment of inertia for the unreinforced beam section, using Point 7 for the location of the PNA setting *Y2* equal to *h*, can be found. This should be sufficiently accurate. To this must be added an approximation of the contribution for the reinforcing. This contribution can be estimated by taking the area of the reinforcing times the distance squared about the PNA. This is a very approximate, but simple calculation that is demonstrated in the example problem.

Local Buckling

Local buckling of the beam web, both at any concentrated loads and at the beam ends, must be checked. This is not illustrated here but it is only mentioned as a reminder.

EXAMPLE PROBLEM

This example illustrates the procedure described in this paper to calculate the required reinforcing size to upgrade a typical composite floor beam from a 50 pound per square foot (PSF) live load to a 100 PSF live load. There is plenty of room below the existing beam to be strengthened so a WT section is chosen as the reinforcing.

Step 1

Given:

Beam Span = 40 ft-0 in.
Beam Spacing = 10 ft-0 in.
W21×44-A36 Steel with 16¾-in. ϕ Shear Connectors over length of beam

Dead Loads:

3¼-in. Lightweight Concrete (f_c' = 3.5 ksi) on 3 in. × 20 ga. deck	= 48 psf
Mechanical, Electrical, Plumbing	= 5
Ceiling	= 3
Partitions	= 20
Total Dead Load	76 psf

Step 2

In determining the maximum applied factored moment, the 100 psf live load will be used. Since the live load is equal to or greater than 100 psf, the 20 psf partition load does not have to include the in the dead load. Thus:

$w_{udl} = 1.2(10 \times 56\text{ psf} + 50\text{ plf}) / 1000 = 0.73$ klf
$w_{ull} = 1.6(10 \times 100\text{ psf}) / 1000 = 1.60$
Total factored uniform load = 2.33 klf

$M_u = 2.33(40)^2 / 8 = 466$ ft-kips at midspan

Step 3

The capacity of a single ¾-in. ϕ shear connector is 19.8 kips and there are 8 each side of the maximum moment.

a) $C_c = 8 \times 19.8 = 158$ kips **Controls**
b) $C_c = 0.85 \times 3.5\text{ ksi} \times 3.25\text{-in.} \times 120\text{-in.} = 1160$ kips

Step 4

Since the required moment capacity is 466 ft-kips versus a tabulated ϕM_n of 360 ft-kips for the existing section (an increase of about 32 percent), a relatively small WT6 reinforcing section is chosen and z is estimated to be 4.5 inches.

Step 5

$$y = z + d + h - a/2 = 4.5 + 20.66 + 6.25 - 0.44/2 = 31.2 \text{ inches}$$

where

$$a = 158 / (0.85 \times 3.5 \times 120) = 0.44$$

Step 6

From Table 1, for a W21×44, A36 material: A = 12.6 and B = 373

Step 7

Using Equation 10:

$$T_r = 12.6[2 \times 4.5 + 20.66] + 158 \pm 2 \times 12.6[4.5^2 + 20.66 \times 4.5 + 373 + \{31.2 \times 158 - (466 \times 12/0.85)\} / 12.6]^{1/2} = 1007 \text{ or } 57 \text{ kips}$$

Since a quadratic equation is being solved, there are two roots. In this case, the 1007 kips required seems unreasonable so the 57 kips is assumed to be the correct root.

Step 8

From Equation 11, calculate the required area of the reinforcing:

$$A_{sr} = T_r / F_{yr} = 57 \text{ kips}/36 \text{ ksi} = 1.58 \text{ in.}^2$$

Choose a WT6×7 with an area of 2.08 in.^{2} and a z distance of 4.2 inches.

Step 9

From Equation 5, calculate the distance x to locate the PNA:

$$x = 20.66/2 + (57 - 158)/(2 \times 0.35 \times 36) = 6.32 \text{ inches}$$

$t_f \leq x = 6.32 \leq d - t_f$, therefore the PNA is in the beam web as assumed.

Step 10

Calculate the actual nominal moment capacity from Equation 9:

$$M_n = C_c[d + h - a/2] + T_r z + t_w F_{yb}[2dx - x^2] + C$$

From Table 1, $C = -676$, and so:

$$M_n = 158[20.66 + 6.25 - 0.44/2] + 36 \times 2.06 \times 4.2 + 0.35 \times 36[2 \times 20.66 \times 6.32 - (6.32)^2] - 676$$

$$M_n = (4217 + 315 + 2787 - 676)/12 = 554 \text{ ft-kips}$$

$$\phi M_n = 0.90 \times 554 = 471 \text{ ft-kips} > 466 \text{ ft-kips} \quad \textbf{o.k.}$$

Cut-Off Point

To determine the point at which the reinforcing WT6 can be terminated, the point on the moment diagram where the applied moment is equal to or less than the unreinforced beam moment capacity needs to be found. By simple statics, the location where the applied moment equals the unreinforced beam moment capacity (366 ft-kips) is 17.6 feet from the support. See Figure 2. Therefore, theoretically, the WT6 needs to be (20 ft. – 17.6 ft.) × 2 = 4.8 feet long and centered in the 40-ft. span. The actual length will be a little longer than this, however, to account for construction tolerances and development length, as is shown below.

Welds

First, the welds needed to develop each end of the WT section in tension will be considered. To keep material preparation simple, fillet welds will be used. Keeping in mind that the minimum size fillet weld for this WT section (stem thickness = 0.22 inches) is ⅛-in., and assuming $F_w = 60$ ksi (the nominal strength of the weld metal), the following can be written:

$$\text{Length of Weld} = T_r / [2 \times \phi \times 0.60 \times F_w \times A_w]$$

where

A_w = Effective area of the weld

$$\text{Length of Weld} = 36 \times 2.02/[2 \times 0.75 \times 0.60 \times 60 \times 0.707 \times 0.125] = 15.23 \text{ inches}$$

For practical purposes, a development length of 2 feet on each end will be used. Therefore, the length of the reinforcing WT6 will be 4.8 ft + 2 × 2 ft., or say 9 ft-0 in. long, centered in the 40 ft. span. ⅛-in. fillet welds 24-in. long between the WT stem and the bottom of the beam on both sides of the stem at each end will be used. Also, to stitch the stem to the bottom of the beam between the ends of the WT, ⅛-in. fillet welds 2-in. long at 12-in. o.c. both sides, staggered will be specified. This will result in a 2-in. weld every 6 inches, which will satisfy the minimum requirements. See Figure 3 for a design sketch.

Beam End Connections

The adequacy of the existing beam end connection for the increased end shear must now be investigated. The new

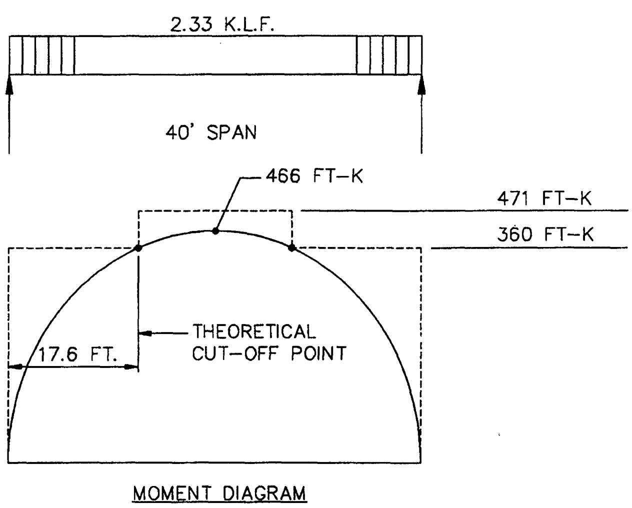

Fig. 2. Reinforcing cut-off point.

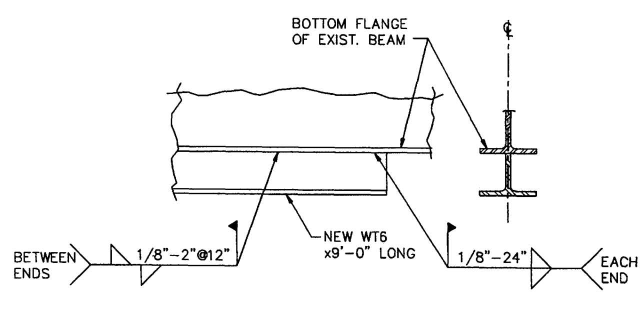

Fig. 3. Design sketch.

factored beam end reaction is 20 ft. × 2.33 klf = 46.6 kips. This is 11.2 kips greater than the maximum factored existing reaction of 35.4 kips. If it is given that a standard double angle framed connection with 4¾-in. diameter A325-N bolts was used, from Table II-A in the LRFD *Manual*,[2] the capacity of the connection is 124 kips, which far exceeds the slightly larger new factored reaction. It can be seen that even a nominal framed connection usually has ample reserve capacity. However, other types of connections, such as single plates, may not have reserve capacity, especially for severely increased loadings. It is very important to investigate the existing connections through field measuring and, if available, original shop drawing review. If a connection needs to be strengthened, there are a variety of methods available to the designer depending on the actual conditions, and will not be treated here.

Deflection

As described earlier in this paper, for an approximate deflection check, a summation of the lower bound moment of inertia for the existing beam and the contribution to the moment of inertia of the reinforcing section will be used. From the "Lower Bound Elastic Moment of Inertia" table in the LRFD *Manual*[2] for a W21×44, and using point 7 and *Y2* equal to 6 inches, the lower bound moment of inertia is found to be 1540 in.^{4}.

Now an approximation of the contribution to the total moment of inertia of the reinforcing section must be made. If the statical moment of the reinforcing is taken about the PNA, this should be on the conservative side. Thus, from Figure 1, the area of the reinforcing times the squared distance of $z + d - x$ is:

$$I_r = A_r(z + d - x)^2$$

$$I_r = 2.06(4.2 + 20.66 - 6.32)^2 = 708 \text{ in.}^4$$

Thus the total moment of inertia can be estimated to be 1540 + 708 = 2248 in.^{4}, and the live load deflection of the beam is approximately:

$$\delta_{LL} = 5 \times 1.00 \times 40(40 \times 12)^3 / (384 \times 29{,}000 \times 2248) = 0.88 \text{ in.}$$

Which is equivalent to *L*/543 **o.k.**

If the deflection turns out to be greater than allowables, a more detailed analysis may be required.

CONCLUSION

A rapid manual procedure for determining the required amount of tensile reinforcement added to the bottom of a composite steel beam to increase its nominal moment capacity is derived based on LRFD theory. Flat steel plate or WT reinforcing sections are included in the analysis. Tabularized constants for a variety of steel beam sizes in Grade 36 and 50 are given for use in the hand calculations.

NOMENCLATURE

A = constant given in Table 1
A_{sr} = area of steel reinforcing, in.^{2}
a = $C_c / (0.85 f_c' b)$, inches
B = a constant given in Table 1
b = effective width of concrete slab, inches
C = a constant given in Table 1
C_c = $0.85 f_c' A_c$ or SUM Q_n (smaller), kips
C_{sf} = compressive force in the steel beam flange, kips
C_{sw} = compressive force in the steel beam web, kips
d = steel beam depth, inches
F_{yb} = specified yield stress of the beam, ksi
F_{yr} = specified yield stress of the reinforcing, ksi
f_c' = specified strength of concrete, ksi
h = distance from top of steel beam to top of concrete slab, inches
M_n = nominal moment capacity of the composite section, in-kips
M_u = applied factored moment, in-kips
Q_n = nominal shear strength of an individual shear connector, kips
t_f = thickness of the steel beam flanges, inches
t_w = thickness of the steel beam web, inches
T_r = tensile force in the reinforcing, kips
T_{sf} = tensile force in the steel beam flange, kips
T_{sw} = tensile force in the steel beam web, kips
x = distance from top of steel beam flange to the PNA, inches
z = estimated distance from the centroid of the bottom reinforcing to the bottom of the steel beam flange

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Table 1.
Constants *A*, *B*, and *C*

Shape	$F_y = 36$			$F_y = 50$		
	A	***B***	***C***	***A***	***B***	***C***
W36×232	31.3	1411	1027	43.5	722	1426
W36×210	29.9	1330	–479	41.5	657	–665
W36×194	27.5	1324	–197	38.3	659	–274
W36×182	26.1	1309	–277	36.3	650	–384
W36×170	24.5	1298	–251	34.0	644	–348
W36×160	23.4	1272	–589	32.5	623	–818
W36×150	22.5	1239	–1050	31.3	596	–1459
W36×135	21.6	1151	–2429	30.0	520	–3373
W33×169	24.1	1215	1712	33.5	643	2378
W33×152	22.9	1150	645	31.8	589	896
W33×141	21.8	1115	140	30.3	561	195
W33×130	20.9	1067	–587	29.0	520	–815
W33×118	19.8	1012	–1337	27.5	473	–1857
W30×148	23.4	997	1309	32.5	526	1818
W30×132	22.1	933	308	30.8	473	427
W30×124	21.1	917	136	29.3	462	189
W30×116	20.3	886	–299	28.2	436	–415
W30×108	19.6	847	–831	27.3	403	–1155
W30×99	18.7	810	–1288	26.0	3?1	–1788
W30×90	16.9	809	–1071	23.5	373	–1487
W27×161	23.8	960	4727	33.0	580	6565
W27×146	21.8	943	4218	30.3	569	5859
W27×129	22.0	831	1493	30.5	450	2074
W27×114	20.5	781	743	28.5	409	1032
W27×102	18.5	769	651	25.8	402	904
W27×94	17.6	740	277	24.5	378	385
W27×84	16.6	701	–201	23.0	345	–279
W24×117	19.8	737	2949	27.5	443	4095
W24×104	18.0	718	2500	25.0	428	3472
W24×103	19.8	655	1064	27.5	355	1477
W24×94	18.5	636	833	25.8	341	1157
W24×84	16.9	617	616	23.5	327	855
W24×76	15.8	593	333	22.0	307	462
W24×68	14.9	561	–39	20.8	279	–54
W24×62	15.5	492	–1112	21.5	210	–1544
W24×55	14.2	473	–1178	19.8	195	–1636
W21×111	19.8	618	3083	27.5	387	4282
W21×101	18.0	616	2875	25.0	388	3993
W21×93	20.9	494	551	29.0	260	765
W21×83	18.5	491	596	25.8	262	828
W21×73	16.4	487	587	22.8	261	815
W21×68	15.5	479	497	21.5	255	690
W21×62	14.4	466	364	20.0	246	506
W21×57	14.6	423	–296	20.3	202	–411
W21×50	13.7	393	–564	19.0	176	–783
W21×44	12.6	373	–676	17.5	160	–938
W18×106	21.2	476	2662	29.5	301	3697
W18×97	19.3	479	2560	26.8	306	3556
W18×86	17.3	469	2260	24.0	300	3139
W18×76	15.3	463	2016	21.3	298	2800
W18×71	17.8	377	637	24.8	206	885
W18×65	16.2	378	665	22.5	210	924
W18×60	14.9	376	649	20.8	210	901
W18×55	14.0	366	528	19.5	202	734
W18×50	12.8	362	484	17.8	200	672
W18×46	13.0	330	53	18.0	167	74
W18×40	11.3	325	55	15.8	165	77
W18×35	10.8	296	–185	15.0	140	–257

Table 1.
Constants *A*, *B*, and *C*

Shape	$F_y = 36$			$F_y = 50$		
	A	*B*	*C*	*A*	*B*	*C*
W16×77	16.4	395	2008	22.8	259	2789
W16×67	14.2	393	1794	19.8	260	2492
W16×57	15.5	310	617	21.5	175	857
W16×50	13.7	306	563	19.0	173	782
W16×45	12.4	301	502	17.3	171	698
W16×40	11.0	300	479	15.3	172	665
W16×36	10.6	276	262	14.8	151	364
W16×31	9.9	256	36	13.8	130	50
W16×26	9.0	234	–107	12.5	111	–149
W14×74	16.2	325	2012	22.5	225	2795
W14×68	14.9	321	1849	20.8	222	2568
W14×61	13.5	316	1656	18.8	219	2300
W14×53	13.3	279	1132	18.5	182	1573
W14×48	12.2	273	1010	17.0	178	1402
W14×43	11.0	269	902	15.3	176	1253
W14×38	11.2	245	518	15.5	146	719
W14×34	10.3	237	429	14.2	140	595
W14×30	9.7	220	274	13.5	124	380
W14×26	9.2	203	85	12.8	106	118
W14×22	8.3	188	–10	11.5	93	–15
W12×58	13.0	272	1605	18.0	198	2229
W12×53	12.4	257	1391	17.3	185	1932
W12×50	13.3	228	1062	18.5	154	1475
W12×45	12.1	225	956	16.8	152	1328
W12×40	10.6	225	876	14.8	154	1217
W12×35	10.8	208	560	15.0	130	778
W12×30	9.4	202	467	13.0	126	649
W12×26	8.3	197	396	11.5	122	550
W12×22	9.4	149	–24	13.0	73	–33
W12×19	8.5	140	–64	11.8	66	–90
W12×16	7.9	125	–148	11.0	53	–205
W12×14	7.2	120	–154	10.0	50	–214
W10×45	12.6	180	980	17.5	129	1361
W10×39	11.3	170	816	15.8	121	1134
W10×33	10.4	154	622	14.5	107	864
W10×30	10.8	148	416	15.0	93	577
W10×26	9.4	146	364	13.0	92	505
W10×22	8.6	133	254	12.0	81	352
W10×19	9.0	111	56	12.5	59	78
W10×17	8.6	102	–4	12.0	51	–5
W10×15	8.3	93	–57	11.5	43	–79
W10×12	6.8	89	–58	9.5	40	–80