CHAPTER 6 SEISMIC DESIGN OF A 16-BOLT EXTENDED STIFFENED MOMENT END-PLATE CONNECTION

6.1 INTRODUCTION

This chapter presents some of the findings of an ongoing study to develop design procedures for large moment end-plate connections to be used in seismic areas. In particular, a large connection, shown in Fig. 6-1, is considered with special emphasis on design requirements stated in the *AISC Seismic Provisions for Structural Steel Buildings* (1997). The connection is tagged 16ES moment end-plate connection to designate the 16-bolt extended stiffened configuration. The purpose of the connection is to allow the engineer a moment end-plate configuration that can be designed stronger than the adjoining beam. The design procedure for the largest configuration currently available, the eight-bolt extended stiffened end-plate, is unable to develop numerous practical beam sections. The 16ES moment end-plate connection allows for the design of an economical moment connection that will meet the requirements for ordinary moment frames as stated in the *AISC Seismic Provisions for Structural Steel Buildings*, while avoiding any type of experimental testing requirement. A simplified design procedure that considers the limit states of end-plate yielding and bolt rupture is presented, based on results obtained using yield line analysis and the finite element method, respectively. The procedure is then validated for two beams (W21x101 and W27x146) using the finite element method.

The ANSYS finite element package is used to model the beam-to-column connection. Since the problem is three-dimensional in nature, solid eight-node and twenty-node brick elements that include plasticity effects are used to model the members and connecting elements, respectively. Contact elements are included between the end-plate and the column flange to represent the nonlinear behavior of this complex interaction problem. Bolt pretensioning effects are included and prying forces are tracked throughout the loading process. In conclusion, the effectiveness of the 16ES end-plate connection is discussed and seismic applications are provided.

6.2 YIELD LINE ANALYSIS

Borgsmiller (1995) summarizes the controlling yield line mechanisms for the most commonly used end-plate configurations. All the patterns are developed for gravity and wind loading. Such connections usually only have two bolts at the compression flange and it is assumed that the internal work done in this region is negligible. However, for seismic design, the connection should be able to resist a complete reversal of loading and the tension and compression flange bolt configurations are designed symmetrically. Given the large number of bolts at each flange for this end-plate configuration, it is shown that the internal work should be considered at both flanges for the 16ES connection.

6.3 PROCEDURE

The controlling yield line pattern for the 16ES moment end-plate connection is shown in Fig. 6-2. It was found by considering the components of the cases considered in Borgsmiller (1995), and other feasible patterns. The patterns (inside the top flange) shown in Fig. 6-2 can also be used for the eight-bolt extended stiffened end-plate and some multi-row end-plates used in metal building design. It has been determined by the author that this pattern results in end-plate strengths three to ten percent less than those presented in Borgsmiller (1995). Here, it is assumed that the part of the end-plate below the bottom flange (about which rotation takes place) remains flat and any internal work there is negligible. However, note that yield lines above the bottom flange are considered. The letters A through O as shown in the figure correspond to the beam web centerline locations where the deflections δ_i from i = A through O represent the linear variation in deflection from the bottom of the beam to the top of the end-plate. Using virtual work, and rotating the bottom flange a value of $\theta = 1/(d+p_{ext})$ so that the top of the end-plate deflects one unit, the strength of the plate can be obtained. Including the internal work done by all the yield lines of Fig. 6-2, the end-plate bending strength is found as

$$\begin{aligned}
M_n &= M_p((b_f + 2b_{ext})(1 + \frac{d + p_{ext}}{p_{ext} - p_f - p_b} + \frac{1}{p_f s_1 s_5}(2(p_b p_f (s_1 - s_5) \\
&+ p_f^2 (s_1 - s_5) + 2ds_1 s_5 + p_f (s_5 (d - t_f) + s_1 t_f)))) \\
&+ 6g_1 + \frac{8}{g_1 - t_w}(dp_b + p_b^2 + dp_{ext} + p_b p_{ext} + dp_f - p_b p_f + p_f p_{ext} + ds_1 \\
&- p_b s_1 - p_f s_1 - p_b s_2 + s_2^2 - p_b s_3 + s_3^2 - p_b s_4 + s_4^2 + p_b s_5 + p_f s_5 - s_1 t_f + t_f s_5) \\
&- \frac{t_w}{p_f (p_b - p_{ext} + p_f) s_1 s_5}(2p_b^2 p_f s_1 - 2p_b p_{ext} p_f s_1 + 4p_b p_f^2 s_1 - 2p_{ext} p_f^2 s_1 \\
&+ 2p_f^3 s_1 + 2dp_b p_f s_5 - 2p_b^2 p_f s_5 - 2dp_{ext} p_f s_5 + 2p_b p_{ext} p_f s_5 + 2dp_f^2 s_5 \\
&- 4p_b p_f^2 s_5 + 2p_{ext} p_f^2 s_5 - 2p_f^3 s_5 + 4dp_b s_1 s_5 - 4dp_{ext} s_1 s_5 + 3dp_f s_1 s_5 \\
&+ 7p_b p_f s_1 s_5 - 8p_{ext} p_f s_1 s_5 + 7p_f^2 s_1 s_5 + 2p_f t_f (p_b - p_{ext} + p_f)(s_1 - s_5)))
\end{aligned} \qquad (6-1)$$

where $M_p = F_{yp}t_p^2/4$ is the plastic moment capacity of the yield line per unit length, and the unknown lengths s_1, s_2, s_3, s_4, and s_5 are obtained by minimizing the internal work and are found to be

$$s_1 = s_5 = \frac{1}{2}\sqrt{(2b_{ext} + b_f - t_w)(g_1 - t_w)}$$

$$s_2 = s_3 = s_4 = \frac{p_b}{2} \qquad (6-2)$$

Neglecting the thickness of the web, the solution simplifies to

$$\begin{aligned} M_n = {} & M_p((b_f + 2b_{ext})(1 + \frac{d + p_{ext}}{p_{ext} - p_f - p_b} + \frac{1}{p_f s_1 s_5}(2(p_b p_f (s_1 - s_5) \\ & + p_f^{\,2}(s_1 - s_5) + 2ds_1 s_5 + p_f(s_5(d - t_f) + s_1 t_f)))) \\ & + 6g_1 + \frac{8}{g_1}(dp_b + p_b^{\,2} + dp_{ext} + p_b p_{ext} + dp_f - p_b p_f + p_f p_{ext} + ds_1 \\ & - p_b s_1 - p_f s_1 - p_b s_2 + s_2^{\,2} - p_b s_3 + s_3^{\,2} - p_b s_4 + s_4^{\,2} + p_b s_5 + p_f s_5 - s_1 t_f + t_f s_5)) \end{aligned} \qquad (6-3)$$

and

$$s_1 = s_5 = \frac{1}{2}\sqrt{(2b_{ext} + b_f)(g_1)}$$

$$s_2 = s_3 = s_4 = \frac{p_b}{2} \qquad (6-4)$$

Equation (6-1) can be further simplified if it can be shown that the internal work done by plate deformations in the proximity of the bottom flange of the beam is relatively small. Hence, neglecting these yield lines, Eqs. (6-1) and (6-2) become

$$
\begin{aligned}
M_n &= 2M_p(b_{ext}\left(-1+\frac{d+p_{ext}}{p_{ext}-p_f-p_b}+\frac{2(-p_f^{\ 2}+s_1(2d-t_f)+p_f(d-p_b-t_f))}{p_f s_1}\right) \\
&+\frac{1}{2}(4g_1+\frac{1}{p_f(p_b-p_{ext}+p_f)s_1}(b_f(2dp_b p_f-2p_b^{\ 2}p_f-2dp_{ext}p_f+2p_b p_{ext}p_f \\
&+2dp_f^{\ 2}-4p_b p_f^{\ 2}+2p_{ext}p_f^{\ 2}-2p_f^{\ 3}+4dp_b s_1-4dp_{ext}s_1+3dp_f s_1-p_b p_f s_1 \\
&-p_f^{\ 2}s_1-2t_f(p_b-p_{ext}+p_f)(s_1+p_f))) \\
&+\frac{8}{g_1-t_w}(dp_{ext}+dp_f+p_f p_{ext}-p_f^{\ 2}+ds_1-p_f s_1+s_2^{\ 2}+s_3^{\ 2}+dp_b+p_b p_{ext} \\
&-2p_b p_f-p_b s_1-p_b s_2-p_b s_3-t_f p_b-t_f p_f-s_1 t_f) \\
&+\frac{t_w}{p_f(p_b-p_{ext}+p_f)s_1}(2p_b^{\ 2}p_f+2dp_{ext}p_f-2dp_f^{\ 2}-2p_{ext}p_f^{\ 2}+2p_f^{\ 3}+4dp_{ext}s_1 \\
&-3dp_f s_1+4p_{ext}p_f s_1-3p_f^{\ 2}s_1-2t_f(p_{ext}-p_f)(p_f+s_1)+p_b(-2dp_f-2p_{ext}p_f \\
&+4p_f^{\ 2}-4ds_1-3p_f s_1+2t_f(p_f+s_1))))
\end{aligned}
\qquad (6-5)
$$

and

$$
\begin{aligned}
s_1 &= \frac{1}{2}\sqrt{(2b_{ext}+b_f-t_w)(g_1-t_w)} \\
s_2 &= s_3 = \frac{p_b}{2}
\end{aligned}
\qquad (6-6)
$$

respectively.

Finally, neglecting the thickness of the web, Eqs. (6-5) and (6-6) simplify to

$$
\begin{aligned}
M_n &= 2M_p(b_{ext}\left(-1+\frac{d+p_{ext}}{p_{ext}-p_f-p_b}+\frac{2(-p_f^{\ 2}+s_1(2d-t_f)+p_f(d-p_b-t_f))}{p_f s_1}\right) \\
&+\frac{1}{2}(4g_1+\frac{1}{p_f(p_b-p_{ext}+p_f)s_1}(b_f(2dp_b p_f-2p_b^{\ 2}p_f-2dp_{ext}p_f+2p_b p_{ext}p_f \\
&+2dp_f^{\ 2}-4p_b p_f^{\ 2}+2p_{ext}p_f^{\ 2}-2p_f^{\ 3}+4dp_b s_1-4dp_{ext}s_1+3dp_f s_1-p_b p_f s_1 \\
&-p_f^{\ 2}s_1-2t_f(p_b-p_{ext}+p_f)(s_1+p_f))) \\
&+\frac{8}{g_1}(dp_{ext}+dp_f+p_f p_{ext}-p_f^{\ 2}+ds_1-p_f s_1+s_2^{\ 2}+s_3^{\ 2}+dp_b+p_b p_{ext} \\
&-2p_b p_f-p_b s_1-p_b s_2-p_b s_3-t_f p_b-t_f p_f-s_1 t_f))
\end{aligned}
\qquad (6-7)
$$

and

$$s_1 = s_5 = \frac{1}{2}\sqrt{(2b_{ext} + b_f)(g_1)}$$
$$s_2 = s_3 = s_4 = \frac{p_b}{2} \qquad (6-8)$$

respectively. The results of all four solutions is compared in the following section of this chapter.

For moment end-plate connections, the design strength for the limit state of bolt rupture is quite difficult to determine. Prying forces must be included and as shown in Chapter 5, they can change significantly as a function of the bolt geometry. Nevertheless, the problem is simplified here. The thick end-plate design minimizes prying forces, and the width of the beam flange only allows for certain geometric configurations regarding bolt layout. Using fully tensioned high strength bolts and minimum bolt pitch to beam flange values of p_f=d_b+1/2 in. to minimize prying forces, the author has found that the following simplified relationship can be used to determine the number of bolts that are effective in resisting the design flange force:

$$n_{eff} = \begin{cases} 7 & \text{for} \quad 1/4'' \le (b_f - (g_1 + 2g_2))/2 \le 1/2'' \\ 8 & \text{for} \quad 1/2'' \le (b_f - (g_1 + 2g_2))/2 \le 3/4'' \\ 9 & \text{for} \quad 3/4'' \le (b_f - (g_1 + 2g_2))/2 \le 1.25'' \\ 9.5 & \text{for} \quad (b_f - (g_1 + 2g_2))/2 \ge 1.25'' \quad (d_b > 1'') \\ 10.5 & \text{for} \quad (b_f - (g_1 + 2g_2))/2 \ge 1.25'' \quad (d_b \le 1'') \end{cases} \qquad (6-9)$$

The relationship was determined as the lower bound of the ultimate strengths of various 16ES end-plate connections designed to develop the adjoining beam. In particular, W30x173, W24x162, and W21x101 connections were designed with different bolt sizes and analyzed to determine the ultimate capacity of the connection via bolt rupture. Using the capacity of the connection to determine the applied flange force, the number of bolts that are effective in resisting the design flange force was obtained (see Appendix B for more information). For the case when the distance from the centerline of the outside column of bolts to the beam flange tip is greater than 1.25 in., the number of effective bolts has been shown to be dependent on the bolt size. Apparently, smaller diameter bolts have better stress redistribution characteristics in the inelastic range of the bolt stress-strain curves. This must be related to the smaller bolt heads on the smaller diameter bolts. Given that 16 bolts are at the tension flange, the values 7 through 10.5 bolts seem like a relatively small number of effective bolts. However, it will soon be shown that the four bolts farthest from the intersection between the beam flange and the beam web or stiffener are not effective at all, regardless of the configuration. In other words, it is really 7 to 10.5 out of 12 bolts that are effective.

6.4 SAMPLE APPLICATIONS

To show the simplicity and effectiveness of the design procedure discussed in the previous section, connections will be designed to develop a W21x101 and a W27x146 beam section to be part of an ordinary moment frame. This connection was chosen to represent a worst case lower bound design for a specific bolt spacing. For this purpose, the actual bolt tensile strength will be used in lieu of the design tensile strength. This will show the effectiveness of the design procedure. However, to account for strength variability in design using LRFD, an appropriate resistance factor must be used. Current SAC discussions suggest that the normal resistance factor (ϕ=0.75) for bolts in tension may be too conservative when combined with other load variability considerations. Also, stress redistribution between bolts increases the ability to more accurately predict the connection's strength.

Tables 6-1 and 6-2 provide the details of the specimens considered here and shown in Fig. 6-1. The values listed are based on minimum bolt spacing and tightening clearances. The symbols F_{yp}, F_{ys}, and F_{yb} represent the nominal yield stress of the end-plate, stiffener, and beam, respectively. Note that the tension flange (top flange) bolts are numbered in Fig. 6-1. Individual bolts will be referred to by these numbers in forthcoming sections of this chapter.

According to the *AISC Seismic Provisions for Structural Steel Buildings* (1997), to avoid any type of experimental testing requirement, the connections must be designed to resist $1.1R_yM_p$ or the maximum moment that can be delivered to the system, where R_y is the ratio of expected yield stress to nominal yield stress and M_p is the plastic moment capacity of the beam in question. This requirement is specified to ensure that beam hinging occurs during an earthquake.

For the W21x101 beam, the ultimate moment, M_u, is found as

$$M_u = 1.1 R_y M_p = 1.1 \times 1.1 \times 1054 = 1276 \quad k-ft \qquad (6-10)$$

The end-plate bending strength must be greater than M_u. For the purpose of illustrating the effectiveness of the method, the nominal design strength is used here (i.e., ϕ=0.90 for end-plate bending is not used here). Using the data from Table 6-1 and Eqs. 6-1, 6-3, 6-5, and 6-7, a 1.00 in. thick end-plate results in the following strengths: 1413 k-ft; 1392 k-ft; 1277 k-ft; and 1259 k-ft. The full solution, 1413 k-ft, which includes all yield lines forming the mechanism, is the most complete solution. Conservatively neglecting yield lines at the compression flange (Eq. 6-5) results in a 9.6 percent decrease in strength. On the other hand, neglecting the thickness of the beam web (Eq. 6-3) only reduces the strength 1.5 percent. Neglecting the yield lines at the compression flange and thickness of the beam web (Eq. 6-7) decreases the strength 10.9%. It is recommended by the author, and is good seismic engineering practice, to increase the thickness of the end-plate by, say, 10 percent to ensure that no inelastic prying forces develop in the bolts prior to beam hinging. Although a mechanism will not form in the end-plate prior to reaching the end-plate bending limit state, significant yielding of the end-plate will occur and this could cause some unwanted prying action. In lieu of using the complex Eq. 6-1 and increasing the end-plate thickness, Eq. 6-3 can be used conservatively for the design. Since 1277 k-ft is greater than 1276 k-ft, a 1.00 inch thick end-plate will be used. Also, a 7/8 in. end-plate will be considered to show how the connection may be inadequate for developing the beam if the end-plate is not sufficiently thick. The end-plates are tagged “thick” and “thin” plates based on their relative thickness.

The flange force, F_f, is now determined to be:

$$F_f = \frac{M_u}{d - t_f} = \frac{1276 \times 12}{21.36 - 0.8} = 745 \quad k \qquad (6-11)$$

Equation 6-9 is then used to determine the number of effective bolts:

$$(b_f - (g_1 + 2g_2))/2 = (12.29 - (5 + 2 \times 3.33))/2 = 0.32 \quad \text{in.} \tag{6-12}$$

$$n_{eff} = 7$$

Since 0.32 in. is greater than 0.25 in., but less than 0.5 in., seven bolts are effective.

Dividing the flange force by seven results in 106.43 k per bolt. From Table 8-15 of the *AISC Manual of Steel Construction, Vol. II,* 1 1/4 in. A325 bolts are required. These bolts have actual tensile strengths of 82.8 / 0.75 = 110.4 k per bolt (note that a resistance factor of 0.75 is used in this table). Again, it should be pointed out that the design bolt tensile strength of 82.8 k should not be divided by the resistance factor in actual design. Once the proper resistance factor has been established by SAC, the actual bolt tensile strength should be multiplied by this factor and not by ϕ=0.75.

For the W27x146 beam, the ultimate moment, M_u, is found as

$$M_u = 1.1 R_y M_p = 1.1 \times (1.1) \times 1921 = 2323 \quad \text{k} - \text{ft} \tag{6-13}$$

The end-plate bending strength must be greater than M_u. Using the data from Table 6-2 and Eqs. 6-1, 6-3, 6-5, and 6-7, a 1 3/16 in. thick end-plate results in the following strengths: 2547 k-ft; 2504 k-ft; 2332 k-ft; and 2295 k-ft. Neglecting yield lines at the compression flange (Eq. 6-5) results in an 8.4 percent decrease in strength. On the other hand, neglecting the thickness of the beam web (Eq. 6-3) only reduces the strength 1.7 percent. Neglecting the yield lines at the compression flange and thickness of the beam web (Eq. 6-7) decreases the strength 9.9%. A 1 3/16 in. thick end-plate will be used here. Also, a 1 1/16 in. end-plate will be considered to show how the connection may be inadequate for developing the beam if the end-plate is not sufficiently thick. The end-plates are tagged “thick” and “thin” plates based on their relative thickness.

The flange force, F_f, should now be determined:

$$F_f = \frac{M_u}{d - t_f} = \frac{2323 \times 12}{27.38 - 0.98} = 1056 \quad \text{k} \tag{6-14}$$

Eq. 6-9 is then used to determine the number of effective bolts:

$$(b_f - (g_1 + 2g_2))/2 = (13.97 - (5.5 + 2 \times 3.66))/2 = 0.58 \text{ in.} \qquad (6-15)$$

$$n_{eff} = 8$$

Since 0.58 in. is greater 0.5 in. but less than 0.75 in., 8 bolts are effective.

Dividing the flange force by 8 results in 132 k per bolt. From Table 8-15 of the *AISC Manual of Steel Construction, Vol. II,* 1 3/8 in. A325 bolts are required. These bolts have a design tensile strength of 100 / 0.75 =133.3 k per bolt.

6.5 FINITE ELEMENT RESULTS

Using the finite element method and loading the beam at the beam tip with increasing loads, it was found that the 16ES moment end-plate connections designed in the previous section could successfully develop large beams. Figure 6-3 shows the formation of a plastic hinge in the W21x101 beam. Note that stresses in excess of the beam yield stress of 50 ksi occur just past the stiffener. Similar results have been obtained in the laboratory for extended stiffened connections. See, for example, Sumner et al. (2000). Figure 6-4 shows the von Mises stress distribution at the tension flange. It is interesting to note the smooth load path that the beam flange and stiffener provide to the bolts. However, there is very little stress around bolt #3 and bolt #8, indicating that there is very little load distribution to these bolts. This is expected, as no source of stiffness (i.e., beam flange, beam web, or stiffener) is provided to these bolts. Figure 6-5 shows the von Mises stress distribution across the thin end-plate. Likewise, Fig. 6-6 shows the von Mises stress distribution across the thick end-plate. The patterns are quite similar but the plots are at different applied moments. The maximum applied moments were 1300 and 1350 k-ft for the thin and thick end-plate connections, respectively. Although both the thin and thick end-plate are able to develop the beam, the thin end-plate connection fails via bolt rupture prior to large inelastic deformations and at a lower maximum applied moment.

Figure 6-7 shows the von Mises stress distribution at the tension flange of the W27x146 beam and the formation of a plastic hinge just outside of the stiffener. Since the flange is wider for this beam, an even smoother load path is observed here. Figures 6-8 and 6-9 plot the von Mises stress distribution across the thin and thick end-plate at the connection's maximum moment, respectively. The maximum moments were 2110 and 2321 k-ft for the thin and thick end-plate connections, respectively. In this case, only the thick end-plate was able to develop the beam. Hence, once again, the plots are at different applied maximum moments.

Figure 6-10 plots the applied moment vs. maximum plate separation for the W21x101 beam. As expected, for the same applied moment, the thin end-plate deflects more than the thick one. Figure 6-11 plots the applied moment vs. inelastic rotation for the W21x101 beam. It is apparent that the connection response is similar for the thick and thin plates. However, note that the thin plate solution diverges at around 0.015 radians of inelastic rotation, far less than the value 0.035 of its thick plate counterpart. Figures 6-12 and 6-13 plot the bolt stress in all the bolts vs. applied moment for the thin and thick end-plate, respectively. Since the end-plate thicknesses are so close, it is difficult to see much difference. However, note that the critical bolts reach their maximum values at an earlier applied moment for the thin end-plate. It is interesting that bolt #3 and bolt #8 actually lose load as a result of their location.

Figure 6-14 plots the applied moment vs. plate separation for the W27x146 16ES connection. It is clear that the thin plate deflects more for the same applied moment, but reaches a significantly lower maximum moment. Figure 6-15 shows that the beam does not form a hinge for the W27x146 thin plate connection. Bolt rupture occurs prior to the hinge formation. Finally, Figs. 6-16 and 6-17 plot the stresses in the bolts vs. applied moment. Once again, it is clear that little redistribution of bolt forces is possible for the thin plate connection.

6.6 CONCLUSION

It has been shown that the 16ES moment end-plate connection can be used to develop large beams to be part of seismic lateral force resisting systems. The design procedure is simplified by using a thick end-plate and 16 bolts at each flange. As expected, a properly detailed connection allows beam hinging to provide the 0.01 radians of inelastic rotation required for ordinary moment frames. In fact, for the thick end-plate case (W21x101) considered here, 0.03 radians of inelastic rotation, as required for special moment frames, is surpassed. The reason for the lack of any experimental testing requirement is readily apparent for this design approach. For ordinary moment frames, as long as the design engineer can ensure that the connection is stronger than the adjoining beam, beam hinging can provide enough ductility to the system. It has also been shown that bolts #3 and #8 are ineffective at resisting the flange force delivered to the connection. Hence, these bolts may be left out of the design without any loss in strength. Although not presented in this chapter, the two designed connections of this chapter were analyzed without bolts #3 and #8 and found to have almost identical ultimate strengths as the complete connections. Even as a source of redundancy, these bolts are questionable.

TABLE 6-1. Connection details for W21x101 16ES connection.

Dimension	Value
d	21.36 in.
b_f	12.29 in.
t_f	0.80 in.
t_w	0.50 in.
g_1	5.00 in.
g_2	3.33 in.
b_{ext}	2.00 in.
p_{ext}	7.00 in.
p_f	1.75 in.
p_b	3.33 in.
t_p (plate thickness)	1.00 in. (and 0.88”)
Bolt designation	1 1/4” A325
F_{yp} (plate yield stress)	36 ksi
F_{ys} (stiffener yield stress)	50 ksi
F_{yb} (beam yield stress)	50 ksi
M_u (ultimate moment)	1276 k-ft

TABLE 6-2. Connection details for W27x146 16ES connection.

Dimension	Value
d	27.38 in.
b_f	13.97 in.
t_f	0.98 in.
t_w	0.61 in.
g_1	5.5 in.
g_2	3.67 in.
b_{ext}	2.00 in.
p_{ext}	7.75 in.
p_f	1.88 in.
p_b	3.67 in.
t_p (plate thickness)	1.19 in. (and 1.06”)
Bolt designation	1 3/8” A325
F_{yp} (plate yield stress)	36 ksi
F_{ys} (stiffener yield stress)	50 ksi
F_{yb} (beam yield stress)	50 ksi
M_u (ultimate moment)	2323 k-ft