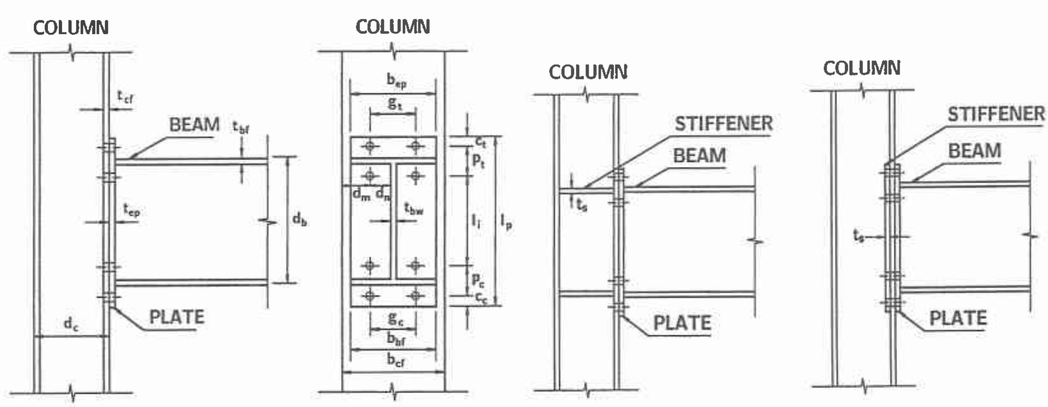
I −66 EXTENDED END-PLATE CONNECTIONS TO SUSTAIN HIGH MOMENT

R. Hasan — M. JSCE, Muroran Institute of Tech.
N. Kishi — M. JSCE, Muroran Institute of Tech.
Y. Goto — M. JSCE, Nagoya Institute of Tech.
M. Komuro — M. JSCE, Muroran Institute of Tech.

1. INTRODUCTION

Rigid frame construction is commonly seen as a viable solution, particularly, for high-rise buildings in earthquake prone areas. Steel beam-to-column connections in a rigid frame are assumed to be sufficiently rigid so that the original angles between intersecting members remain unchanged. In the version of moment-rotation ($M - \theta_r$) relation, a fully rigid connection can be described as the connection which obey $\theta_r = 0$ equation. But this is merely a theoretical phenomenon. No practical connections can hold this true. Therefore, for the purpose of practicality, a designer needs to design a connection which at least can replicate some proximate moment-rotation behavior to that of the fully rigid connection. In this perspective, extended end-plate connections are very often used as rigid connections. But a recent study (Hasan, 1995b) shows that all extended end-plate connections, in general, may not serve the purpose of rigid connections. Therefore, if a designer intends to design a rigid frame with the extended end-plate connections as beam-to-column connections, he needs to be assured that the connections he is designing possess sufficient rigidity and strength. That is, we need some criteria which can be used as measuring yardsticks in the design process through which the designer can readily check that his connection serves the purpose. The objective of this paper is to establish of such criteria and provide a design guideline so that extended end-plate connections can be unequivocally used as rigid connections.



(a) unstiffened　(b) transversely stiffened　(c) longitudinally stiffened

Fig. 1 General configurations of extended end-plate connections

2. EXTENDED END-PLATE CONNECTION AND ITS VARIATIONS

The extended end-plate connections consist of a steel plate profile welded to the beam end, bolted to the column flange and extended beyond the beam flange(s) as shown in Fig. 1. The end-plates are usually extended in two ways: (1) end-plates extended beyond the tension beam flange only and (2) end-plates extended beyond both the tension and compression beam flanges in case of moment reversal anticipation. However, the presence of the extension of the end-plate on the compression side has a limited effect on the connection behavior and strength (R. Zandonini et al., 1988). Therefore, no special attention is paid on the mode of end-plate extension.

On the other hand, column flange stiffening has a significant influence on the stiffness and strength of the connection. Based on the presence of column flange stiffener, the extended end-plate connections can be classified into two types: (1) unstiffened connections (Fig. 1a) and (2) stiffened connections (Figs 1b and c). Column flange stiffening is achieved by two means: (a) providing transverse stiffener (Fig. 1b) or (b) providing longitudinal stiffener (Fig. 1c). This study covers these three types of connections and considers a four-bolt cluster equally spaced above and below the beam flange(s).

3. CRITERIA ENSURING RIGID CONNECTION CONDITION

The behavior of a connection can mostly be characterized in terms of stiffness and strength. Therefore, there are two essential elements which can be used to define the minimum requirements of a rigid connection, viz., (i) initial connection stiffness R_{ki} and (ii) ultimate moment capacity M_u of a connection. In the following sub-sections, we will show how we can establish these two criteria in explicit terms for the purpose of rigid connection requirements.

3.1. Ultimate Moment Capacity M_u

The maximum moment capacity to let a connection attain should be determined from the capacity of the beam and connection assemblage itself. Since, an extended end-plate connection is intended to be used in rigid frame constructions, chances are there that the capacity of the connection assemblage may be targeted to exceed beam capacity. And experimental evidences in steel beam-to-column data bases (Kishi et al., 1986, Hasan et al. 1995b) indeed show that the ultimate moment capacity of extended end-plate connections surpassed the plastic moments of the beams M_p in many occasions. But a connection with over-capacity (i.e., $M_u > M_p$) will simply ensure beam failure prior to connection failure but of no use. Therefore, it is for the efficient design purpose that the ultimate moment capacity M_u of a connection should be aimed to attain at best to the level of plastic moment of the connecting beam M_p.

3.2. Initial Connection Stiffness R_{ki}

An extensive frame analysis was conducted by Hasan et al. (1995b) using a total of 112 experimental tests data and was found that about 60% of the practical connections having initial connection stiffness less than 10^6 kip-inch/radian fail to serve the purpose of rigid connections. On the other hand, most of the connections whose initial connection stiffnesses are greater than 10^6 kip-inch/radian responded like rigid connections in frame. Therefore, there were sufficient reasons to conclude that the minimum initial connection stiffness of a rigid connection should be taken as 10^6 kip-inch/radian.

4. ANALYTICAL SOLUTIONS

4.1. Ultimate Moment Capacity M_u

The moment applied to a beam-to-column connection can be calculated from the idealization that an internal couple consisting of two flange forces F_u with a moment arm of $(d_b - t_{bf})$ equals the external moment M_u (Fig. 2), where d_b is beam depth and t_{bf} is beam flange thickness. The flange force playing the main role in connection failure is actually dependent on the mode of failure. The common failure modes, as identified by Hasan et al. (1995a) can be listed as:

(1) Mechanism A: connection fails due to bolt failure,
(2) Mechanism B: connection fails due to end-plate failure,
(3) Mechanism C: connection fails due to column flange failure.

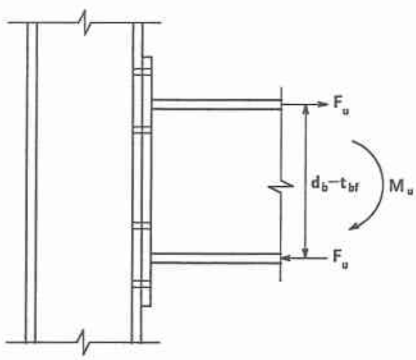


Fig. 2. Relation between internal couple and ultimate moment capacity

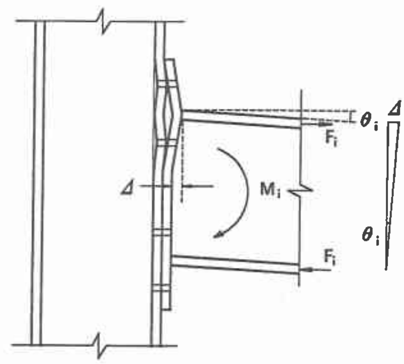


Fig. 3. Relation between connection deformation and initial stiffness

The minimum flange force among these failure modes will obviously be the governing one for the moment capacity determination. Thus, the ultimate moment capacity will be,

$$M_u = F_u(\min.)\times(d_b - t_{bf}) \tag{1}$$

Table 1 Flange force F_u for failure mechanisms A, B and C

Failure Mechanism		Expressions for flange force	Reference
Mechanism A		$F_u = 4A_{bo}\sigma_{ybo} - (b_{ep}t_{ep}^2\sigma_{yep} / 2.4c_t)$	
Mechanism B		$F_u = \sigma_{yep}t_{ep}^2\left[\frac{2b_{ep}}{(p_t - t_{bf})} + \frac{1.2(d_b - t_{bf})}{(g_t - t_{bw})}\right]$	Surtees J.O. et al. (1970)
Mechanism C	unstiffened column flange	$F_u = \sigma_{ycf}t_{cf}^2\left\{\pi + \frac{2d_n + p_t - d_{ho}}{d_m}\right\}$	Packer, J.A. et al. (1977)
	column flange stiffened with transverse stiffener	$F_u = \sigma_{ycf}t_{cf}^2\left\{\left(\frac{2}{p_t - t_s} + \frac{1}{w}\right)(2d_m + 2d_n - d_{ho})\right\} + \sigma_{ycf}t_{cf}^2\left\{\frac{p_t - t_s + 2w - d_{ho}}{d_m}\right\}$, $w = \sqrt{d_m(d_m + d_n - 0.5d_{ho})}$	Packer, J.A. et al. (1977)
	column flange stiffened with longitudinal stiffener	$F_u = \sigma_{ycf}t_{cf}^2\left\{\pi + \frac{p_t + 2d_n - d_{ho}}{d_m}\right\} + \sigma_{ys}t_s^2\left\{\frac{p_t + 2d_n - 4d_{ho}}{2d_m} + 2\right\}$	Moore, D.B. et al. (1986)

So far, these three mechanisms had intrigued many researchers and many solutions were presented by them. Some selective solutions are presented in Table 1, where σ is yield stress of the steel material and the suffices 'bo', 'ep', 'cf' and 's' denote for bolt, end-plate, column flange and stiffener, respectively. Besides these, A_{bo} is bolt area, d_{ho} is bolt hole diameter and the rest of the parameters used in this table are shown in Fig. 1.

4.2. Initial Connection Stiffness R_{ki}

The analytical derivations of initial connection stiffness is shown in detail in Hasan et al., (1995a). The principle used in that study is that the initial stiffness, as shown in Fig. 3, can be related to initial deformation of connection elements as:

$$R_{ki} = \frac{M_i}{\theta_i} = \frac{F_i\,(d_b - t_{bf})}{\theta_i} = \frac{F_i\,(d_b - t_{bf})^2}{\Delta} = \frac{F_i\,(d_b - t_{bf})^2}{\Delta_{ep} + \Delta_{cf}} \tag{2}$$

in which M_i is initial moment, F_i is flange force, θ_i is initial rotation, Δ = total deformation occurred at the connection, Δ_{ep} = deformation due to end-plate flexure and Δ_{cf} = deformation due to column flange flexure. Therefore, the task of evaluating initial connection stiffness involves mostly with the determination of the two components of the above equation, viz., Δ_{ep} and Δ_{cf} and they were formulated as follows (Hasan et al., 1995a):

$$\Delta_{ep} = \frac{F_i\,Z_{ep}}{E}\left[\frac{1}{8} - \frac{q_s}{2}\left\{\frac{3}{4}\,\alpha_{ep} - \alpha_{ep}^{\,3}\right\}\right] \tag{3}$$

$$\Delta_{cf} = \frac{F_i\,Z_{cf}}{E}\left[\frac{1}{8} - \frac{q_s}{2}\left\{\frac{3}{4}\,\alpha_{cf} - \alpha_{cf}^{\,3}\right\}\right] \tag{4}$$

where

$$q_s = \frac{Z_{ep}(1.5\alpha_{ep} - 2\alpha_{ep}^3) + Z_{cf}(1.5\alpha_{cf} - 2\alpha_{cf}^3)}{Z_{ep}(6\alpha_{ep}^2 - 8\alpha_{ep}^3) + Z_{cf}(6\alpha_{cf}^2 - 8\alpha_{cf}^3) + \dfrac{K}{A_{bo}}} \tag{5}$$

The functions Z_{ep}, Z_{cf}, α_{ep}, α_{cf} and K, expressed in terms of different connection parameters, are listed in Table 2. All the parameters used in this table are described in Figs 1.

Table 2 Expression for Z_{ep}, Z_{cf}, α_{ep}, α_{cf} and K

Type	Z_{ep}	Z_{cf}	α_{ep}	α_{cf}	K
Stiffened	$Z_{ep} = \dfrac{2\,(2c_t+p_t)^3}{b_{ep}\,t_{ep}^{\,3}}$	$Z_{cf} = \dfrac{2\,(2c_t+p_t)^3}{b_{cf}\,t_{cf}^{\,3}}$	$\alpha_{ep} = \dfrac{c_t}{(p_t+2c_t)}$	$\alpha_{cf} = \dfrac{c_t}{(p_t+2c_t)}$	1.25 × grip
Unstiffened	$Z_{ep} = \dfrac{2\,(2c_t+p_t)^3}{b_{ep}\,t_{ep}^{\,3}}$	$Z_{cf} = \dfrac{2b_{cf}^{\,3}}{(2c_t + p_t)\,t_{cf}^{\,3}}$	$\alpha_{ep} = \dfrac{c_t}{(p_t+2c_t)}$	$\alpha_{cf} = \dfrac{b_{cf} - gt}{2b_{cf}}$	

For unstiffened and/or transversely stiffened connections:

$$\text{grip} = t_{ep} + t_{cf} \tag{6}$$

For longitudinally stiffened connections:

$$\text{grip} = t_{ep} + t_{cf} + t_s \tag{7}$$

in which t_s is the thickness of stiffener.

5. SUGGESTED DESIGN PROCEDURE

As discussed in the preceding section, from the design point of view, the maximum moment to let an extended end-plate connection attain should be equated to the plastic moment of the connecting beam M_p. Following design recommendations are made in eight routines based on the principle that the M_p of beam equals to the connection moment capacity M_u at the time of connection failure. Therefore Eq. (1) can be re-written as:

$$M_p = F_u \times (d_b - t_{bf}) \tag{8}$$

Of the routines, all but (7) use Eq. (8) and flange force equations of Table 1.

(1) Calculate a trial bolt dia d_{bo} assuming prying force equals to 1/3 rd of bolt force i.e., use the following equation:

$$d_{bo} = \sqrt{\frac{M_p}{2.10\ \sigma_{ybo}\,(d_b - t_{bf})}} \tag{9}$$

(2) Assume end-plate width b_{ep}, gauge distance g_t, pitch p_t, d_m and d_n.

(3) Calculate end-plate thickness t_{ep} from the following relation:

$$t_{ep} = \sqrt{\frac{M_p}{\lambda_{ep}\sigma_{yep}(d_b - t_{bf})}} \tag{10}$$

where λ_{ep} is a non-dimensional parameter corresponds to failure mechanism B and can be expressed as:

$$\lambda_{ep} = \left[\frac{2b_{ep}}{(p_t - t_{bf})} + \frac{1.2(d_b - t_{bf})}{(g_t - t_{bw})}\right] \tag{11}$$

(4) Calculate required column flange thickness t_{cf} using Eq. (10) replacing t_{ep} and λ_{ep} with t_{cf} and λ_{cf} (unstiff.), respectively.

$$\lambda_{cf}\,(\text{unstiff.}) = \left\{\pi + \frac{2d_n + p_t - d_{ho}}{d_m}\right\} \tag{12}$$

If this calculated t_{cf} proves inadequate comparing with available t_{cf}, consider to provide column flange stiffener either (i) transverse or (ii) longitudinal. Decide which type of stiffener would be used and then design accordingly. If transverse stiffener is chosen skip routine (6) and if longitudinal stiffener is preferred then skip routine (5).

(5) Transverse stiffener: Assume stiffener thickness t_s and calculate required column flange thickness t_{cf} using Eq. (10) replacing t_{ep} and λ_{ep} with t_{cf} and λ_{cf} (trans. stiff.), respectively.

$$\lambda_{cf}\,(\text{trans. stiff.}) = \left\{\left[\frac{2}{p_t - t_s} + \frac{1}{w}\right]\ (2d_m + 2d_n - d_{ho})\right\} \tag{13}$$

$$+ \left\{\frac{p_t - t_s + 2w - d_{ho}}{d_m}\right\} \tag{14}$$

where,

$$w = \sqrt{d_m\,(d_m + d_n - 0.5d_{ho})} \tag{15}$$

If the available column flange thickness still fails to meet requirement, then calibrate stiffener thickness t_s until t_{cf} (available) > t_{cf} (required).

(6) Longitudinal stiffener: Assume stiffener thickness t_s and then calculate required column flange thickness t_{cf} using the Eqs (16), (17) and (18).

$$t_{cf}\,(\text{long. stiff.}) = \sqrt{\frac{M_p}{\lambda_{cf}\,(\text{long. stiff.})\ \sigma_{ycf}\,(d_b - t_{bf})} - \sigma_{ys}\, t_s^2\, \lambda_s\,(\text{long. stiff.})} \tag{16}$$

$$\lambda_{cf}\,(\text{long. stiff.}) = \left\{\pi + \frac{p_t + 2d_n - d_{ho}}{d_m}\right\} \tag{17}$$

$$\lambda_s\,(\text{long. stiff.}) = 2\left\{\frac{p_t + 2d_n - 4d_{ho}}{2d_m} + 2\right\} \tag{18}$$

If the available column flange thickness still fails to meet requirement, then calibrate stiffener thickness t_s until t_{cf} (available) > t_{cf} (required).

(7) Calculate initial connection stiffness R_{ki} using Eqs (2) through (7) and check R_{ki} (calculated) > 10^6 kip-inch/radian. If not, repeat the routines (2) through (6), where necessary, by increasing end-plate thickness until requirement for initial connection stiffness is met.

(8) If every connection details (except d_{bo}) is finally decided, calculate exact d_{bo} using the following equation:

$$d_{bo} = \sqrt{\frac{M_p}{\pi\sigma_{ybo}(d_b - t_{bf})} + \frac{b_{ep}t_{ep}^2\sigma_{yep}}{7.539\sigma_{ybo}c_t}} \tag{19}$$

6. SUMMARY AND CONCLUSION

Extended end-plate connections are very often used to sustain high moment in order to serve the purpose of rigid connections. But, till now, no specific design criteria have been set so that the designed connection can guarantee the achievement of this goal. In this study, firstly, a set of criterion has been set from extensive statistical and numerical analyses. Secondly, a detail procedure to design the connection parameters have been shown.

7. REFERENCES

(1) Hasan, R., Kishi, N., Chen, W.F., and Matsuoka, K.G. (1995a), "$M - \theta_r$ relationship of end-plate connections," Proceedings of the 4th Pacific Structural Steel Conference, Singapore, 121-128.

(2) Hasan, R., Kishi, N., Chen, W.F. and Komuro, M. (1995b), "Evaluation of rigidity of extended end-plate connections by utilizing updated data base," Struc. Engrg. Report No. CE-STR-95-19, School of Civil Engineering, Purdue Univ., West Lafayette, IN.

(3) Kishi, N., and Chen, W.F. (1986), "Data base of steel beam-to-column connections," Structural Engineering Report No. CE-STR-86-26, School of Civil Engineering, Purdue University, West Lafayette, IN.

(4) Moore, D.B. and P.A.C. Sims (1986). "Preliminary investigations into the behavior of extended end-plate steel connections with backing plates," J. Construct. Steel Research, 6, 95-122.

(5) Packer, J. A. and L.J. Morris (1977). "A limit state design method for the tension region of bolted beam-column connections," The Struct. Engrg., 55(10), 446-458.

(6) Surtees, J.O., and A.P. Mann (1970). "End-plate connections in plastically designed structures," Conference on Joints in Structures, 1(5).

(7) Zandonini, R., and Zanon, P., (1988), "Experimental analysis of end plate connections," Connections in Steel Structures; Behavior, Strength and Design, Edited by Bjorhovde, R., Brozzetti, J., and Colson, A., Elsevier Applied Science Publishers, London, 41-51.