Directional Moment Connections—A Proposed Design Method for Unbraced Steel Frames

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PRESENT PRACTICE in both elastic and plastic design of unbraced frames often results in uneconomical structures. This is because both methods require fully rigid connections. As a result, columns must be designed to resist beam gravity moments and connections must be able to develop the full moment capacity of the girders. Unfortunately, columns are not efficient members when resisting moments and full moment connections are often expensive, especially if column stiffeners are required.

This paper proposes a simple new design procedure which eliminates the need for fully rigid connections in unbraced frames. Based on a more realistic analysis of the loading-unloading behavior of beam-to-column connections, the procedure results in more efficient connections and columns and, at the same time, permits the girders to be utilized to their maximum strength and stiffness. This method also rationalizes the long established elastic design method of "flexible wind connections" (the apparently paradoxical design method whereby the girders are designed as simple beams, the connections are designed for wind moment only, and no additional moments are assumed to be induced in either the connection or column by gravity loads on the girder.)[1]

Included in this paper is a rational method for determining the effective length of columns in unbraced frames when the connections are designed for a moment capacity less than that of the girder.

PLASTIC AND ELASTIC CONNECTIONS

Conventional structural theory assumes that after the plastic moment of a connection has been reached the connection loses all stiffness—hence the term "plastic hinge." However, this is true only if the loading application tends to rotate the connection in the same direction as the moment which initially produced the plastic hinge. If the loading rotates the connection in the opposite direction, the connection acts elastically. This phenomenon is well established[2, 3] and can be seen by examining a typical moment-rotation curve of a steel moment connection (Fig. 1).

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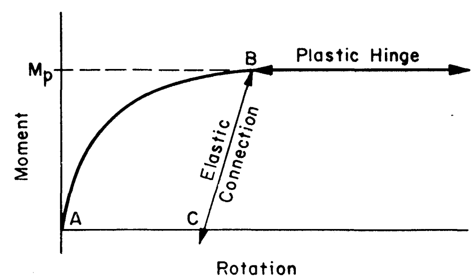

Figure 1

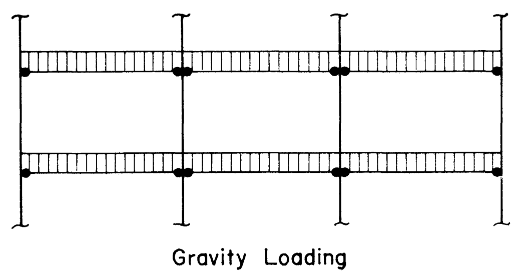

Figure 2

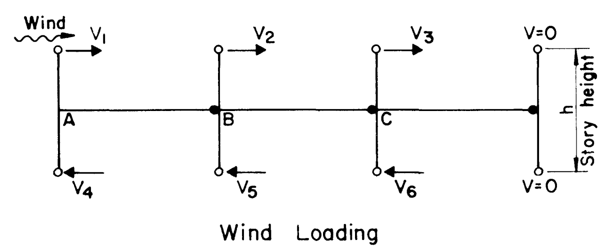

Figure 3

As a moment is applied to the connection, the initial M-ϕ relationship is non-linear (**A** to **B**). However, the curve levels out and, if properly designed, the connection has considerable ductility at the plastic moment.[4] When the connection is unloaded (or a moment imposed in the opposite direction) the connection acts linearly (**B** to **C**). *A moment connection, therefore, can result in a plastic hinge in one direction but an elastic connection in the opposite direction.*

Recognition of the elastic unloading behavior of a plastic moment connection is the basis for this proposed design procedure.

CONNECTION DESIGN

The connection is designed so that the components (angles, plates, or tee stubs, etc.) yield under the factored ($F = 1.3$) wind moment.* Using allowable stress design values, this is equivalent to designing the connection for the working load wind moment with the permissible $\frac{1}{3}$ increase in allowable stress. This results in a required plastic moment capacity less than that of the girder and, therefore, plastic hinges could develop in the connections under gravity loading prior to application of wind loads (Fig. 2).

As wind loads are applied they produce moments at the leeward ends of the beams in the same direction as the gravity moments; however, at the windward ends the wind loads produce moments opposite in direction to the gravity moments. If plastic moments already exist due to gravity loads, the leeward ends act as plastic hinges, but the windward ends act as elastic moment connections. Therefore, only the windward end connections are available to resist the wind. (See Fig. 3.)

As shown in Fig. 3, the leeward column is not braced with a rigidly connected girder, so it cannot participate in the wind shear distribution. The total story wind shear, therefore, is distributed among the remaining columns. Figure 3 also shows that each connection is designed for the full tributory story shear. The required moment for each connection can be calculated in the following manner:

$$M_A = (V_1 + V_4)\frac{h}{2}$$

$$M_B = (V_2 + V_5)\frac{h}{2}$$

$$M_C = (V_3 + V_6)\frac{h}{2}$$

where V_1, V_2, V_3, V_4, V_5, V_6, are working wind shears.

Welds must be proportioned so that there is no possibility of overstress at a load below that which would cause yielding of the connection material. AISC recognizes the importance of this by requiring that in Type 2 connections "The connections have adequate inelastic rotation capacity to avoid overstress of the fasteners or welds under combined gravity and wind loadings." (See Specification Sect. 1.2.)

The bolts that resist moment are designed with friction-type connection values, so that rigid behavior is assured when the moment on the connection reverses direction.

* *This is conservative. It can be shown that the connection design could be based on a moment equal to the wind moment less the dead load gravity moment. However, a conservative estimate of the dead load moment would result in an unconservative wind moment, so this practice is not recommended.*

GIRDER DESIGN

The girders are designed as simple beams for gravity loads. This recognizes that the moment connection at one or the other end may be reduced to zero by the wind moment. Of course, the opposite end of the girder would be restrained by the plastic moment capacity of the connection, but it is conservative to ignore this.

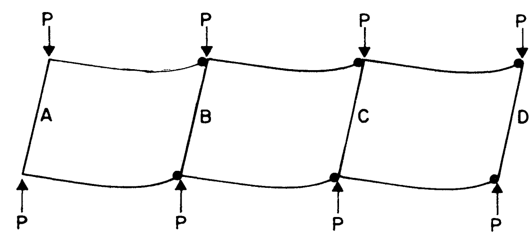

Figure 4

COLUMN DESIGN—GRAVITY LOADS ONLY

The columns must be designed to provide frame stability under gravity loads only.

With the connections designed for wind moments only, it has been shown that plastic hinges may develop in the beam connections at gravity loads below the ultimate load capacity of the beam. It is usually assumed that the frame has reached its ultimate load because, apparently, it has become an unstable mechanism. (See Fig. 2.)

Actually, this is not true. For instance, if the frame tends to sway to the right, the left-hand connection on each beam tends to unload and, as we have seen, is elastic and provides the necessary rigidity between the column and the girder (Fig. 4).

Conversely, if the frame sways to the left, the right-hand connections provide the rigidity.

The columns can be designed for this condition using using the effective length method for columns in unbraced frames, but with two essential differences from conventional rigid frame design:

1. Each column is restrained against rotation by only one girder and that girder is essentially pinned at its far end. Therefore, in computing the stiffness factor for use in the AISC nomograph, the girder stiffness should be modified by using 0.5 times I_g/L_g as shown on page 49 of Ref. 5.**

2. The exterior leeward column can not participate in frame stability. See Fig. 4. The stability of the frame, however, can be assured by designing the remaining

** *The moment of inertia of the connection is less than that of the girder, reducing the stiffening effect somewhat. Because of the comparatively short length of the connection, the reduction is small and may be ignored.*

columns to support the total frame load. The procedure is described in Ref. 6. There are many alternatives, of course, by which the frame load may be distributed to the participating columns. In the example in this paper, the interior columns are first sized for weak-axis strength and then their strong-axis capacity is evaluated. Their combined strong-axis strength is then deducted from the entire frame load and the single participating exterior column is designed (strong-axis) for this load.

Columns also must resist some moment under gravity loads. Although considerably smaller than in rigid frame design, these moments may affect design.

For the exterior column, the moment in the beam-to-column joint, M_j, is equal to the moment capacity of the connection, M_c. It may be assumed that this moment is distributed one-half to the upper column and one-half to the lower column.[7] The moment in the exterior column, therefore, is $M_c/2$.

For the interior column, M_j is the greatest realistically possible difference between the two moments resulting from the girders framing into the joint. Checkerboard loading may be a factor in this respect.

$$M_j = M_c - M'$$

where M_c is the larger of the two connection moment capacities and M' is the other connection moment capacity or the dead load fixed end moment of the other girder, whichever is smaller.

COLUMN DESIGN—COMBINED GRAVITY AND WIND LOADS

Columns must be designed to resist combined gravity and wind loads, utilizing the ⅓ increase in allowable stress permitted by the Specification.

It has been seen that the girders may develop hinges in the connections prior to the application of wind loads. Since the wind load produces moment at the windward end of the beam in the opposite direction as the gravity moment, the windward hinge reverses direction under wind loads and becomes elastic, providing the necessary rigid connection. See Fig. 3.

Wind loads may be determined from an elastic analysis or an assumed wind shear distribution. In Fig. 3, the left-hand, lower, exterior column would be designed for a moment at the allowable stress, $M = V_4(h/2)$.

DESIGN EXAMPLE

Given:

Design the girders, columns, and connections for the intermediate story of a multistory building shown in Fig. 5. Story height is 12 ft. Use A36 steel. The frame is braced in the direction normal to that shown. Checkerboard loading need not be considered.

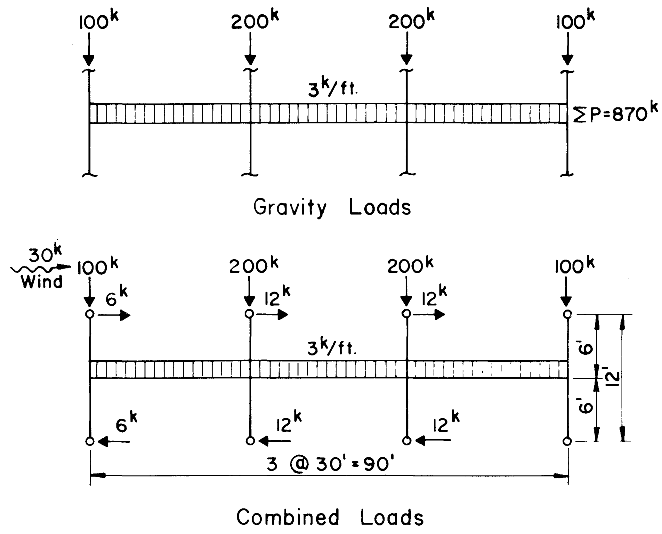

Figure 5

Solution:

Girder Design:

$M = \frac{1}{8}(3)(30)^2 = 338$ kip-ft

Use: W24×76 ($M_R = 352$ kip-ft; $I_x = 2100$ in.^{4})

Interior Column Design:

Y-Y axis—gravity loads:

$P = 290$ kips

Try W14×61 ($P_y = 314$ kips @ 12 ft)

X-X axis—gravity loads (frame stability):

Try W14×61:

$$\text{Elastic } G_T = G_B = 0.5\left(\frac{2 \times 641/12}{2100/30}\right) = 3.05$$

$f_a = 290/17.9 = 16.2$ ksi

Stiffness Reduction Factor $= 0.565$ (see Ref. 8)

Inelastic $G_T = G_B = 0.565(3.05) = 1.72$

From AISC nomograph: $K = 1.5$

$$\frac{KL}{r_x/r_y} = \frac{1.5(12)}{2.44} = 7.4 \text{ ft}$$

From AISC manual, pg. 3–16: $P_x = 349$ kips

The interior columns can support $2 \times 349 = 698$ kips of the total 870 kip frame load. The remaining load must be supported by one exterior column.

X-X axis—combined loads:

$M = 12(6) = 72$ kip-ft

Try W14×61.

Use the approximate interaction formula from Ref. 9, modified by a coefficient of 0.75 to account for the ⅓ increase in allowable stress:

$$P_{eff} = 0.75[P_0 + mM]$$

$$m = 1.7 \text{ (from Table 1 of Ref. 9)}$$

$$P_{eff} = 0.75[290 + 1.7(72)]$$
$$= 309 \text{ kips} < 349 \text{ kips } \mathbf{o.k.}$$

Use: W14×61 for interior columns

Exterior Column Design:

X-X axis—gravity loads (frame stability):

Each exterior column must be designed for the portion of the total frame load not supported by the interior columns.

$$P = 870 - 698 = 172 \text{ kips}$$

Try W12×45:

$$\text{Elastic } G_T = G_B = 0.5\left(\frac{2 \times 351/12}{2100/30}\right) = 1.67$$

$$f_a = 145/13.2 = 11.0$$

Stiffness Reduction Factor = 0.973

Inelastic $G_T = G_B = 0.973(1.67) = 1.62$

From AISC nomograph: $K = 1.5$

$$\frac{KL}{r_x/r_y} = \frac{1.5(12)}{2.65} = 6.8 \text{ ft}$$

From AISC Manual, pg. 3–18:
$P_x = 251$ kips > 172 kips **o.k.**

X-X axis—combined loads:

$$M = 6(6) = 36 \text{ kip-ft}$$

Try W12×45:

$$P_{eff} = 0.75[P_0 + mM]$$
$$= 0.75[172 + 2.1(36)] = 185.7 < 251 \text{ } \mathbf{o.k.}$$

Y-Y axis—gravity loads:

$$P = 145 \text{ kips}$$

Try W12×45 ($P_y = 211$ kips @ 12 ft)

Use: W12×45 for exterior columns

Connection Design (see Fig. 6):

Design moments:

$$M_{ext} = 6 \text{ kips} \times 6 \text{ ft} \times 2 \text{ cols.} = 72 \text{ kip-ft}$$

$$M_{int} = \frac{12 \text{ kips} \times 6 \text{ ft} \times 2 \text{ cols.}}{2 \text{ girders}} = 72 \text{ kip-ft}$$

$$\text{Flange force} = \frac{72(12)}{24} = 36 \text{ kips}$$

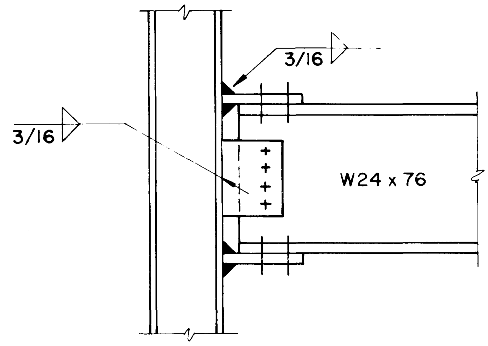

Figure 6

Plate net area req'd (considering ⅓ allowable stress increase):

$$A = 0.75(36)/22 = 1.23 \text{ in.}$$

Flange bolts:

Use ⅞-in. dia. A325 friction-type bolts.

No. of bolts req'd = 36/9.02 = 3.99 (use 4 bolts)

Flange plates:

Assume 8-in. wide plate

$$\text{Thickness req'd} = \frac{1.23}{8 - 2(0.875 + 0.125)} = 0.21 \text{ in.}$$

Use: ¼ × 8-in. plates

Plate-to-column welds:

Try $^3\!/_{16}$-in. fillet weld, E70XX:

Allowable fillet weld stress = 21.0 ksi

$$(2 \times 8)(21.0 \times 3/16 \times 0.707) = 45 \text{ kips}$$
$$> 0.75\ (36) = 27 \text{ kips } \mathbf{o.k.}$$

Shear plate:

$V = 3$ kips/ft (30 ft)/2 = 45 kips (at all columns)

Web bolts:

Use ⅞-in. dia. A325 bearing-type bolts, threads excluded from shear plane.

Single shear value = 13.23 kips

No. of bolts req'd = 45/13.23 = 3.40 (use 4 bolts)

Plate: Use $^5\!/_{16}$-in. thick A36 steel

$$F_v = 14.0 \text{ ksi}$$

$$\text{Plate length req'd} = \frac{45}{(0.3125)(14.5)} = 9.93 \text{ in.}$$

Use: $^5\!/_{16}$ × 12-in. plate

Plate-to-column weld:

Try $^3\!/_{16}$-in. fillet weld, E70XX.

$$(2 \times 12)(21.0 \times 3/16 \times 0.707) = 67 \text{ kips}$$
$$> 45 \text{ kips } \mathbf{o.k.}$$

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