# Wind Connections with Simple Framing

ROBERT O. DISQUE

IF A MULTI-STORY building is designed with simple framing (Type 2, AISC Specification Sect. 1.2), the frame must be braced to resist horizontal wind loads. This bracing may be provided by X-bracing, shear walls or "wind connections".

Wind connections are normally designed to carry *only* the moments due to wind, without regard to the additional moments caused by gravity loading of the girders. This assumption that a connection is "intelligent" and "knows" which moments to carry and which not to carry may seem paradoxical. However, the validity of such a connection in providing wind bracing for a simple frame can be justified.

The moment-rotation characteristic of a typical riveted or bolted moment connection is shown in Fig. 1. The shape of the curve depends on the stiffness of the connection and can only be determined by test. Tee-stub connections, for instance, usually have a steeper slope than does the typical "cap and seat angle" moment connection. The actual shape of the curve, however, does not effect the performance of the connection as a "wind connection" in conjunction with simple framing.

Fig. 2 is a typical beam line for a uniformly loaded girder. It plots the end rotation of the girder, ø, as a function of the end moment. For instance, if the end moment is 1/12 $wl^2$, the end rotation is zero. At the other extreme, if the end moment is zero, the rotation is that of a simple beam. Since ø is directly proportional to *M*, the "beam line" is a straight line between these two points.

In the design procedure considered here, the end connection is designed for the wind moment only. The girder, on the other hand, is designed as a simple beam for gravity loads only. The positive girder moment is not reduced due to any "fixity" supplied by the wind connection. The end moment which actually exists on the beam when full gravity loads are applied with no

*Robert O. Disque is Chief Engineer, American Institute of Steel Construction, New York, N. Y.*

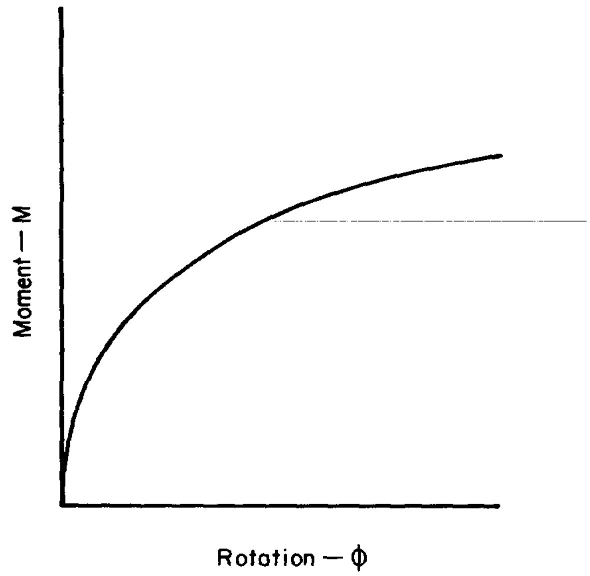


*Fig. 1. M-ø curve for a typical moment connection*

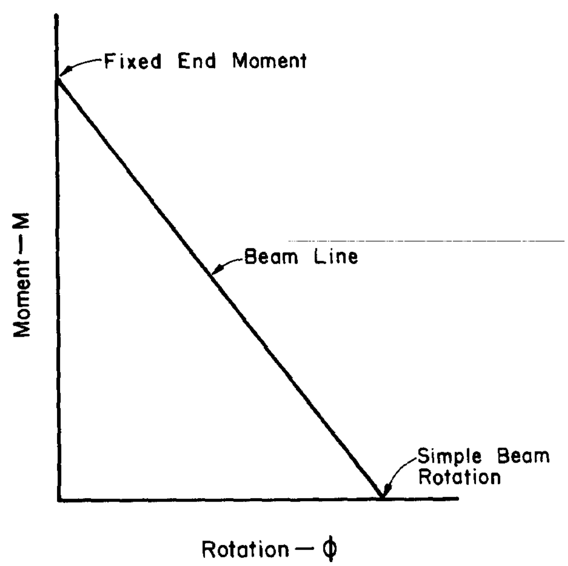


*Fig. 2. "Beam line" for a uniformly loaded girder*

wind load is therefore determined by superimposing Figs. and 2. This is shown as Point 1 on Fig. 3. Point 1, therefore, is the starting point in the history of the connection as the design wind is applied in each direction.

Fig. 4 represents a typical floor beam with wind connections in a tier building. Fig. 5 shows the moment-rotation behavior of Connection A when the initial wind load comes from the right. Let us trace the actual moment history of Connection A through a full wind cycle. The full gravity load is assumed to be applied during the entire wind cycle.

Point 1 *No wind.*
Point 2 *Wind from right.* Point 2 is reached by following the connection curve from Point 1.
Point 3 *No wind.* Point 3 falls on a straight line parallel to the initial slope which is the new connection curve for the yielded connection.
Point 4 *Wind from left.* Connection A is relieved by wind from the right and Point 4 falls on the same line as Point 3.
Point 5 *No wind.*
Point 6 *Wind from right.* Note that the connection will not exceed the moment value of Point 2.
Point 7 *No wind.* Point 7 coincides with Point 5.

As the cycle is repeated, the connection moment varies between Points 4 and 6, and at no time will it ever again exceed Point 6. It can also be shown[1] that the moment represented by the vertical distance between Points 4 and 5, 7 and between points 6 and 5, 7 is exactly one-half of the total girder wind moment.

If the original wind load had been from the left, the moment rotation curve of Connection A would be as shown in Fig. 6.

Point 1 *No wind.*
Point 2 *Wind from left.*
Point 3 *No wind.*
Point 4 *Wind from right.*
Point 5 *No wind.*
Point 6 *Wind from left.*
Point 7 *No wind.* Point 7 coincides with Point 5.

As the cycle repeats, the moment at Connection A varies between Points 4 and 6, but is never greater than the moment at Point 4. As in the case with initial wind load from the right, the moment represented by the vertical distance between Points 4 and 5, 7 is equal to that between 6 and 5, 7 and both are equal to one-half of the girder wind moment.

It is interesting to note that Point 4 in Fig. 5 and Point 6 in Fig. 6 represent very small end moments. The practice, therefore, of designing the girder as a simple beam is not unduly conservative.

Note that Figs. 5 and 6 apply equally to Connection B, except that the moments are caused by wind loads

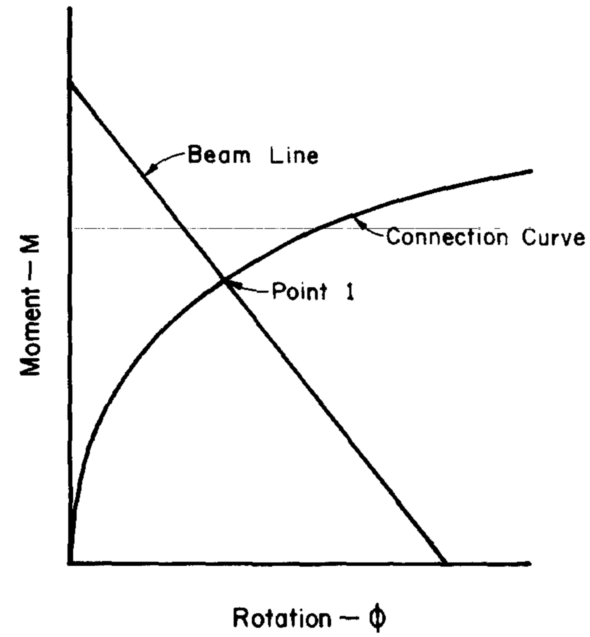


*Fig. 3. Intersection of connection curve and "beam line" determines moment and rotation under gravity load condition*

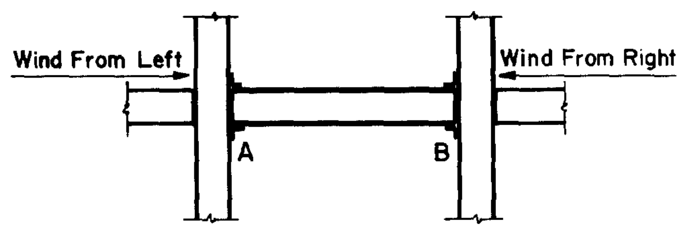


*Fig. 4. Schematic diagram of typical wind girder*

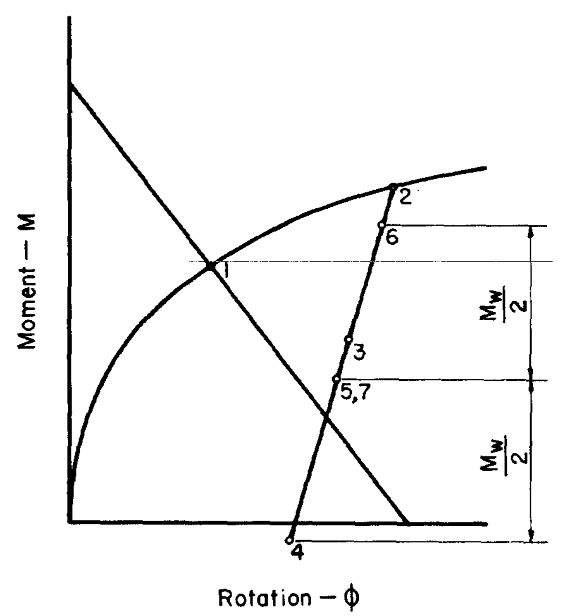


*Fig. 5. History of Connection A when initial wind load is from right*

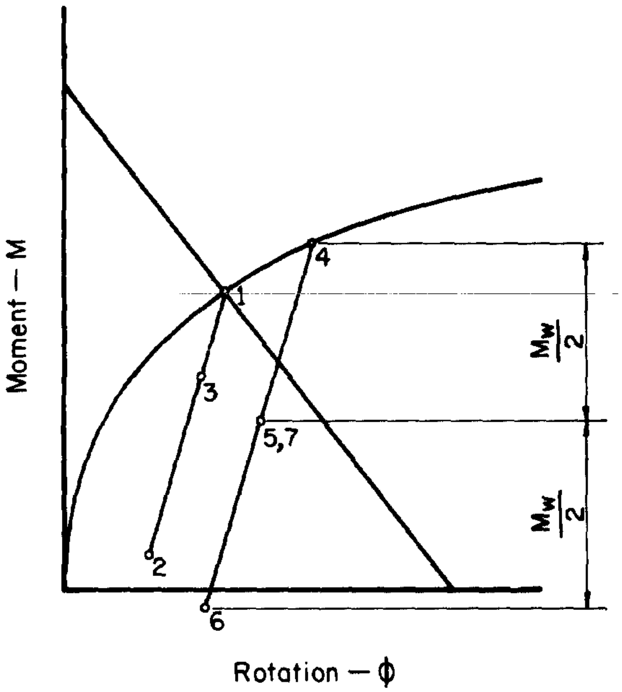


*Fig. 6. History of Connection A when initial wind load is from left*

in the reverse directions from those indicated for Connection A.

Another way of viewing this design procedure would be to consider a connection detailed and fabricated with a slope equal to the dead and live load rotation of a simple beam. The connection would have a positive moment under dead load only and would have exactly zero moment under dead plus live load.

Under such circumstances it can be readily seen that whatever moment capacity is required to resist wind would have to be supplied by the connection.

In using this design method, all the designer is doing is depending upon the gravity loads plus the first complete cycle of wind loading to perform the work of bending the connection to the correct end rotation angle more accurately than it could be detailed, fabricated and erected. To be assured of such performance by the connection, the designer must:

1. Insure that the connection has adequate moment capacity to resist the wind moments.
2. Insure that the detail has adequate rotation capacity (ductility) to avoid overstress of the connectors.
3. Design the beam to support total vertical loads as a simple beam.

A more complete discussion of these principles is given by Sourochnikoff.[1] The conclusion of this reference states, "However, if the connections are such that they can endure large inelastic deformations without failure, it will be found that, if the connections are adequate to resist wind stress moments alone at design stresses computed on the assumption that the connection is elastic, they are also adequate for the combination of gravity loads and wind loads."

This ability to undergo large inelastic rotations without failure is also a requirement for connections used in plastic design. Through the years, many typical moment connections have been tested in the laboratory and through experience, and are generally accepted for application in plastic design. Such connections could also satisfy Sourochnikoff's requirement. As stated by Beedle and Christopher,[2] "Nevertheless, information available to date makes it abundantly clear that if a rotation capacity (or ductility factor) of 8 to 10 is not realized at a steel moment connection, it is because some detail has been underdesigned."

Ever since multi-story steel frames were first built, variations of the design method described here have been used by structural engineers. In the hands of a competent and experienced designer, this method is a sound and economical approach for proportioning the connections and girders for riveted and bolted work. For such structures, the connections can be large and expensive in Type 1 (fully rigid) construction. For welded work, on the other hand, there seems to be little reason not to use Type 1 construction. End plate connections[3] would also probably be more appropriate with Type 1 construction.

The overall rigidity of a structure designed as Type 2 with wind connections is difficult to evaluate. The larger girders, as compared to those in Type 1, would tend to result in a stiffer structure. The lighter connections in Type 2, however, would tend to make the frame more flexible. Goble[4] has done some work on this problem, but to date the results are inconclusive. It would appear, however, that the overall rigidity of the structure is not greatly reduced due to lighter connections. The satisfactory performance of some of the world's tallest buildings seems to confirm this view.

## REFERENCES


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2. *Beedle, Lynn S. and Christopher, Richard* Tests of Steel Moment Connections, *Fritz Engineering Laboratory Report No. 205.79, October 4, 1963.*
3. *Disque, Robert O.* End Plate Connections, *Proceedings of AISC National Engineering Conference, 1962.*
4. *Goble, G. G.* A Study of the Behavior of Building Frames with Semi-Rigid Joints, *A report prepared at Case Institute of Technology for the American Institute of Steel Construction and the Ohio Steel Fabricators Association, 1963.*