Flexible Moment Connections

a new look at an old favorite

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"Type 2 with wind" connections receive an updated treatment—and a new name—in the 2005 AISC *Manual* and a recent *Engineering Journal* paper.

By now, it may be well known that the 2005 AISC *Specification for Structural Steel Buildings* combines ASD and LRFD into a single set of consistent and equivalent design provisions. And it may also be known that the AISC *Manual of Steel Construction*, 13th Edition, will be based upon these provisions and will provide many new benefits for the practical designer. But it may not be known to many that it contains coverage of an oldie but goodie that is like new again—flexible moment connections.

Originally covered in two papers by Disque in the AISC *Engineering Journal* (July 1964 and first quarter 1975), "Type 2 with wind" moment connections have been used in the design of many steel buildings. Use of this approach predated these papers and continues today. The simplicity of the approach and its utility harken back to the days when engineering judgment came first and foremost. But the lack of explicit consideration of more contemporary concerns, such as stability effects and drift, cast doubts on the modern usefulness of the approach. Because of a paper by Geschwindner and Disque just published in the second quarter 2005 *Engineering Journal*, this system continues to be available to the modern designer, including consideration of stability effects and drift. Note the name change: "Type 2 with wind" connections are now called flexible moment connections (FMC).

Aren't FMC Really Partially Restrained Moment Connections?

The use of partially restrained (PR) moment connections, and PR construction in general, is allowed based upon the requirements stipulated in AISC *Specification* Chapter B. It should be noted, however, that the designer attempting to meet these requirements can be in for quite an analytical challenge.

According to Rex and Goverdhan (2000), there is not a single authoritative guide to the design of PR moment connections, and there are "still a lot of gaps and problems with the design guidance" and "a lack of appropriate computer software." These authors are well qualified to make these statements—they work for one of the only (if not *the* only) firm that has regularly used PR moment frames in their design practice.

Rex and Goverdhan (2000) further describe the difficulties associated with the required assumptions as to the worst case of various load combinations. Describing the design of a four-story, multi-bay office building, seven load cases are analyzed. It is assumed that the loads are applied in steps or in a specific sequence. Of course, it is not possible to know for certain how accurate either one of these two assumptions is. In fact, because the actual loading sequence can never really be known, the designer who chooses to use PR construction must do so with care to ensure that the sequences used in design properly brackets the possibilities. If it doesn't, the final design may not conform to ASCE 7 and the AISC *Specification*. The designer must also account for the possibility that an as-built connection could be "softer" than assumed, which might invalidate the analysis and design.

Of critical importance is the determination of the moment–rotation characteristics of the connection to be used. The frame is loaded step-by-step, along the curve, with a particular load case and sequence. For instance, load case S7 from Rex and Goverdhan (2000) is as follows:

Step 1	Step 2	Step 3	Step 4
+1.0DL	+1.0SD	+1.0WL	−2.0WL

Step 5	Step 6	Step 7	Step 8
+2.0WL	−1.0WL	+1.0LL	+1.0NRL

where:
DL = Service Dead Load
SD = Superimposed Dead Load
WL = Service Wind Load
LL = Live Load
NRL = Non-reducible Live Load

When this loading sequence is repeated, the connection "shakes down" to a steady and repeating state along a line parallel to the initial slope of the curve. Note that this requires more than one iteration of sequential analysis and design—the connection behavior after shakedown will differ and the system must be properly designed for both the initial behavior and for that after shakedown occurs.

It is also noted that, in the load cases used by Rex and Goverdhan, the connection never went through a moment reversal. Accordingly, the authors state, if this were the case, more research would be needed.

The Simplifying Case of Flexible Moment Connections

The use of Flexible Moment Connections is a more rigorous and conservative application of PR moment connection concepts. FMC essentially provide a simple determinate procedure (akin to plastic design), with its roots in the "Type 2 with wind" ASD procedure, that has been successfully used for over 85 years in tens of thousands of buildings.

With FMC, the wind girders are designed as simple beams and the connections are designed for a plastic moment to resist the applied wind moment. No load sequence assumptions need to be made because frame members and the connections are designed for the load case that results in the largest load. The "shake down" process begins with the intersection of the beam line and connection curve. The beam line is the moment–rotation line of the beam that, with FMC, is the simple beam design for the worst loading case on the beam.

Geschwindner and Disque (2005) show that as long as the plastic moment capacity of the connection is not exceeded by the moments resulting from the maximum lateral load, the frame can resist all load combinations specified by ASCE 7. Furthermore, plastic design theory emphasizes that, with FMC, the frame cannot be weakened by an overly strong or stiff connection. That is, the Lower Bound Plastic Limit Theorem states that a load computed on the basis of an equilibrium moment distribution, in which the moments are nowhere greater than M_p, is less than or equal to the true plastic load. The actual shape of the connection curve is not relevant in FMC; only its plastic moment capacity is a factor.

In addition to strength, frame stability for second order effects must be checked. Rex and Goverdhan do this by a computer program using a stability function stiffness matrix for the column elements. Geschwindner and Disque do this by a conservative application of the simpler AISC *Specification*-defined K–Factors and column amplification factors.

Proven Performance

The historical performance of structures can be useful in evaluating the success of a particular design procedure. There have been tens of thousands of these structures built and almost a century of successful history. Moreover, practically all of these buildings have been designed with less rigorous requirements than would result from the modern procedure provided by Geschwindner and Disque (2005).

There is a myth that FMC (Type 2 with wind) could only have been used "in the old days" when tall buildings had heavy masonry walls. The myth goes on to say that the procedure results in frames that are too flexible for modern buildings with light curtain walls. This is false.

It is certainly true that the typical structures designed and constructed prior to World War II had heavy masonry exterior walls, which provided additional stability. The change from this type of construction came in 1952, beginning with the 24-story Lever House on Park Avenue in New York City—the first curtain wall building in the world. The architects were Skidmore, Owings and Merrill, and the building made architectural history. The steel frame was designed by Weiskopf and Pickworth. It is Type 2 with wind, still there today and behaving beautifully.

Soon after the Lever House came the UN Building on Manhattan's East River. The structural engineers were Edwards and Hjorth, the engineers who also designed the Empire State Building. It is also Type 2 with wind and has the reputation as being solid as the Rock of Gibraltar in the face of many furious wind storms, and even a few hurricanes. The wind girders, designed as simple for gravity loads, are very stiff. The wind connections, not designed to match the girders, are robust tee stubs. The reason for its stellar performance can only be speculated, but in the authors' opinion it is probably because, at service loads, the connections remain elastic or close to it. Combined with the stiff girder, the result is a very rigid frame.

From the 1950s through the 1970s, thousands of Type 2 with wind structures were built in the U.S. and Great Britain, practically all with light skin. To the authors' knowledge, no problems have ever been reported. And it should be remembered that these venerable old structures were likely designed with little or no consideration of modern niceties like second-order effects.

Useful Today

Geschwindner and Disque (2005) provide a tried and true approach to steel building design that maintains the simplicity of the past while accounting for the modern advances required in today's analysis and design. Their full paper is available in the second quarter 2005 AISC *Engineering Journal* and at **www.aisc.org/epubs**. ★

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