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Braced Frame Seismic Connections

I am trying to understand the requirements for bracing connections in AISC 341-02 (the 2002 AISC seismic provisions). In Sections 13.3a and 14.2, the required strength of the connection is the expected tensile strength of the brace, $R_yF_yA_g$. How can I meet that strength requirement for tension in the connected elements? According to Sec. 13.3b:

> **The design tensile strength of the bracing members and their connections, based upon the limit states of tension rupture on the effective net section and block shear rupture strength, as specified in LRFD Specification Section J4, shall be at least equal to the Required Strength of the brace as determined in Section 13.3a.**

The tensile rupture limit state is given in Sect. J4 as ϕF_uA_e with ϕ = 0.75 (LRFD) and $0.5F_uA_e$ (ASD). For all standard steel grades, $R_yF_yA_g$ is greater than the tension rupture strength. Does that mean that all bracing members need to be reinforced at the connection? If so, how far from the connection should the reinforcing extend?

Yes, in many cases, the brace will need to be reinforced to increase net area in order to satisfy the tension rupture limit state. The extension of reinforcing depends upon the detail used and can be designed for the force in the reinforcing element. Often, minimum edge distance (when using bolts), the length of the tensile and shear planes used in the block shear expression, and other rupture limit states dependent on length, bearing, etc. will control the design.

—Sergio Zoruba, Ph.D., P.E.

1964 Drawings

I am examining a set of documents from 1964. Which manual of steel construction might have been used for steel design in 1964? I see shapes listed as 14B22, 14B17.2, etc. Furthermore, the drawings simply state that "*fs* = 22 ksi." Was steel of this vintage ASTM A36 with F_y = 36 ksi or something else?

In 1964, ASTM A36 was starting to become very common, but ASTM A7 was still being used. A new AISC specification was adopted in 1963 (included in the 6th edition manual). This specification recognized both grades, since ASTM A7 was not officially discontinued until 1967.

The basic allowable working stress for structural steel was $0.6F_y$ prior to the 1963 specification, with a ten percent increase permitted for flexural design of compact sections (to acknowledge the plastic capacity of such compact shapes). Non-compact shapes were still limited to the $0.6F_y$ maximum stress limitation. Given the F_b = 22 ksi notation on the documents, there is a good possibility that the designer of the project was assuming ASTM A36 steel, since 0.6×36 = 21.6 ksi, commonly taken as 22 ksi. You may want to consider some testing to verify what was actually used.

—Kurt Gustafson, S.E., P.E.

Block Shear

I noticed that the tension yield component of the block shear check is no longer included in the 2005 specification, Eq. J4-5. The new procedure is obviously simpler without the need for dealing with the tension yield component. Could you explain the change?

Yes, this change was intentional. When the ASD model (shear rupture-tension rupture) and LRFD model (shear yield-tension rupture, shear rupture-tension yield) were combined in the 2005 AISC specification, we re-evaluated the data. We noticed that the shear component was always dominant and that true block shear rupture did not depend upon the tension yielding. Rather, the shear mechanism would have to form in yield or rupture, and then the tension plane would fail in rupture. Said another way, tension yielding occurred before the shear mechanism was reached, making it unnecessary to consider.

The new approach is easier. We tried to use the even simpler ASD model, but we couldn't assume that rupture-rupture would always be the controlling mechanism.

—Charlie Carter, S.E., P.E.

AISC Specification Seminar

I have a question on an example in the 2005 AISC specification seminar. You apply Eq. E7-17 to obtain the equivalent width of the web. AISC specification Section E7.2(a) indicates that this equation is valid only for $b/t \geq 1.49(E/f)^{0.5}$. The value of this expression is $1.49(29{,}000/28.2)^{0.5}$ = 47.8. The value of b/t = 39.6, which is less than 47.8—and it would seem that Eq. E7-17 is, therefore, not applicable. No alternative to Eq. E7-17 is provided in the specification. What is the intention of the AISC specification in this situation?

You are correct. The approach that I took was to look at the conservative approach on the limit, using F_y rather than f. Once I used Eq. E7-17, the fact that the element was not slender comes out because the resulting value of b_e is greater than b. You note that there is no alternative to E7-17 if $b/t < 1.49(E/f)^{0.5}$. That is correct, because if that is the case, the element is not slender. If you set $b/t = 1.49(E/f)^{0.5}$ and substitute this into E7-17, you will get close to $b_e/t = 1.49(E/f)^{0.5}$. It is not exact because of rounding in the equations.

—Louis Geschwindner, Ph.D., P.E.

Cantilever Framing

I am analyzing and retrofitting a building from 1965. It is a one-story roof structure. Column bays are typically 30 ft by 60 ft. Steel bar joists are spaced at 3 ft o.c. and span the 30 ft dimension. Beams are cantilevered 9 ft over the columns with 42 ft infill beams spliced between the major girders. There is no bottom flange bracing or full-depth stiffeners at any point along the major girders. Are there any references

from this era that describe this system and the design philosophy at the time?

The system that you describe was commonly called a drop-in-cantilever system back in the 1960s. I do not know any specific instruction texts covering the system, but it was quite commonly covered in basic design courses. The concept of the system is to balance the negative and positive moments due to uniform loads on the girders by strategically placing the splice points. Table 3-22b in the 13th edition *Steel Construction Manual* is an aide in developing such layouts.

The omission of bottom flange bracing at the column tops, such as kickers, joist bottom chord extensions with the joists designed for the resulting forces, or stiffeners, is of concern. Refer to page 2-15 in the 13th Edition AISC *Steel Construction Manual* for guidance on this important stability concern, including the above-mentioned solutions.

—Kurt Gustafson, S.E., P.E.

Tee Stem in Flexural Compression

Does the 2005 AISC specification cover the case of a WT member loaded in the plane of symmetry with the stem in flexural compression? Section F9 does not distinguish whether or not the stem is in compression or tension, but it does not seem to check the buckling of the web, which I would suspect is necessary for this case.

The case of a tee stem in flexural compression is embedded in Eq. F9-4 of the 2005 specification; note that the sign of B is assigned based upon whether the stem is in tension or compression. This is discussed in the Commentary to Section F9. Note also the differences in Eqs. F9-2 and F9-3.

—Kurt Gustafson, S.E., P.E.

Backing Bar Requirements

Is a backing bar required when welding the beam bottom flange to the column flange on a SMF if the beam flange is tight to the column?

The AISC Specification does not establish welded joint details. You may want to look at the AWS D1.1 *Structural Welding Code - Steel*. The requirement for a backing bar will be dependent on the type of weld procedure specification being followed. If you are using a CJP groove weld that is being welded from one side only, a backing bar is required—not only to prevent the weld material from dripping out of the joint, but also to establish a good root penetration. A one-sided joint welded in the down-hand position is the prevalent joint for field welding a beam flange to column flange joint. A CJP groove weld made from both sides, such as a TC-U4b, is prequalified without a backing bar, but this would not usually be efficient and would rarely be used in field welding, as the back side would be welded overhead, requiring more time and welders qualified to weld in that position.

—Sergio Zoruba, Ph.D., P.E.

Slotted-Hole Dimensions

Table J3.3 in the 2005 *Specification for Structural Steel Buildings* gives dimensions for slotted holes. What is the definition for the slotted hole length? Is it the total length of the hole (i.e., from the outermost radius points on the circular portion) or the length of the straight portion of the hole?

The specified slot dimensions are the dimensions of the completed slot—that is, the edge-to-edge distance (not the center-to-center distance).

—Sergio Zoruba, Ph.D., P.E.

Lally Columns?

We are reviewing the roof of a steel-framed building built in 1951, but we only have a few sheets of the original contract documents. The columns are referred to as 5-in. "Lally Columns." The column-beam connection is a single ¼-in. steel knife plate through the column with six ¾-in. bolts.

Was "Lally" used as a generic term for pipe columns? The connection described wouldn't work with today's Lally columns. It would seem to me that we are closer to a standard pipe column.

I can't say what the designer had in mind, but in my experience the term "Lally Column" was used in reference to a proprietary concrete-filled column system. However, some terms, such as this, often become distorted as they become inappropriately used in conversation. Your description seems to indicate that this may not be a true Lally column. Have you tested to see if the pipe is concrete filled? I can only suggest that you run some quick calculations to see if the column without the concrete can safely support the design loads of the original structure. This should give you a good idea what the original engineer was thinking.

—Kurt Gustafson, S.E., P.E.

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If you have a question or problem that your fellow readers might help you solve, please forward it to us. At the same time, feel free to respond to any of the questions that you have read here. Contact Steel Interchange via AISC's Steel Solutions Center:

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