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steel interchange

Collector Design

In the 1st Edition of the AISC *Seismic Design Manual*, Example 5.1 does not use the concrete filled steel deck diaphragm as a continuous lateral brace for compression loads. Why does it not account for the beneficial effects that the diaphragm may provide?

You are correct that, in Example 5.1, the unbraced length for weak-axis flexural buckling in compression is based on the spacing of the bottom flange braces. The continuity of the top flange brace is ignored and discrete kickers are provided between the deck and bottom flange. This is a conservative method that was employed at the time due to the absence of relevant published design guidance for this case.

In reality, the continuous bracing provided by the slab reduces the weak-axis unbraced length for flexural buckling in compression to zero. The governing limit state actually is constrained-axis torsional buckling (CATB). This limit state occurs with the cross-section rotating roughly about the flange-to-deck interface. There was a difficulty in calculating this limit state because until recently, it was only treated in advanced mechanics textbooks such as those by Timoshenko and Bleich. Discrete bottom flange braces may still be required; however, if they are, the required spacing will likely be greater than that determined using the more conservative approximation used in Example 5.1.

AISC Design Guide No. 25 contains a treatment of this limit state for monosymmetric I-shapes that you can adapt for use with doubly-symmetric shapes. This topic has also been highlighted in a paper in the ASCE *Journal of Structural Engineering*, "Torsional Bracing of Columns" by Helwig and Yura. In fact, the 2nd Edition AISC *Seismic Design Manual* used these resources to develop a CATB design procedure specifically aimed at doubly symmetric collectors. It is presented in Part 8 and is used in the collector design example.

So the reason that the slab is not used as a brace in the 1st Edition *Seismic Design Manual* design example is not because it was not adequate, it was simply a conservative approximation. I'm happy to say that conservatism is no longer necessary.

Heath Mitchell, S.E., P.E.

Seismic Design

Is it true that the AISC *Seismic Provisions* are not required to be used in the design of seismic force resisting systems with $R \le 3$?

No. This was true when the concept of R=3 was first introduced, but several nonbuilding systems now exist with an R lower than 3 and detailing requirements in the AISC *Seismic Provisions*. The Commentary to the AISC *Seismic Provisions* (a free download at **www.aisc.org/2010sp**) states:

"The *Provisions* are intended to be mandatory for structures where they have been specifically referenced when defining an R factor in *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7 (ASCE, 2010). For steel structures, typically this occurs in seismic design category D and above, where the R factor is greater than 3. However, there are instances where an R factor of less than 3 is assigned to a system and the *Provisions* are still required. These limited cases occur in ASCE/SEI 7 Table 12.2-1 for cantilevered column systems and Table 15.4-1 for nonbuilding structures similar to buildings. For these systems with R factors less than 3, the use of the *Provisions* is required. In general, for structures in seismic design categories B and C, the designer is given a choice to either solely use the *Specification* and the R factor given for structural steel buildings not specifically detailed for seismic resistance (typically, a factor of 3), or to assign a higher R factor to a system detailed for seismic resistance and follow the requirements of these *Provisions*."

There also is a good summary of this topic in the Scope on page xi of the AISC *Seismic Design Manual*, 2nd Edition.

Erin Criste

Bolt Design Strengths

Table 7-1 in the AISC *Steel Construction Manual* lists the shear strength of A307 bolts as 13.5 ksi for ASD. How is this value calculated?

To get to the 13.5 ksi you start with ASTM A307, which states that these fasteners have a tensile strength equal to 60 ksi. You then can go to the Commentary to the AISC *Specification* to find the rest of the story. The Commentary describes the factors that are applied and what they address:

"The factor 0.563 accounts for the effect of a shear/tension ratio of 0.625 and a 0.90 length reduction factor. The factor of 0.450 is 80% of 0.563, which accounts for the reduced area of the threaded portion of the fastener when the threads are not excluded from the shear plane. The initial reduction factor of 0.90 is imposed on connections with lengths up to and including 38 in. (965 mm). The resistance factor, ϕ, and the safety factor, Ω, for shear in bearing-type connections in combination with the initial 0.90 factor accommodate the effects of differential strain and second-order effects in connections less than or equal to 38 in. (965 mm) in length."

So, 60 ksi multiplied by the product of (0.625)(0.8)(0.9) equals 27 ksi. Note that this is the value given in Table J3.2 of the AISC *Specification*. When you divide by the factor of safety, Ω (which is equal to 2.00 in this case), the result is 13.5 ksi.

The other values listed in Table J3.2 can be derived in a similar fashion.

Larry S. Muir, P.E.

Stainless Steel Welding

A project specified directly welded stainless steel members. I have been told that when welding stainless steel, the welders need to be certified in addition to their current certification for mild steel. Is this correct?

Yes. Stainless wires run differently than carbon or low-alloy wires. The welder should be qualified by welding using a stainless wire of the same F number that they will use in production. AWS D1.1 Clause 4 provides qualification information. In addition, there is a chance that the welding procedure is not prequalified. If that is the case, then the contractor as well as the welder will have to qualify the stainless welding procedure. The person (welder) qualifying the procedure can be qualified as a welder by virtue of welding the WPS qualification test.

And let me add one more idea, just in case it applies: Welding carbon steel to carbon steel has one set of requirements, covered by the AWS D1.1 Code. Welding stainless steel to stainless steel has a separate set of requirements, covered in the AWS D1.6 welding code. Welding carbon steel to stainless steel is a third, completely different item with separate considerations. There is not a specific welding code on this latter topic, but there is some discussion of it in the Commentary to AWS D1.6.

Thomas Schlafly

End Plate Design

End-plate moment connection design assumption #9 on page 12-10 of the AISC *Steel Construction Manual* states: "For non-seismic connections, when the required moment is less than the available flexural strength of the beam, the end plate connection can be designed for the required moment but it is recommended that the connection be designed for not less than 60% of the available flexural strength of the beam." Does this apply only to the connection of the beam to the end plate or does it also apply to the connection of the end plate to the column?

The original source of the recommendation you reference is AISC Design Guide 16. It states:

"Normally, the beam flange to end plate weld is designed to develop the yield strength of the connected beam flange. This is usually done with full-penetration welds but alternatively, fillet welds may be used for thin flanges. When the applied moment is less than the design flexural strength of the beam, the beam flange to end plate weld can be designed for the required moment strength but not less than 60% of the specified minimum yield strength of the connected beam flange."

Thus, the intent is that only the beam flange to end plate weld is designed for this minimum strength. The minimum demand is intended to account for uneven stress distributions that can occur across the flange-to-end plate weld.

Heath Mitchell, S.E., P.E.

SFRS Column Base Design

At what point do the amplified seismic loads that may have been required for the connection or member design in the upper frame structure, cease to apply at the column base?

This is covered in Section D2.6 of AISC 341-10. Each subsection that discusses required strengths has wording to the effect that "the required strength of column bases, including their attachment to the foundation, shall be…" In addition, *column base* is defined in the glossary of AISC 341 as:

"Assemblage of structural shapes, plates, connectors, bolts and rods at the base of a column used to transmit forces between the steel superstructure and the foundation."

Therefore, the anchor rods, shear keys, etc. and their connection to the foundation are subject to the required strength in Section D2.6. However, the foundation itself is not subject to these design requirements.

Heath Mitchell, S.E., P.E.

Vertical Welding

An inspector has pointed out that many vertical welds on one of our jobs were started from the top instead of the bottom. Will these welds need to be ground out and redone?

Clause 3.7.1 of AWS D1.1 requires that vertical welds be made upward if they are to be considered prequalified. One option you have is to qualify the weld per Clause 4 of AWS D1.1. Your goal should be to reproduce the process used to make the existing welds so that you will have an accurate representation of the existing condition.

If the weld does not pass qualification, then you will probably have to remove the existing weld and re-weld.

Larry S. Muir, P.E.

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