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A36 plate with F_y of Grade 50 plate

The material test reports for the A36 plates used in my projects consistently conform to material with a yield of 50 ksi. I have two questions: Can I design these plates accounting for the higher yield strength? And should I start specifying all plate material as A572 Grade 50 instead of A36?

A36 often has a yield strength in excess of 36 ksi and sometimes in excess of 50 ksi. If you have purchased the steel as A36, it should be designed assuming 36 ksi. If the product is certified as meeting a standard with a higher yield strength, as may be the case if the plate is dual certified, you could base the design on the characteristics of the other grade.

You should base your plate grade specification on your experience and the advice of your fabricator. If there is an advantage to using 50-ksi steel and it is readily available, then there is no reason not to use it. However, keep in mind that availability can vary in terms of region of the country and volume of materials purchased.

Larry S. Muir, P.E.

Local Buckling

The local buckling limits in AISC Specification Table B4.1 are all based on the square root of the inverse of the yield strength. This means for a given shape, a lower-strength steel will result in a compact section while a higher-strength steel will not. How can the shape made from the higher-strength steel have a lower strength, as predicted by the compactness limit?

The compactness of the section does not directly correspond to the yield strength of the member. The local buckling load is independent of the yield strength. However, the limits and equations you see use F_y because the slenderness limits are used to predict whether the element will yield or buckle.

It may be easier to understand this by thinking in terms of Euler buckling. Let's say we have a perfect column that buckles in the elastic range at an Euler load for which the stress on the cross section is 40 ksi. If the column is made from A36 steel, then it will yield, in theory, when the stress in the member reaches 36 ksi. If the column is made from A992 steel ($F_y = 50$ ksi), then it will buckle when the stress in the member reaches 40 ksi. Thus, in this case buckling is the controlling limit state for the A992 material; it is not for the A36 material. It's the same idea for local buckling versus yielding.

As a final thought, note that even though the section made from A992 material buckles before it yields, it still has a greater available strength than the same shape in A36 material.

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Distortion of Moment End Plates

The plates in an end-plate moment connection have distorted due to the shrinkage of the welds. Is there a tolerance on such distortions?

No, but these distortions can be accounted for. The erector can use some judgment and may elect to pull the plies together when they are bolted—if the gaps are small enough that the bolts can be used to do that. Filling gaps with finger shims is also an option. The Commentary to Section 6.9 of AISC 358-10 states: "Tests have shown that the use of finger shims between the end plate and the column flange do not affect the performance of the connection (Sumner et al., 2000a)."

Several years ago, I looked into the effect of this distortion on the connection performance. All of the available research, which included several full-scale tests that were intentionally fabricated with distortions up to 5 mm, concluded that the strength is not affected by distortion. All of the research papers I reviewed are in the reference list below.

References

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- Zachev, S. (1989), "Influence of Initial Distortions of Flange on Strength of End-Plate Connections," *Proceedings of the International Colloquium on Bolted and Special Structural Connections*, Vol. 3, USSR, Moscow, May 15-20, pp. 111-117.

Bo Dowsnell, P.E., Ph.D.

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Pretensioned Bolts in Moment End Plates

I have designed end-plate moment connections using the procedure in AISC Design Guide 16, which allows the use of snug-tightened A325 bolts. The erector has installed tension control (TC) bolts, which are pretensioned. Will this cause a problem?

The first thing that needs to be recognized is that there is no upper limit on the pretension that can be applied to a snug-tightened bolt. A snug-tightened joint is not a joint without pretension, but more properly should be viewed as a joint in which the bolts are acceptable with an undetermined level of pretension—where the level of pretension is irrelevant in meeting the requirements of the connection. Even if TC bolts had not been installed, it is possible that the bolts would have significant pretension.

The calculations on page 11 of Design Guide 16 indicate that the bolt rupture limit state considering prying action is dependent on the level of pretension. When the connection is designed assuming a snug-tightened condition, a pretension significantly less than the full pretension is assumed. A higher pretension than that assumed in the calculations can only result in greater strength. Therefore the fact that the bolts have potentially been pretensioned will not be detrimental to the strength of the connection.

I also have to mention that the use of TC bolts does not guarantee that full pretension will be achieved. Only the use of TC bolts in conjunction with the proper installation procedures as outlined in Section 8 of the RCSC *Specification* will ensure proper pretension.

Carlo Lini, P.E.

Flange-to-Web Weld in Plastic Section

I have typically used the term VQ/I to determine the shear on the welds joining the flanges and web of a built-up girder. When a girder is designed based on the plastic section modulus, the flexural strength is significantly higher than that predicted using the elastic modulus. Is VQ/I still appropriate when the plastic section modulus is used?

Many texts illustrate the calculation of shear flow using VQ/I , and this approach is clearly based on an elastic model. Though engineers commonly think the higher flexural strength predicted by the plastic section modulus must be accompanied by a greater demand on the flange-to-web weld, the elastic model will actually predict a higher demand on the flange-to-web weld than the plastic model.

The shear flow can be derived by drawing the free body for a 1-in. segment of the beam and applying equal and opposite shears at either end. The shear will produce a moment equal to the shear force multiplied by the 1-in. moment arm. An equal and opposite moment must exist in the beam's cross section. When a section is cut between the web and the flange, the force that must be transferred through the weld, the shear flow, is VQ/I if an elastic stress distribution is assumed. If the built-up section is symmetrical, then the shear flow will be equal to the force in the flange:

$$\frac{V \left(1 - \frac{t_f}{d}\right) b_f t_f}{I}$$

When the plastic section modulus is assumed, the flange force, $(V/Z)b_f t_f$, and the associated shear flow will always be less than the shear flow predicted by the elastic model. This makes sense, since the web will resist a greater portion of the moment in the plastic model, thereby reducing the force in the flange.

Larry S. Muir, P.E.

Large-Diameter Bolts

I require 2-in.-diameter A490 bolts on my project. Are these bolts available? How should they be installed?

ASTM A490 bolts are only available in diameters from ½ in. through 1½ in. The AISC *Specification* designates ASTM A490, A490M, F2280 and A354 Grade BD as Group B since all have similar strengths. The 2-in.-diameter bolts you need can be specified as A354 Grade BD.

Relative to installation, the 2-in.-diameter A354 Grade BD bolts can likely be installed snug-tight. If pretensioning is required, such as for a slip-critical connection, *Specification* Section J3.1 states: "When ASTM A354 Grade BC, A354 Grade BD or A449 bolts and threaded rods are used in slip-critical connections, the bolt geometry including the thread *pitch*, thread length, head and nut(s) shall be equal to or (if larger in diameter) proportional to that required by the RCSC *Specification*. Installation shall comply with all applicable requirements of the RCSC *Specification*, with modifications as required for the increased diameter and/or length to provide the design pretension." Table J3.1 of the *Specification* indicates that the minimum pretension is "equal to 0.70 times the minimum tensile strength of bolts, rounded off to nearest kip, as specified in ASTM specifications for A325 and A490 bolts with UNC threads." Beyond that, you will have to adapt from what is covered to what you have. Checking your method of pretensioning using samples of the bolts and a Skidmore will help you confirm your adaptation.

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