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steel interchange

Classifying Sections for Local Buckling

I am designing a wide-flange section in compression. I have chosen a W27×94 as a trial section. Table 1-1 indicates it is slender for compression. Table B4.1of the *Specification* indicates for an unstiffened element that $\lambda_r = 0.45\sqrt{(E/F_y)}$. For ASTM A992 steel this results in $\lambda_r = 9.89$. For a stiffened element the *Specification* indicates that $\lambda_r = 1.49\sqrt{(E/F_y)}$. For A992 steel this results in $\lambda_r = 32.8$. What I do not understand is what is meant by unstiffened and stiffened. The definition for unstiffened—"supported along one edge parallel to the direction of the compression force"—does not make sense to me. The section I am designing is 20 ft long and rigidly attached only at each end. There is no connection along either edge parallel to the direction of the compression force. Please provide guidance on the classification of this section as slender.

I think you are missing the intent. We are talking about the edges of the elements of the cross section of the shape—the web and flanges in this case.

Section B4.1 states: "For compression, sections are classified as nonslender element or slender-element sections. For a nonslender element section, the width-to-thickness ratios of its compression elements shall not exceed λ_r from Table B4.1a. If the width-to-thickness ratio of any compression element exceeds λ_r, the section is a slender-element section."

The *Specification* defines a slender-element section as, "Cross section possessing plate components of sufficient slenderness such that local buckling in the elastic range will occur."

So the intent is to check if any element of the section will buckle before the overall section buckles. If it does, then only a portion of the section is effective. Based on this, the classification of stiffened or unstiffened is related to the element, not the overall section as you indicate near the end of your question. Since the flange is supported only at one (unloaded) edge, at the web, it is unstiffened. Since the web is supported at both (unloaded) edges, at each flange, it is stiffened. In the case of the W27×94, the flange is nonslender, but the web is slender. The flanges will be fully effective, but the web will not.

The cases you refer to from Table B4.1a, Cases 3 and 8, address more general conditions. The flanges and web of a wide-flange section are check more appropriately using Cases 5 and 1.

Larry S. Muir, PE

More Classifying Sections for Local Buckling

What limiting width-thickness ratio for compression elements should I use for rectangular bars? My software uses $\lambda_r = 1.49\sqrt{(E/F_y)}$, which is indicated for a stiffened element. Why would a rectangular bar be considered a stiffened element instead of an unstiffened element?

This question comes up with some regularity. It would be interesting to have the software company explain how they arrived at their decision.

Let's first look at the definitions of stiffened and unstiffened elements.

Stiffened element: "Flat compression element with adjoining out-of-plane elements along both edges parallel to the direction of loading."

Unstiffened element: "Flat compression element with an adjoining out-of-plane element along one edge parallel to the direction of loading."

If you have only a rectangular plate which is free between the supports (which are also the loaded ends), then you have an element that is neither stiffened nor unstiffened since there is no out-of-plane element between the supports.

The *Specification* defines a slender-element section as, "Cross section possessing plate components of sufficient slenderness such that local buckling in the elastic range will occur." In essence a slender-element section is a section where some part of the section will buckle before the entire section buckles. This does not apply to a rectangular plate for which local and global buckling are the same. Depending on how you look at it, either you cannot have a slender element rectangular plate or you always have a slender element rectangular plate. Nonsense, right?

This discussion highlights how important it is for engineers to examine and understand the assumptions made in the software they are using. In this case the consequences of performing this check are likely minor since essentially a check is being performed that is unnecessary. It might result in a plate that is thicker than necessary and only a slight increase in overall cost. However, it also abuses basic structural mechanics and the intent of the *Specification*. Ultimately, this is why it is the engineer of record who is responsible for all design decisions.

Larry S. Muir, PE

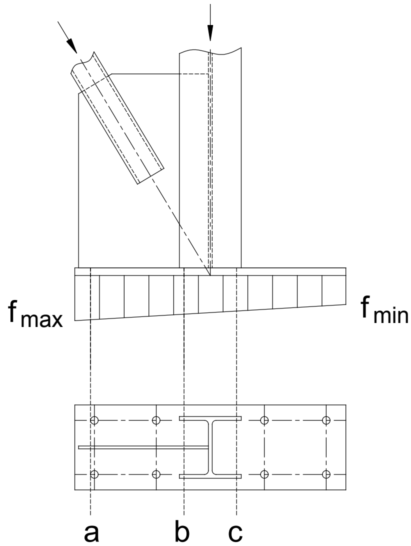
Base Plate Models

AISC Design Guide 1: *Base Plate and Anchor Rod Design* (a free download for members from www.aisc.org/dg) describes methods that can be used to design column base plates. How should these procedures be modified when the connection includes longitudinal plates such as stiffeners or gusset plates for bracing elements as shown above?

The base plate with gusset attached to the left side is subjected to axial compression loads in both the column and the brace. Assuming the triangular distribution solution from Appendix B in AISC Design Guide 1, a bearing pressure is generated below the entire plate with f_{max} and f_{min} at opposite ends. To check the bending strength of the plate due to bearing pressure, bending moments should be calculated about bending lines. For this connection, since there is bearing pressure on both sides of the column (no tension in anchors) we need to check the strength of the plate for both sides.

For the right side, where there base plate is not stiffened, the critical section is at "c".

The critical section for the left side is less clear. If I consider the section at "b," I am neglecting the stiffening effect of the gusset plate. Which is the critical section?



In practice, the presence of the gusset plate is typically neglected in the base plate design.

If the base plate is centered on the column, f_{max} and f_{min} will be equal. In this case, the right side of the base plate will control the design. However, this assumes an infinitely stiff base plate. Although this is a reasonable assumption for typical base plates, extremely long plate cantilever lengths can be a source of significant error caused by a highly nonlinear bearing pressure distribution. Using a uniform or linearly-varying bearing pressure across the full width will result in a conservative base plate thickness and a non-conservative concrete/grout bearing pressure. It may be more accurate to assume a shorter effective plate width in the calculations.

The effect of the gusset plate is typically neglected for the following reasons:

- If the gusset plate provides a stiff support to transfer loading into the base plate, the base plate bend lines will probably extend diagonally (actually curved) from near the column flange tip to near the gusset edge. However, due to the column web bending flexibility at the gusset-to-column interface, it is doubtful that a stiff support can be assumed.
- Presumably, the column design would typically be based on axial load alone. The assumption that the gusset plate alters the bending strength of the base plate would induce a moment in the column (which would be transferred through the gusset-to-column interface). This situation can be alleviated by using stiffeners on both sides of the column web.
- The gusset plate would need to be designed for the additional loads induced by the bearing pressure at the bottom edge of the gusset plate.
- The gusset interface connections, including the local strength of the column web, would need to be designed for the additional loads induced by the bearing pressure at the bottom edge of the gusset plate.

Neglecting the presence of the gusset plate is also consistent with the implicit assumptions made in Section 4.3 of AISC Design Guide 29: *Vertical Bracing Connections—Analysis and Design* (a free download for members from www.aisc.org/dg) where only shear is transferred at the base-plate-to-gusset connection.

If the gusset plate is considered in the base plate design, it is not clear where the critical section would occur. Ultimately, you must use your own judgment to determine what is appropriate for your situation. I have suggested a simple and common design model. More complex models are possible.

Bo Dowswell, PE, PhD

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