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Calculation of Weights

If I purchase steel plate as a raw material for a project and the customer is quoted a price per pound for the product, is he required to pay for the remaining skeleton of the steel plate if it is not useable in the project? Example: I purchase sheets that are 4 ft by 10 ft and burn two pieces that are 44 in. by 58 in. from each piece. I have a skeleton left over that is not useable. Does the customer pay for 40 sq. ft of material or only the weight of the two pieces?

The calculation of weight for this example is stated in Section 9.2.2(c) of the AISC *Code of Standard Practice* (a free download at www.aisc.org/code) as follows:

"When parts can be economically cut in multiples from material of larger dimensions, the weight shall be calculated on the basis of the theoretical rectangular dimensions of the material from which the parts are cut."

Kurt Gustafson, S.E., P.E.

Document Discrepancies

In case of a discrepancy between plans and specifications for buildings, which one governs?

The subject of document discrepancies is covered in Section 3.3 of the AISC *Code of Standard Practice for Steel Buildings and Bridges* (a free download at www.aisc.org/code) as follows:

"When discrepancies exist between the Design Drawings and Specifications, the Design Drawings shall govern."

Note that this section also states that any discrepancies that are discovered must be reported for resolution. It also states that it is not the responsibility of the construction team to discover discrepancies.

This may seem like a confusing answer, so let's go further. If a discrepancy is noted by the fabricator or detailer, it should be reported so that the design team can advise what information is correct and the work can be performed with the correct information. However, the fabricator, detailer, and others on the construction team are not expected to find discrepancies. Sometimes, the presence of a discrepancy only comes to light after a piece has been detailed, fabricated and/or erected. The quoted sentence provides a way to resolve if the work already performed has been performed properly, and who should pay for any re-work that is needed.

Kurt Gustafson, S.E., P.E.

Pipe Design

Section F8 of the 2005 AISC *Specification* addresses flexural design of round HSS. Can Section F8 be used for the flexural design of steel pipe?

Yes, it is common to do so, and the AISC *Specification* explicitly includes steel pipe complying with ASTM A53 Gr. B. The Glossary of the 2005 AISC *Specification* defines HSS as a square, rectangular or round hollow structural section produced in accordance with a pipe or tubing product specification. Section A3.1a(3) of the *Specification* lists pipe as meeting the ASTM A53/A53M, Gr. B standard.

Kurt Gustafson, S.E., P.E.

Allowable Stresses in 1967

What was the allowable stress for A36 steel, fabricated in 1967?

Like today, the allowable stresses in 1967 were based on the limit state being investigated. F_y for ASTM A36 steel was and still is 36 ksi. Some example cases are as follows:

Flexure

The allowable strong-axis bending stress for a compact shape braced at a small enough interval to preclude lateral-torsional buckling was:

$$F_b = 0.66F_y = 23.8 \text{ ksi (use of 24 ksi was common)}$$

The allowable strong-axis bending stress for a non-compact shape braced at a small enough interval to preclude lateral-torsional buckling was:

$$F_b = 0.60F_y = 21.7 \text{ ksi (use of 22 ksi was common)}$$

The allowable bending stress for bracing at larger intervals was lower than these values.

Compression

The allowable axial compression stress was based on the slenderness ratio with a maximum of $F_a = 0.60F_y$. The actual allowable was much lower for any typical column length, of course.

Tension

The allowable tension on the net section, except at pin holes, was

$$F_t = 0.60F_y.$$

See the 1963 AISC *Specification* for further information.

Kurt Gustafson, S.E., P.E.

Large Bolted Connections

We currently have a job with 1¼-in. diameter bolts (approximately 50 per connection) with 2 plies of 3-in.-thick steel. The hole size specified is 1⁵⁄₁₆ in. Needless to say all of the holes do not exactly line up perfect. What are the dimensional tolerances for the locations of holes in large bolted connections?

Neither the AISC *Code of Standard Practice* nor the AISC *Specification* provides tolerances on the locations of bolt holes. The holes however must be placed such that the other tolerances given in the AISC *Code of Standard Practice* can be maintained and the bolts can be installed in the holes. That is, the only requirement is that the joint must fit up, and it is up to the fabricator to employ a method that will achieve this. Some suggestions for how to do this follow.

When dealing with thick plates, consideration must be given to the use of oversized holes and slip-critical connections. The use of slip-critical connections will usually require more bolts. This may be detrimental to economy in both the shop and the field, due to the greater number of holes to be drilled, and bolts to be installed. There will also be additional cost involved in surface preparations and inspections.

The use of bearing bolts in standard holes will mean fewer holes to drill and bolts to install, but may require reaming if things do not fit-up. This reaming can be both time-consuming and costly in the field.

Larry S. Muir, P.E.

steel interchange

High-Seismic Column Splice

Section 8.4a(2) of AISC 341 requires the available strength for each flange (LRFD) is noted as $0.5R_yF_yA_f$. Is the term A_f the area of one flange or the total area of the two flanges?

The term A_f refers to the area of one flange of the smaller column connected.

Thanks for your question—this has been clarified in the draft of the 2010 Seismic Provisions where the term A_f is proposed to be replaced by the term $b_f t_f$, which is defined as the area of one flange of the smaller column connected.

Kurt Gustafson, S.E., P.E.

Steel Properties at Elevated Temperatures

What is the reasoning between the different material ratios vs. temperature (i.e. Modulus of Elasticity vs. Temperature & Yield Strength vs. Temperature) given in the AISC 13th ed. Table A-4.2.1 versus the graphs (Figures 3.2 and 3.3) published in ASCE, *The Structural Design of Air and Gas Ducts for Power Stations and Industrial Boiler Applications*? When comparing the ratios for yield strength, AISC gives a reduction beginning at 800 °F (i.e. 0.94), and the ASCE publication shows almost a linear reduction in yield strength beginning at 100 °F. Conversely, when comparing the ratios for Modulus of Elasticity, the AISC ratios drop much faster than what is given by ASCE. I would appreciate your help in understanding the discrepancies between these reported values.

The properties in Appendix 4 of the AISC *Specification* are deemed suitable for the ultimate state (pre-collapse) conditions of structures exposed to severe fires. Such conditions involve very high levels of thermal and structural strains, very large deformations, and implicitly assume irreparable damage. Therefore, the associated properties are not suitable for the design of in-service ducts at elevated temperatures.

There is more than one way to test steel for mechanical properties at elevated temperatures. Even from the same set of tests at elevated temperatures, there are several ways to derive mechanical properties. The steel properties at elevated temperatures reported in the literature often vary considerably due to these variations in testing and derivation methods.

The yield strength values/ratios in Appendix 4 of the AISC *Specification* could be associated with stress at 2% strain (this is quite different from the usual 0.2% offset method), and they are essentially equal to the ultimate/tensile strength values/ratios (note that ultimate strength ratios are normalized by the yield strength in the *Specification*) at 750 °F and higher temperatures.

Extra conservatism of elasticity modulus values/ratios in the AISC *Specification* follows from concerns about column stability. Part of the difference may also be due to differences in testing and derivation methods.

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U-Factor in 1989 Column Tables

How was the factor *U* that was tabulated in the 9th edition ASD *Manual* column tables calculated? This factor is used to determine an equivalent axial load for beam-columns.

The *U* value for beam-columns published in the 9th edition column tables was a hold-over from that factor first shown in the 8th edition *Manual*. Unfortunately, I could find no explanation in either manual as to how these values were derived. I did some searching in my library of old publications, however, and found a derivation in my U.S. Steel *Column Design Curve* book from 1969 that gave the Equation $U = 0.66S_x/0.75S_y$. This is the ratio of the strong-axis to weak-axis allowable bending stresses for a compact shape. I checked a few numbers in the 9th edition and found them accurate for the compact shapes. The Equation $U = 0.60S_x/0.75S_y$ was used for non-compact shapes.

Kurt Gustafson, S.E., P.E.

High-Seismic Column Splice Location

AISC 341 requires column splices to be no closer than 4 ft, 0 in. from beam to column connection. What is the basis of this requirement? Can the splices be closer than 4 ft, 0 in. if complete-joint-penetration groove welds are used?

This 4-ft requirement is intended to keep the column splice away from the beam/column intersection, and locate it closer to the point of inflection between stories. The OSHA requirement for safety erection of exterior columns also influenced the 4-ft dimension.

In the draft of the 2010 AISC *Seismic Provisions*, it is proposed to relax this requirement for columns that are spliced with CJP groove welds. In such cases, it is proposed that the splice be permitted to be located closer to the beam-to-column flange connections, but not less than the depth of the column. Note that the OSHA requirements for fall protection still must be met, and if the column is not extended up high enough, another means of attaching the perimeter cables must be provided.

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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