Questions from a Non-Engineer

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Every industry has its own jargon and implicit assumptions—and the structural engineering community is no different. One curious reader sent us his questions about some of the specifics of steel design and construction. Here are our answers.

MOMENT CONNECTIONS

Why are moment connections called moment connections?

Great question. The general definition of "moment" is the tendency to produce a turning or rotation of a body on which a force acts, such as the tendency of a beam and column to rotate relative to each other when a building sways due to wind or an earthquake. A moment connection is a connection designed to force the beam and column to rotate together at the joint, thereby keeping the angle between them unchanged. This activates bending resistance in the beams and columns to resist the wind or earthquake without the need for bracing.

What are some of the considerations in the design of a moment connection?

The basic considerations are the amount of rotation that the joint will undergo and the amount of moment that the joint will be required to resist as the load is applied.

What are some of the big "no-nos" in their use, in welding them, and in working with them?

A common type of welded moment connection is to have complete-joint-penetration groove welds of the beam flanges to a column flange and to have a "simple" connection to the beam web. One "no-no" in such a connection is to detail the joint to require overhead welding. Another is to expect that proper inspection of such a joint can be accomplished "after the fact." Another "no-no" is assuming that only a directly welded flange type detail is useful. In many applications, flange plated connections and moment end-plate connections can be used and may be preferred.

CAMBERED BEAMS

How is the camber of a beam determined?

There are two types of camber: one is the natural rolling tolerance that may occur in a structural shape as it is produced at the mill; the other is an intentionally induced camber that is specified in attempt to offset some anticipated portion of load deflections. Permissible rolling tolerances for camber are stipulated in ASTM A6—*Standard Specification for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*. Induced camber is specified by the project designer and will likely be based on some portion of the anticipated self-weight of the slab/beam system as the concrete is cast in attempt to achieve a relatively level slab at that time. Fabricators typically induce camber through brute force: the two ends of a beam are held in place and pressure is applied with two hydraulic rams in the span of the member. According to the AISC *Code of Standard Practice*, camber is measured in the shop and not in the field where shipping and installation may affect the camber.

WELDING

Who determines which jobs require an AWS Certified Welding Inspector (CWI)?

On the building side, there is no specific requirement for a CWI. AISC Certified Fabricators working on buildings need to have either a CWI or the equivalent on staff (while bridge fabricators are specifically required to have a CWI). Fabricators can demonstrate that their weld inspector is equivalent to a CWI through resumes, work histories, and training records that show experience, education, or training in both fabrication and inspection methods. Acceptable training providers may be any one of the following: a nationally recognized provider; a qualified person or school in the fabricator's local area; or an experienced person employed by the fabricator.

In addition, the current IBC *Model Building Code* has instituted requirements that certain projects (almost everything that involves a design professional) have a certain level of weld inspection, and that the basis for weld inspection and welding inspector qualification be in accordance with AWS D1.1.

Why would non-CWIs be assigned to inspect steel projects?

Acceptable qualifications include: (1) AWS CWI, (2) Canadian Welding Bureau (CWB), or (3) appropriate training and experience (see the answer to the previous question).

What are some good references for non-engineers to study in order to strengthen their backgrounds and become more valuable to engineers?

Everyone involved in design and construction should read *Why Buildings Stand Up: The Strength of Architecture* by Mario Salvadori; *Why Buildings Fall Down: How Structures Fail* by Matthys Levy, Mario Salvadori, and Kevin Woest; *The Art of Construction: Projects and Principles for Beginning Engineers and Architects* by Mario Salvadori, Sarlinda Hooker, and Christopher Ragus; and *Structures: Or Why Things Don't Fall Down* by J.E. Gordon. All of them are readily available for less than $15 each (and even less if you don't mind used copies).

What can CWIs do to make themselves more marketable for structural jobs?

Probably one of the most important things to consider in improving marketability is to increase the scope of useable services. AISC and AWS are scheduled to introduce a Steel Structures Inspector program next year. This program will qualify you beyond welding and include other aspects of construction, including bolting, erection, joists, and deck.

How much are structural engineers involved in amendments and revisions to the AWS Code?

The AWS Code process is ANSI-accredited, which means that representatives from all interested parties, including structural engineers, participate. The committee has 150 members and a significant number are structural engineers.

GENERAL

What considerations go into choosing a certain ASTM grade of steel for construction?

There are many considerations in selecting not only a grade, but also a type of steel member (wide-flange or hollow structural section, beam or truss, etc.). Among these may be strength, serviceability (stiffness/section properties), availability, whether the steel will be exposed after construction, and, of course, cost. It is usually most cost effective to use the base grade for the given shape being designed. For example, ASTM A992 is the most common steel for hot-rolled structural W-shapes in today's market for steel building construction. Other shapes have different base grades, as listed in the AISC *Steel Construction Manual*. When cost is justified and availability is confirmed, grades other than the base grade can be specified.

What is the LRFD manual and what does it contain?

LRFD stands for load and resistance factor design. It is a structural engineering design approach that looks at the likelihood of a particular load (that's the load factor part) and the reliability of the strength of the structure (that's the resistance factor part). In ASD (allowable stress design), both of these factors were lumped together in the factor of safety.

With the release of the 2005 AISC *Specification for Structural Steel Buildings*, LRFD and ASD became essentially equivalent methods, differing only in which set of load combinations the designer selects (either LRFD load combinations for LRFD or ASD load combinations for ASD). Once this choice is made, the resistance factor is used to determine strength for LRFD or the safety factor is used to determine strength for ASD.

Explain "tree column" and "point column."

Though neither term has a technical definition, the term "tree column" is typically used to describe a column shaft that is shop-fabricated with parts of the beams attached at one or more levels. The shop-fabricated assembly looks somewhat like a tree with the column as the trunk, and the attached beam stubs as branches. The assembly is erected by connecting assemblies one on top of another and by adding beams between the beam stub "branches" of adjacent assemblies.

The term "point column" is a lot more vague, but is occasionally used to describe a column that does not extend the full height of a structure (such as an intermediate column between two floors) or a column that stops short of the roof to accommodate drainage.

What are some of the considerations that go into the size and spacing of web-to-flange stiffener requirements?

Stiffeners may be added to a beam or column to reinforce a section that is subjected to a concentrated force, such as a concentrated load on top of a beam flange or at bearing points of a beam or girder. Stiffeners may also be used to anchor a beam into the floor slab above to create torsional stiffness and a braced point along the length of the beam. Stiffeners may also be used to reinforce column sections if required to resist concentrated forces resulting from the rigidity of a (moment) connection. Even with the current higher material pricing, it is often more economical to increase a member size rather than to use stiffeners.

Stiffeners are also used to reinforce webs for shear. You will often see these used in deep plate girders with relatively thin webs. Sometimes diagonal stiffeners are used in a column web adjacent to rigid connections, also to reinforce the web for excess panel shear.

Given a span dimension between columns, what determines the size of the connecting girder?

The size of the horizontal spanning member is likely sized for gravity loads based on the magnitude of the applied loads (strength) and the serviceability (vertical deflection) limitations to be accommodated. If the horizontal member is also part of a lateral force resisting frame, such as using moment connections, the girder stiffness required to help provide frame action may also be a factor in determining the member size.

Tell me more about the AISC *Specifications for the Qualification of Steel Structures Inspectors*?

The document is in the final review stages and should be published early in 2006.

Concerning complete-joint-penetration and partial-joint-penetration groove welds, what consideration goes into the decision on which one is used?

All types of welds, whether fillet welds, partial-joint-penetration groove welds, or complete-joint-penetration groove welds, are typically designed based on the level of load to be transferred through the weld. There are some exceptions to this. For example, seismic design criteria also have ductility requirements that must be met. As a rule of thumb, partial-joint-penetration groove welds are less expensive than complete-joint-penetration groove welds and are therefore used unless they either provide inadequate strength—or a complete-joint-penetration groove weld is specified.

On some jobs, I have seen welds not allowed to wrap the ends of gussets, stopping one inch short from the end. I've seen others that required welds to wrap around the end. What is the difference?

The difference on the particular job may be the requirements stipulated on the contract documents. Very often the "weld all around" symbol is used when it is not really necessary for a stress transfer requirement. AWS does not specifically stipulate that a weld should either be wrapped or stopped, unless it is required to allow for the flexibility assumed in the connection design or if it is in a lap joint subjected to tension. There is also an AWS stipulation that fillet welds on opposite side of a common plane shall be interrupted at the corner common to both welds. See AWS D1.4-2004, Section 2.8.3.5.

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