IF YOU'VE EVER ASKED YOURSELF "WHY?" about something related to structural steel design or construction, *Modern Steel Construction*'s monthly Steel Interchange column is for you! Send your questions or comments to solutions@aisc.org.

Historic Lattice Columns

In the December 2007 Steel Interchange, there was a question pertaining to lattice columns. Ted Galambos, Ph.D., was kind enough to add his expertise on the subject:

You can find information about these historic types of columns in Section 3.4 of the 5th edition of the *SSRC Guide*, as well as in Section 2.18 of Timoshenko and Gere's *Elastic Stability*. In such columns, one needs to consider the effect of the reduction of the stiffness because of shear. Ignoring this effect is very serious and it was one of the causes of the first collapse of the Quebec Bridge in 1905.

Flexural Strength Comparisons

The elastic moment strength of a beam listed in Table 3-6 of the 13th edition AISC manual seems low as compared to the 9th edition ASD manual and the 3rd edition LRFD manual. As an example, for a W16×77 with F_y = 50 ksi:

13th edition, Table 3-6, pp. 3-60
ASD: M_r/Ω = 234 kip-ft
LRFD: ϕM_r = 352 kip-ft

9th edition ASD, pp. 2-11
ASD: M_R = 369 kip-ft

3rd edition LRFD, Table 5-4, pp. 5-62
LRFD: ϕM_r = 405 kip-ft

Could you tell me why there are such differences?

The nomenclature is somewhat different between the manuals. In the 2005 specification, M_r is the moment capacity when L_b = L_r, the unbraced length at which the shape transitions from the inelastic to the elastic lateral torsional buckling range.

In the 9th edition manual, the M_R that is listed in the Part 2 Table is the beam resisting moment where F_b = 0.66F_r. For the difference between the 9th and 13th edition ASD procedure, you would really want to compare the M_p/Ω (374 kip-ft) in the 13th edition tables with the M_R (369 kip-ft) value in the 9th edition tables.

The change between the 3rd edition LRFD and the 13th edition LRFD capacities is based on different equations of the buckling curves given in the two specifications. In the 3rd edition table, you will note that L_r was 25.3 ft. for the W16×77. In the 13th edition table, you will note that L_r is 27.8 ft.

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

Multiple Cranes in Runway

I am designing a building with three top-running bridge cranes. The cranes all run in the one aisle and are separated from each other by 20 in. The runway beams will be a simple span. Would I need to design the runway beam and building frame for vertical and horizontal loads from all three cranes being adjacent to each other and being fully loaded at the same time? The MBMA design manual says to design for the single crane producing the most unfavorable effect and for the loads of two adjacent cranes producing the most severe effect. Is this due to a low probability of the cranes all being fully loaded at the same time?

The following advice pertaining to crane design parameters was provided by John A. Rolfes, P.E., S.E., of Computerized Structural Design, S.C.:

With regard to the effect of multiple cranes in one aisle (or in multiple aisles), no direction is provided in the building code. The appropriate combination of loadings is dependent upon the specific application and the judgment of the designer. The MBMA manual provides the recommendation cited by the questioner. AISE Technical Report No. 13 recommends three different load combinations (for crane loads). These are as follows:

1. For fatigue design considerations: maximum vertical loads from one crane, including vertical impact plus side thrust forces from one crane at 50% of maximum predicted values.
2. For strength design consideration: maximum vertical loads from one crane, including vertical impact plus side thrust forces from one crane at 100% of maximum predicted values.
3. For strength design consideration: maximum vertical loads (without impact) from the full number of cranes that may impart loads to the particular element of the structure being designed plus side thrust from one crane at 100% of maximum predicted values. (It is also not uncommon to consider 50% side thrust from multiple cranes acting simultaneously).

Disparities between the maximum loads from load cases 2 and 3 above are typically more pronounced in buildings with large bays. For all of these load combinations, the crane(s) must be positioned to generate the worst loading effect for the member being designed. The designer should try to understand the various operations that the crane is going to be used for and what the likelihood is for cranes to be spaced close together and fully loaded. We have worked on numerous projects where multiple cranes in an aisle are used simultaneously with a common lift beam to lift loads that exceed the capacity of each crane individually. This type of application would be covered by the use of load combination 3 above.

Evaluation of an Existing Structure

If a building was built in the early 1900s, can we utilize LRFD design to check the existing steel beams?

You can use the current AISC specification, either ASD or LRFD, when investigating older structures, as long as you stay consistent in the load approach for load assumptions and capacity parameters. See Appendix 5 of the AISC specification (a free download at **www.aisc.org/2005spec**) for further discussion. When a local jurisdiction specifically requires a version of the AISC specification that precedes the most current version, we recommend asking the authority having jurisdiction for approval to use the more current version.

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

RBS Moment Connections

1. When the cuts for an RBS are determined by *a*, *b*, and *c* (identified in AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*), does this cut section need to develop the maximum moment as required for that member?

2. Bracing as per AISC 358-05, Section 5.3.1 ,(7) is required for RBS beams. If there is a steel decking floor system with shear studs on top of the RBS beam, can this flooring system be considered an adequate form of bracing for this beam?

1. If the RBS is being used as a prequalified connection (as in AISC 358), then yes, the section has to develop its own plastic moment capacity. This moment capacity is, of course, smaller than the plastic moment capacity of the beam where the section is not reduced.
2. As per section 5.3.1 (7) Exception, supplemental bracing is not required if there is concrete structural slab (including concrete on deck) if the conditions in the exception are met. Please note that you cannot have shear studs installed in the protected zone above the RBS unless you have a tested assembly that includes them in this zone.

Amanuel Gebremeskel, P.E.

Backing Bar Thickness

Is there an industry standard for the size (thickness) of backing bars used in moment connections?

Section 5.10.3 of AWS D1.1 provides a table of recommended minimum nominal thickness of backing bars. The thicknesses in that table depend on the weld process.

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

Flexure of Single Angles

Is there an accepted procedure for the design of a steel angle in bending supporting a uniform load? The compression leg is upward and unrestrained, while the load is seated on the other (horizontal) leg.

Please refer to section F10 of the 2005 AISC specification (a free download at **www.aisc.org/2005spec**) to see how to design such an angle. Don't forget to check the vertical leg for buckling according to F10.3.

Amanuel Gebremeskel, P.E.

Edge Distance for Anchor-Rod Holes in Base Plates

I am trying to find a table or equation that gives the minimum/maximum bolt spacing required for the base plate, as well as the minimum edge distances. Any suggestions of where to look in the AISC manual or elsewhere would be much appreciated.

You will not find tables of anchor rod spacing or edge distance requirements for holes in base plates in the AISC manual. Such requirements result from the design process, base layout, and the intended function of the anchor rod in the base anchorage system.

For example, the AISC recommendation is that anchor rods should not be used to resist shear. If shear is not required to be resisted by the anchor rod, there is no specific required edge distance from the rod to the edge of the base plate. See FAQ 7.1.7 on the web site at **www.aisc.org/faq** for discussion on this subject. However, if engineers do assume that shear is resisted by the anchor rods, the bearing strength at the edge will need to be checked by the EOR.

Spacing of the anchor rods is a similar function of the design process as to what type of resistance the rod is intended to provide. There is an OSHA requirement of a minimum of four anchor rods for a column. If the rod is intended to resist tension forces, the cone development of the embedment will likely influence the spacing.

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

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Kurt Gustafson is the director of technical assistance and Amanuel Gebremeskel is a senior engineer in AISC's Steel Solutions Center. Charlie Carter is AISC's chief structural engineer, and Lou Geschwindner is AISC's vice president of engineering and research.

Steel Interchange is a forum to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

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:

Steel Solutions Center
One East Wacker Dr., Suite 700
Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.803.4709
solutions@aisc.org