STEEL INTERCHANGE

Steel Interchange is an open forum for *Modern Steel Construction* readers 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.

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

If you have a question or problem that your fellow readers might help you to 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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MOMENT FRAME/SEISMIC

I am preparing to design a four-story moment frame office building on the east coast (seismic zone 0) and was wondering what other folks have been doing. What are some other engineers in this area requiring? What is standard practice?

Question sent to www.seaint.org

It all depends upon the *R* factor you select (or are permitted to select in the applicable building code). If the soil is poor or the importance factor drives you to a higher seismic performance category, you may have to use a system from the AISC *Seismic Provisions* (SMF, IMF, OMF). Otherwise, you will likely be permitted to choose $R = 3$ and design and construct the building according to the requirements in the AISC *Specification for Structural Steel Buildings* without the additional requirements in the AISC *Seismic Provisions*. If you choose to take *R* greater than 3, you must meet the requirements that correspond to the higher *R* in the AISC *Seismic Provisions*.

Usually, $R = 3$ systems are less expensive than systems with higher *R* factors. $R = 3$ as described above gets you a system of normal ductility that will remain nominally elastic for the design seismic forces. For moment frames, this can be achieved using the basic moment connections shown in the AISC *Manual*, such as flange plates, end plates, and welded flanges. You can also consider flexible moment connections and PR moment connections if you wish.

Charles Carter, S.E., P.E.
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WELDING THROUGH PAINT

Is there an AISC spec for painting at moment connections or is our erector just complaining about his welders having to burn through paint to make field welded moment connections?

Question sent to the steel detailers list server at Yahoo Groups (http://groups.yahoo.com/group/steel-detail/)

The 1999 *LRFD Specification*, Sections M3.5 and M4.5 contain the following provisions:

Section M3.5: Unless otherwise specified in the design documents, surfaces within two inches of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

Section M4.5: Shop paint on surfaces adjacent to joints to be field welded shall be wire brushed if necessary to assure weld quality...

The complaint from the erector is probably due to the fume hazard on the workers, depending upon proximity and actual fume generation. Another concern is the impact on weld quality when welding through paint. Usually, paint is no match for welding, however, you will need to confirm this with the electrode manufacturer.

The 1999 *LRFD Specification* is available as a free download from the AISC web site at www.aisc.org/lrfdspec.

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STEEL DECK DETAILS

from July 2002 Steel Interchange

Does anyone know what to do when you have a moment connection with a thick plate on the top of the upper beam flange welded to it and to the column and you have to place the steel deck? Do I place the steel deck on top of it (it doesn t look good from the lower floor or level) or do I cut the steel deck around the perimeter of the plate so that the deck would rest on the upper flange level of the beam?

Question sent to www.seaint.org

The metal deck requires continuous bearing perpendicular to its direction and it may not be distorted or warped by force to accommodate an out-of-plane arrangement. Accordingly, the deck should bear at the base of the thick plate. If necessary, bearing plates should be welded to the side(s) of the beam flange to facilitate proper bearing conditions. In a composite floor deck, cell closures and other accessories should be used around the perimeter of the affected area to contain the concrete during construction.

Isaac Gordon, P.E.

STEEL INTERCHANGE

LACING COLUMN DESIGN

May 2002 Steel Interchange

Please provide any additional sources of information relative to AISC *Specification* Section E4 (ASD *Manual*, ninth edition, pages 5-43 and 5-44) covering the proportioning of lacing members to resist a 2% shear stress. Additional references with discussions, the history of its origin, derivation and/or examples of the proper application of this specification would be greatly appreciated.

W.H. Parker

If a column is perfectly straight and the load concentrically applied, there would be no moment or shear in the column due to this load. In reality, every column deviates from the straight condition. There is a varying moment in the column, which depends upon the deviation of the column from a straight line. Since the moment varies, shear must be present. If a column consists of structural members connected by lacing bars, these lacing bars must resist the shear. The amount of shear is not certain because the deviation of a column is not certain. The reference by Salmon shows that with the usual imperfections the shear will be at least 1 percent of the load. Moore and Talbot found from experiment that the shear varied from 1 to 3 percent of the load.

Williams-Harris give an excellent example illustrating the design of lacing. Their example uses an old Railroad Code for allowable stresses, but the method of analysis is still valid. Both Williams-Harris and Roark-Young present the following formula for the transverse shear:

$$V = 0.01P[100/(s+10) + 0.01s]$$

V is the shear, P the column load and s is the slenderness ratio l/r. If $s = 50$, $V = 0.0217P$. If $s = 120$, $V = 0.0197P$ Both round off to $0.02P$, showing that V is not sensitive to s.

REFERENCES

Roark, R.J. and Young, W.C. *Formulas for Stress and Strain*, fifth edition. McGraw-Hill, 1975.

Salmon, E.H. *Columns*. Oxford Technical Publications, Henry Frowde and Hodder & Stoughton, 1921.

Talbot, A.N. and H.F. Moore. Tests on Built-up Steel and Wrought Iron Compression Pieces, *Transactions of the American Society of Civil Engineers*, Vol. 65, p. 202, 1909. (Also University of Illinois Engineering Experiment Station, Bulletin 44).

Williams, Clifford D. and Harris, Ernest C. *Structural Design in Metals*, second edition. Roland Press, 1957.

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NEW QUESTIONS

PRETENSIONING ANCHOR RODS

ASTM F1554 Grade 105 anchor rods will be used in a costal area with wind gusts of 130 mph located in a very high seismic zone. We are planning to pretension the anchor rods to avoid the risk of inducing tensile fatigue from loading cycles resulting from wind loads. Do you agree with this as being a valid reason to pretension these rods?

Question sent to AISC s Steel Solution Center

CIRCULAR BASE PLATE DESIGN

I would like to design circular column base plates. However, there appears to be little or no information on the subject. Does anyone know of papers, articles or design guidelines for the design of circular base plates?

Question sent to AISC s Steel Solution Center

PR CONNECTIONS

Is it appropriate, as stated by Ackroyd in his 1990 Engineering Journal paper, to use a representative connection stiffness equal to 50% of the initial connection stiffness for distribution of the flexible end moments for design? Are there any recommendations for an appropriate percentage of initial connection stiffness to be used when performing drift calculations for PR frames?

Question sent to AISC s Steel Solution Center