PROPOSAL 8-50R (2008)

SCOPE: Section 12.2.5 of the *2008 Provisions*

PROPOSAL FOR CHANGE:

Revise Section 12.2.5 as follows:

12.2.5 System Specific Requirements. The structural framing system shall also comply with the following system specific requirements of this section.

No change proposed for Sections 12.2.5.1 thru 12.2.5.3

12.2.5.4 Increased ~~Building~~ Height Limit for Steel Braced Frames and Special Reinforced Concrete Shear Walls.

12.2.5.4.1 Special Steel Concentrically Braced Frames, Steel Eccentrically Braced, Steel Buckling-Restrained Braced Frames and Special Reinforced Concrete Shear Walls. The height limits in Table 12.2-1 are permitted to be increased from 160 ft (50 m) to 240 ft (75 m) for structures assigned to Seismic Design Categories D or E and from 100 ft (30 m) to 160 ft (50 m) for structures assigned to Seismic Design Category F that have special steel concentrically braced frames, steel eccentrically braced frames, steel buckling-restrained braced frames or special reinforced concrete cast-in-place shear walls and that meet both of the following requirements:

1. The structure shall not have an extreme torsional irregularity as defined in Table 12.2-1 (horizontal structural irregularity Type 1b).

2. The braced frames or shear walls in any independent line of resistance ~~one plane~~ shall resist no more than 60 percent of the total seismic forces in each direction, neglecting accidental torsional effects.

12.2.5.4.2 Ordinary Steel Concentrically Braced Frames. The height limit in Table 12.2-1 are permitted to be increased to 160 ft (50 m) for structures assigned to Seismic Design Categories D or E and to 100 ft (30 m) for structures assigned to Seismic Design Category F that have ordinary steel concentrically braced frames conforming to the detailing provisions for ordinary concentrically braced frames specified in AISC 341 provided the Response Modification Coefficient, R, is taken as 0.67, the System Overstrength Factor, Ω_o, is taken as 1.5 and the Deflection Amplification Factor, C_d, is taken as 0.67.

No change proposed for Section 12.2.5.5

12.2.5.6 Increased Height Limits for Single-Story Steel Ordinary and Intermediate Moment Frames in Structures Assigned to Seismic Design Category D or E. Single-story steel ordinary moment frames and intermediate moment frames in structures assigned to Seismic Design Category D or E are permitted up to a height of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m2). In addition, the dead loads tributary to the moment frame, of the exterior wall more than 35 ft above the base shall not exceed 20 psf (0.96 kN/m2).

12.2.5.7 Increased Height Limits for Multi-Story~~Other~~ Steel Ordinary and Intermediate Moment Frames in Structures Assigned to Seismic Design Category D or E.

12.2.5.7.1 Steel Ordinary Moment Frames within Light-Frame Construction. Steel ordinary moment frames in structures assigned to Seismic Design Category D or E not meeting the limitations set forth in Section 12.2.5.6 are permitted within light-frame construction up to a height of 35 ft (10.6 m) where neither the roof nor the floor dead load supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m2). In addition, the dead load of the exterior walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m2).

12.2.5.7.2 Steel Intermediate Moment Frames. Steel intermediate moment frames in structures assigned to Seismic Design Category D or E not meeting the limitations set forth in Section 12.2.5.6 are permitted as follows:

1. In Seismic Design Category D, intermediate moment frames are permitted to a height of 35 ft (10.6 m).
2. In Seismic Design Category E, intermediate moment frames are permitted to a height of 35 ft (10.6 m) provided neither the roof nor the floor dead load supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m2). In addition, the dead load of the exterior walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m2).

12.2.5.7.3 Ordinary Steel Moment Frames. The height limit in Table 12.2-1 are permitted to be increased to 160 ft (50 m) for structures assigned to Seismic Design Categories D or E that have ordinary steel moment frames that conform to the detailing requirements of AISC 341 including truss type systems that behave as moment frames that conform to design requirements for ordinary steel concentrically braced frames of AISC 341 provided the Response Modification Coefficient, R, is taken as 0.67, the System Overstrength Factor, Ω_o, is taken as 1.5 and the Deflection Amplification Factor, C_d, is taken as 0.67.

12.2.5.8 Increased Height Limit for Single-Story Steel Ordinary and Intermediate Moment Frames in Structures Assigned to Seismic Design Category F. Single-story steel ordinary moment frames and intermediate moment frames in structures assigned to Seismic Design Category F are permitted up to a height of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m2). In addition, the dead loads of the exterior walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m2).

12.2.5.9 Increased Height Limit for Multi-Story~~Other~~ Steel Ordinary and Intermediate Moment Frames ~~Limitations~~ in Structures Assigned to Seismic Design Category F.

12.2.5.9.1 Steel Intermediate Moment Frames within Light-Frame Construction. Steel intermediate moment frames in structures assigned to Seismic Design Category F not meeting the limitations set forth in Section 12.2.5.6 are permitted within light-frame construction up to a height of 35 ft (10.6 m) where neither the roof nor the floor dead load supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m2). In addition, the dead load of the exterior walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m2).~~In addition to the limitations for steel intermediate moment frames in structures assigned to Seismic Design Category E as set forth in Section 12.2.5.7~~.1~~, steel intermediate moment frames in structures assigned to Seismic Design Category F are permitted in light-frame construction.~~

12.2.5.9.2 Ordinary Steel Moment Frames. For ordinary steel moment frames assigned to Seismic Design Category F including steel truss moment frames that conform to the detailing provisions for ordinary concentrically braced frames specified in AISC 341, the height limit in Table 12.2-1 is permitted to be increased to 100 ft (30 m) provided the Response Modification Coefficient, R, is taken as 0.67, the System Overstrength Factor, Ω_o, is taken as 1.5 and the Deflection Amplification Factor, C_d, is taken as 0.67.

No change proposed for Section 12.2.5.10

REASON FOR PROPOSAL:

This change does two things. First it provides editorial cleanup by properly identifying the contents of the subsections and breaking up the subsections into logical sub-subsections. Second it introduces the concept of allowing ordinary steel moment frames and braced frames to have higher height limit than currently permitted provided they are designed to stay elastic for the Maximum Considered Earthquake level ground motions (R = 0.67). It should be noted that omega zero values were selected in consultation with the chairs of TS-2 and TS-8. The C_d value is taken equal to the R value based on the equal displacement rule. It should be noted that the both ordinary systems are required to conform to the detailing requirements of AISC 341 even though the value of R is less than 3. Because of the detailing requirements and the high likelyhood that the nominal member and connection strength is likely to be in the range of 1.5 times the minimum design strength, this is believed that this an extremely conservative set of design parameters.

This proposal went through some preliminary reviews by TS-2, TS-6 and TS-8. It should be recalled that TS-8 members is a marriage of the old TS-8 and TS-13 Technical Subcommittees. When originally proposed, it was intended to have a separate definition for industrial buildings that could utilize Chapter 15 design values and height limits. When this was originally proposed to TS-8, the previous TS-13 members fully supported while the old TS-8 members (who primarily have a building structure background) opposed. Old TS-8 members preferred the change apply to all buildings. It should be noted that some in TS-2 suggested a separate definition in Chapter 15 for industrial buildings. Unfortunately, because of the current setup of the subcommittee membership, this approach is not workable since we could not get that type of proposal out of subcommittee.

There were some other suggestions that we should include in a tabular format such as that provided in Chapter 15 that show different height limits with different design value options. We consider that to be an editorial comment that can be handled by the ASCE 7 Seismic Subcommittee if this proposal is approved.

There were those on TS-2 who wanted more analytical and testing backup (especially ATC-63 studies) to justify the values provided. We do not have the resources to provide such backup so instead we decided to reduce the design parameters to such low values that analytical backup would not be required and judgment would be sufficient. These values were selected in consultation of the chairs of TS-2 and TS-6. It should be that steel OCBFs and steel OMFs are not new systems. They have existed for a long time. Below is a history of the height limits and restrictions for these systems in the NEHRP Provisions since 1994. In no cases were analytical studies provided to justify the changes in the height limits for these ordinary systems. Indeed the commentary provides no quantifiable justification for the changes in these limits other than the judgment of TS-2 and TS-6. What was stated was these systems have less ductility than special systems. We do not argue with that assessment and that is the reason they have much lower R values. However we do argue with the height limits. The detailing requirements for these systems has advanced much since 1994 and the R values have been greatly reduced but height limits have also reduced greatly in SDC D, E and F without much justification (except judgment) in our opinion.

	OCBFs				OMFs			
	R	SDC D	SDC E	SDC F	R	SDC D	SDC E	SDC F
1994 NEHRP	5	160 ft	100 ft	(100 ft)*	4 ½	160 ft	100 ft	(100 ft)*
1997 NEHRP	5**	160 ft	100 ft	100 ft	4	35 ft	NP	NP
2000 NEHRP	5**	35 ft	NP	NP	3 ½	NP	NP	NP
ASCE 7-05	3 ¼	35 ft	35 ft	NP	3 ½	NP	NP	NP

*The equivalent to SDC F did not exist in the 1994 NEHRP
**Required that members of bracing system be designed for Ω_o forces.

There were some members of TS-2 who suggested that having a tradeoff of R value for height should be treated as a global issue and not just be something that is associated with structural steel. While this is a nice idea, TS-2 should be the one to initiate such ideas. This has not happened. In fact, Section 12.2.5 exists in ASCE 7-05 (and in previous editions of NEHRP) that has always provided specific relaxation of restrictions for structural steel. With the exception of Section 12.2.5.4 which provides a special relaxation for regular Special Reinforced Concrete Walls, all the relaxations in the height limits requirements provided in Section 12.2.5 are for structural steel. Furthermore, in Table 12.2-1, a special entry (Row H) is provided at the end of the table for Steel Systems not specifically detailed for Seismic Resistance excluding Cantilever Column Systems. This entry allows unlimited height of non seismic detailed steel systems for structures assigned SDC A, B or C provided an R = 3 is used. No justification is provided in the NEHRP commentary for this entry other than judgment. And it is my understanding that TS-2 is not suggesting any changes to this entry so we find the idea of not providing special relaxations for structural steel by trading off for a lower R an inconsistent and incoherent argument.

What is very interesting is the extreme change in design requirements that occurs at the boundary where a steel structure may be assigned Seismic Design Category C or D. For a 160 foot high normal occupancy structure, this occurs when the Sd1 value is about 0.20. On one side of the boundary, the steel structure can be designed using any system using an R=3 with not seismic detailing. With the current proposal, the same system can be used except the design forces need to be 4.5 times larger and seismic detailing is required. It would be interesting in performing ATC-63 type studies on the two different systems.

There were some who suggested we needed to have more design requirements for ordinary truss moment frames. Again because of lack of resources and time, the alternate option was to just lower the value of R so that they system stayed elastic and existing design requirements would be without question adequate. Truss moment frames in industrial buildings are a very common form of construction and have been built for a long time. It did not seem warranted to develop new rules if the R values are low enough.

Finally some wanted examples of the type of structures that currently have problems. As mentioned previously, this is primarily a problem with large spanning single story industrial buildings. However, again we did not have the resources and time to develop the examples requested so instead we simply lowered the R value to the lowest common denominator.

This change is needed to allow the design of certain types of industrial buildings to be constructed in areas of high seismicity where special systems are not feasible. It is fully expected once the ATC-63 project is completed and studies are performed on these ordinary systems, it will be determined that the design coefficients and factors will be shown to be very conservative and will be permitted to be increased. However, in the interim, these conservative values are proposed to permit design and construction of these important types of structures to proceed. It was suggested in discussion, that the alternate means and methods section of the code be used. While this is always a possibility, most firms that design these type facilities do not have the technical horsepower to go this direction and its not really fair to require them to engage high powered consultants to get through a building department. Remember, a few years ago (a few weeks ago in the case of California) these systems were permitted.

TS 8 Vote:

Y – 2 YR – 8 N – 0 NV – 3

Summary of ballot comments:

Bob Bachman - My Vote is YR. I would change my vote to Y if the value of R = 1.5, the value of Omega Zero = 1.5 and the value of Cd = 1.5. Reason: I think the proposed values are too conservative. It's my judgment that these systems possess some ductility (at least a ductility factor of 1.5) and that their nominal strength is at least 1.5 times the minimum specified strength. Together, it's my judgment that the factors I am proposing will provide more than an adequate cushion to satisfy the proposed MCE collapse criteria being proposed by the Seismic Design Procedures Reassessment Group and that future studies may in fact increase the R factor values.

Harold Sprague - I would like to vote YR with a suggestion to use an R of 1 1/2. The lowest R in the Building Frame Systems is currently 1 1/2. With the same well considered logic and engineering analysis that was used to develop all of the R factors and height limits, I see no reason to go below 1 1/2.

Jeffrey R. Soulages - I vote YR. I agree with Bob that the values are now TOO low. In addition, there is no consistency anymore with Table 15.4-1 of Chapter 15. As I have said many times before, these two should be consistent so that one day, we can eliminate the confusion of when to use Chapter 12 rather than the Chapter 15. The current values in Table 15.4-1 for OCBF in SDC D/E/F are R=2.5, Omega=2 and Cd=2.5. These are likely too high for most voters. I would advocate R=1.5, Omega=1.0 and Cd=1.5 which is listed for unlimited height and for buildings we would use the same values for 160/160/100 and include AISC 341 detailing.

Phil Caldwell - YR - This is a move in the right direction to resolve the gap in existing code on large nonbuilding structures. I concur with Bob Bachman's comments that the proposed values for R, Omega Zero and Cd are too conservative and would change my vote to Yes if more realistic values were assigned.

Greg Soules – I vote YR. The proposed values are way too conservative for the systems and materials under consideration. I would change my vote to Yes if more realistic values of R, Omega, and Cd were used. I believe that R = 1.5, Omega = 1, and Cd = 1.5 should be used.

Victor Azzi – I vote YR on Proposal 8-50R. I concur with Bachman's analysis and rationale for this change.

John Silva – YR - As an "old" TS-8 member, I am willing to move this into open discussion within the PUC. My reservation is not about the low R, but rather with the overall approach to this problem, which apparently does not lend itself to a defined scope.

> As Bob pointed out in his reason statement, there were insufficient resources and time to clearly document the issue with examples. Long-span single story industrial buildings are one thing. Very tall braced frames are another. The authors have tried to explain the problem in terms of structures that are currently permitted. The question is twofold:

1. Are the structures currently permitted (or, rather, that were permitted) good structures? This is of course the debate about past performance, observed behavior, etc.

2. Perhaps more importantly, what structures will be 'permitted' under this language change that we did not intend?

Hopefully, ATC-63 will come to the rescue, but in the meantime we don't want to open up a Pandora's Box (i.e., that can't be closed later).

I commend Bob for his tireless efforts.

John Gillengerten - YR - I agree with John Silva’s sentiments.