

## PROPOSAL 8-50RR (2009)

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### SCOPE: Section 12.2.5 of the 2009 Provisions

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#### PROPOSAL FOR CHANGE:

##### 1 **Revise Section 12.2.5 as follows:**

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3 **12.2.5 System Specific Requirements.** The structural framing system shall also comply with the  
4 following system specific requirements of this section.

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6 *No change proposed for Sections 12.2.5.1 thru 12.2.5.3*  
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##### 8 9 **12.2.5.4 Increased ~~Building~~ Height Limit for Steel Braced Frames and Special Reinforced** 10 **Concrete Shear Walls.**

##### 11 12 **12.2.5.4.1 Special Steel Centrally Braced Frames, Steel Eccentrically Braced,** 13 **Steel Buckling-Restrained Braced Frames and Special Reinforced Concrete Shear Walls.**

14 The height limits in Table 12.2-1 are permitted to be increased from 160 ft (50 m) to 240 ft (75  
15 m) for structures assigned to Seismic Design Categories D or E and from 100 ft (30 m) to 160 ft  
16 (50 m) for structures assigned to Seismic Design Category F that have special steel  
17 centrally braced frames, steel eccentrically braced frames, steel buckling-restrained braced  
18 frames or special reinforced concrete cast-in-place shear walls and that meet both of the  
19 following requirements:

- 20  
21 1. The structure shall not have an extreme torsional irregularity as defined in Table 12.2-1  
22 (horizontal structural irregularity Type 1b).  
23  
24 2. The braced frames or shear walls in any independent line of resistance ~~one plane~~ shall resist  
25 no more than 60 percent of the total seismic forces in each direction, neglecting accidental  
26 torsional effects.

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28 **12.2.5.4.2 Ordinary Steel Centrally Braced Frames.** The height limit in Table 12.2-  
29 1 are permitted to be increased to 160 ft (50 m) for structures assigned to Seismic Design  
30 Categories D or E and to 100 ft (30 m) for structures assigned to Seismic Design Category F that  
31 have ordinary steel centrally braced frames conforming to the detailing provisions for  
32 ordinary centrally braced frames specified in AISC 341 provided the Response  
33 Modification Coefficient,  $R$ , is taken as 1.0, the System Overstrength Factor,  $\omega$ , is taken as 1.6  
34 and the Deflection Amplification Factor,  $C_d$ , is taken as 1.0

1        *No change proposed for Section 12.2.5.5*  
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5        **12.2.5.6 Increased Height Limits for Single-Story Steel Ordinary and Intermediate**  
6        **Moment Frames in Structures Assigned to Seismic Design Category D or E.** Single-story  
7        steel ordinary moment frames and intermediate moment frames in structures assigned to Seismic  
8        Design Category D or E are permitted up to a height of 65 ft (20 m) where the dead load  
9        supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m<sup>2</sup>). In addition, the dead  
10       loads tributary to the moment frame, of the exterior wall more than 35 ft above the base shall not  
11       exceed 20 psf (0.96 kN/m<sup>2</sup>).  
12

13       **12.2.5.7 Increased Height Limits for Multi-Story Other Steel Ordinary and Intermediate**  
14       **Moment Frames in Structures Assigned to Seismic Design Category D or E.**  
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16        **12.2.5.7.1 Steel Ordinary Moment Frames within Light-Frame Construction.** Steel  
17        ordinary moment frames in structures assigned to Seismic Design Category D or E not meeting  
18        the limitations set forth in Section 12.2.5.6 are permitted within light-frame construction up to a  
19        height of 35 ft (10.6 m) where neither the roof nor the floor dead load supported by and tributary  
20        to the moment frames exceeds 35 psf (1.68 kN/m<sup>2</sup>). In addition, the dead load of the exterior  
21        walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m<sup>2</sup>).  
22

23        **12.2.5.7.2 Steel Intermediate Moment Frames.** Steel intermediate moment frames in  
24        structures assigned to Seismic Design Category D or E not meeting the limitations set forth in  
25        Section 12.2.5.6 are permitted as follows:  
26

- 27        1. In Seismic Design Category D, intermediate moment frames are permitted to a height of 35 ft  
28        (10.6 m).  
29        2. In Seismic Design Category E, intermediate moment frames are permitted to a height of 35 ft  
30        (10.6 m) provided neither the roof nor the floor dead load supported by and tributary to the  
31        moment frames exceeds 35 psf (1.68 kN/m<sup>2</sup>). In addition, the dead load of the exterior walls  
32        tributary to the moment frame shall not exceed 20 psf (0.96 kN/m<sup>2</sup>).  
33

34        **12.2.5.7.3 Ordinary Steel Moment Frames.** The height limit in Table 12.2-1 are permitted  
35        to be increased to 160 ft (50 m) for structures assigned to Seismic Design Categories D or E that  
36        have ordinary steel moment frames that conform to the detailing requirements of AISC 341  
37        including truss type systems that behave as moment frames that conform to design requirements  
38        for ordinary steel concentrically braced frames of AISC 341 provided the Response Modification  
39        Coefficient,  $R$ , is taken as 1.0, the System Overstrength Factor,  $\Omega$ , is taken as 1.6 and the  
40        Deflection Amplification Factor,  $C_d$ , is taken as 1.0.  
41

42        **12.2.5.8 Increased Height Limit for Single-Story Steel Ordinary and Intermediate Moment**  
43        **Frames in Structures Assigned to Seismic Design Category F.** Single-story steel ordinary  
44        moment frames and intermediate moment frames in structures assigned to Seismic Design  
45        Category F are permitted up to a height of 65 ft (20 m) where the dead load supported by and  
46        tributary to the roof does not exceed 20 psf (0.96 kN/m<sup>2</sup>). In addition, the dead loads of the

1 exterior walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m<sup>2</sup>).  
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5 **12.2.5.9 Increased Height Limit for Multi-Story ~~Other~~ Steel Ordinary and Intermediate**  
6 **Moment Frames ~~Limitations~~ in Structures Assigned to Seismic Design Category F.**  
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8 **12.2.5.9.1 Steel Intermediate Moment Frames within Light-Frame Construction.** Steel  
9 intermediate moment frames in structures assigned to Seismic Design Category F not meeting  
10 the limitations set forth in Section 12.2.5.6 are permitted within light-frame construction up to a  
11 height of 35 ft (10.6 m) where neither the roof nor the floor dead load supported by and tributary  
12 to the moment frames exceeds 35 psf (1.68 kN/m<sup>2</sup>). In addition, the dead load of the exterior  
13 walls tributary to the moment frame shall not exceed 20 psf (0.96 kN/m<sup>2</sup>). ~~In addition to the~~  
14 ~~limitations for steel intermediate moment frames in structures assigned to Seismic Design~~  
15 ~~Category E as set forth in Section 12.2.5.7.1, steel intermediate moment frames in structures~~  
16 ~~assigned to Seismic Design Category F are permitted in light-frame construction.~~  
17

18 **12.2.5.9.2 Ordinary Steel Moment Frames.** For ordinary steel moment frames assigned  
19 to Seismic Design Category F including steel truss moment frames that conform to the detailing  
20 provisions for ordinary concentrically braced frames specified in AISC 341, the height limit in  
21 Table 12.2-1 is permitted to be increased to 100 ft (30 m) provided the Response Modification  
22 Coefficient,  $R$ , is taken as 1.0, the System Overstrength Factor,  $\omega$ , is taken as 1.6 and the  
23 Deflection Amplification Factor,  $C_d$ , is taken as 1.0.  
24

25 -----  
26 *No change proposed for Section 12.2.5.10*  
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28 **REASON FOR PROPOSAL:**  
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30 This proposal is a revision to proposal 8-50R. This proposal permits an increase in height limits  
31 if a lower R is used for design for ordinary concentric brace steel frames and ordinary steel  
32 moment frames that are designed to satisfy AISC 341 with R values, Omega Zero values and Cd  
33 values specified. It should be noted that in the previous proposal, the R value was 0.67. In this  
34 proposal, the value has been changed to R= 1.0 in response to the comment of Malley in the  
35 PUC Ballot who indicated that the value should be what TS-8 is recommending rather than the  
36 value he suggested. Trading off an increase of height limits for lower R values is permitted for  
37 certain structural systems in Table 15.4-1 for nonbuilding structure applications. These new  
38 increased height limits are needed to permit construction of certain types of needed building  
39 structures used for heavy industrial applications and for certain metal building system  
40 applications. A fairly extensive justification (based on judgment) was provided in proposal 8-  
41 50R.  
42

43 Since the last PUC meeting in April when the resolution of comments for this proposal were  
44 voted, the Applied Technology Council published its 90% draft of the ATC-63 Project Report.  
45 While the project only studied a few structural system types, it did draw some pertinent

1 conclusions relative to this proposal. The following is a statement from Section 9.2.8 of the draft  
2 report.

3  
4 “Table 9-7 also showed a more important and disturbing trend, that the adjusted collapse margin  
5 ratio (ACMR) decreased (higher collapse risk) with increasing height. Haselton (2006) showed  
6 that this poor performance is caused by damage at localizing more taller moment frames  
7 (MRFs); this is driven primarily by higher P-Delta effects as the building height increases. This  
8 issue could be addressed in various ways. More conservative beam-column strength ratios  
9 could be developed for taller buildings, in order to cause the damage to spread more uniformly  
10 over the height of the building. To reduce P-Delta effects, more restrictive height limits could be  
11 imposed. Strength requirements could be increased for taller buildings, by using a period  
12 dependent R factor; in a recent paper, Krawinkler and Zareian (2007) illustrated how the R factor  
13 would need to change, as a function of period, in order to create uniform collapse probabilities  
14 for moment frame buildings of varying height.”

15  
16 While the ATC-63 project only studied moment frames, it is rational and logical to conclude that  
17 the same types of increases in strength would be required for braced frame structure with longer  
18 periods and increased heights. This is exactly what is provided in Table 15-4.1 of ASCE 7-05.  
19 Krawinkler and Zareian (paper is provided an attachment to this Proposal) suggest that for tall  
20 buildings, the special moment frame R values of 8 be divided by a factor of 2. It should be noted  
21 that the reduction factor on R they suggest is somewhat nonlinear depending on the starting R  
22 value. TS-8 is of the opinion, that the ATC-63 project has provided sufficient justification for the  
23 concept of trading off increased strength (reduced R) for permitted increases in height. The  
24 current proposed values are still based on judgment, but with more technical understanding than  
25 previously. It is hoped that future ATC-63 type studies will provide better justification for higher  
26 R values than those currently provided and proposed for Table 15.4-1.

27  
28 The ATC-63 project primarily studied reinforced concrete special moment frames and ordinary  
29 moment frames. Therefore the preliminary conclusions drawn by the project are not directly  
30 applicable to this proposal. However, it is reasonable and instructive to examine conclusions  
31 drawn for ordinary concrete moment frames and then to use engineering judgment to  
32 understand what the results would likely have been if steel ordinary moment frames and steel  
33 ordinary concentrically brace frames had instead been examined.

34  
35 In the ATC-63 project, several archetype configurations of concrete moment frames were  
36 examined. Of these the ones of most interest for use with higher seismic ground motions are  
37 those with low gravity loading since the ratio of seismic loading to gravity loading in column  
38 members will be less in areas of higher seismicity and since these result in higher required  
39 strengths (lower Rs). In the following discussion only systems with low gravity loading are  
40 considered. Also various height buildings arch type configurations were considered in ATC-63.  
41 In general the results of the evaluations were most applicable to periods that were greater than  
42 Ts. We focus on our attention therefore on buildings with periods greater than Ts. Finally since  
43 only two building heights were extended a range of ground motions, only buildings with 4  
44 stories and 12 stories are reviewed. The example studies described in the ATC-63 report in  
45 Section 9.3 for concrete OCBFs provides data that suggests that for the minimum earthquake  
46 ground motions of Seismic Design Category B ( $S_{DS} = 0.167$  g and  $S_{D1} = 0.067$  g), the

1 maximum earthquake ground motions of Seismic Design Category B ( $S_{DS} = 0.33$  g and  $S_{D1} =$   
 2  $0.133$ g), and although not permitted, the project examined OCBFs for maximum earthquake  
 3 ground motion level of Seismic Design Category C ( $S_{DS} = 0.50$  g and  $S_{D1} = 0.20$ g). Both the 4  
 4 story building and 12-Story buildings have a period greater than  $T_s$  and therefore the design  
 5 base shears and MCE motions are a function of  $S_{D1}$ . It is also assumed that the acceptable  
 6  $ACMR = 2.16$  in all cases. It is further assumed that implied factor is the ratio of the calculated  
 7  $ACMR$  divided by the acceptable  $ACMR$  times 3 which the R values for concrete OMFs. The  
 8 following are the implied R values as a function of  $S_{D1}$  and story height from the ATC-63 90%  
 9 Draft Report to provide a consistent risk of collapse.

10  
 11 Implied R Value to Provide Consistent Risk of Collapse (Concrete OMFs)

12 Building Type	13 $S_{D1} = 0.067$	14 $S_{D1} = 0.133$	15 $S_{D1} = 0.20$	16 $S_{D1} = 0.60^*$
17 4-Story Building (Perimeter)	18 4.9	19 3.2	20 2.5	21 1.0
22 12-Story Building (Perimeter)	23 3.4	24 3.1	25 1.7	26 0.6

27 \*Crudely extrapolated values – not provided in ATC-63 90% Draft Report

28 As can be seen in the above table, the implied value of R is a function of both height and level  
 29 of ground motion. The larger the ground motion and the higher the building, the lower the  
 30 implied value should be. The value of  $S_{D1} = 0.60$  (the maximum value of SDC D) was  
 31 determined by a crude extrapolation in which  $1/R$  is plotted versus  $S_{D1}$ . While having actual  
 32 values would have been preferred in this review, the above provides enough data to provide  
 33 judgments regarding the types of R values required to satisfy the ATC-63 criteria for low  
 34 ductility systems. It should be noted that the above values for  $S_{D1} = 0.60$  are intended for SDC  
 35 D and lower values may be required for SDC E and F since ground motion values are higher.  
 36 However, when  $S_{D1}$  is equal or greater than  $0.60$ g, the near field minimum is required which  
 37 generally required for buildings has periods greater than 2 seconds. For this reason,  
 38 extrapolations beyond  $S_{D1} = 0.60$ g are not provided.

39 It is the opinion of TS-8, that steel braced frames and moments not detailed for seismic have at  
 40 least the seismic collapse capacity of concrete OMFs and probably considerably more. Please  
 41 note that currently ASCE 7-05, concrete OMFS are not permitted in SDC C while steel braced  
 42 frames and moment frames are permitted with unlimited height in SDC C is an R equal to 3 is  
 43 used. It is also the opinion of TS-8 to presume that steel OMFs and steel OCBFs designed to  
 44 AISC 341 requirements have a significantly greater seismic capacity than those not detailed for  
 45 seismic. Therefore based on the above technical evaluation, it appears quite reasonable and  
 46 justified to assign an R value equal to 1 for height limits of 160 feet in Seismic Design Category  
 D and E and 100 feet for SDC F. It is hoped that future ATC-63 type studies will provide better  
 justification for higher R values than those currently proposed. While it would helpful to have  
 those studies now, funding and time are not currently available and it far to important to provide  
 the flexibility for industrial designers now with the opportunity to increase the R values later  
 with more detailed evaluations of steel OCF and OMF structural systems.

1  
2 In the proposal, the value of Cd is set equal to R for consistency with the equal displacement.  
3 The value of  $\rho$  was taken equal to 1.6 for consistency with values for lower ductility systems  
4 identified in the ATC-63 90% draft report. These values may be revised later with more detailed  
5 evaluations of steel OCF and OMF structural systems.

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8

9 **TS 8 VOTE:**

10  
11 *YES-6 Yes with Reservations-1 No-0 Not Voting-4 Abstain-1*

12  
13 *TS-8 members Soulages, Dowty, Lake, and Haupt did not vote because they were on vacation.*  
14 *Their votes/comments will be forwarded when received.*

15

**Yes with Reservations**

**Philip Caldwell** - Yes with Reservations and would change my vote to Yes as follows: The change in height limits proposed for nonbuilding structures in SDC D and higher appears to be based solely on expert judgment without the benefit of empirical knowledge gained from such structures during earthquakes or specialized studies such as the ATC-63 project. Is their more foundation to provide better insight into the basis of design to allow for a consistent application of these proposals by the structural design professional?

**TS-8 Response** - Current height limits and restrictions for structural systems are generally based on observed performance (empirical data) of structural systems in past earthquakes. However, one factor that has not entered these decisions is the impact of significant changes of design strength on these height limits and restrictions. In the past 10 years there have been several changes in required strengths along with increasing in detailing requirements for many structural systems.

Fortunately (or perhaps from a scientific bent unfortunately) there have been no earthquakes that would permit us to find out how new structures designed to these new criteria perform. However, a rational approach to establish R values along with appropriate height limits and restrictions was the primary motivation for the new ATC-63 project. To develop such a project was quite difficult and the members of the ATC-63 project should be applauded for their efforts.

However, the approach they have developed is very dependent on the broadness of the range of example structures they consider in their evaluations. Ordinary systems which have very broad ranges are especially difficult to evaluate since they are virtually unrestrained in geometry and physical design. To do proper ATC-63 evaluations for these types of systems will be quite costly and time consuming. At this point NIST is planning to fund some additional follow-up ATC-63 type studies (ATC-76) but none are focused on the systems that are the subject of this proposal. And the effort required to do a full ATC-63 type evaluation is far beyond that which can be done by the volunteer labor of current code committees. We look forward to the time when such funding is made available and such proper evaluations are performed based on the ATC-63 project methodology.

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