12.8.1.1

Minimum Design Base Shear

Addition (Reinstatement)

At a Glance

The minimum design base shear of $0.044S_{DS}I_eW$, applicable for Seismic Design Categories B through F, is reinstated from ASCE 7-02 in the equivalent lateral force procedure.

2010 Standard

12.8.1.1 Calculation of Seismic Response Coefficient

The seismic response coefficient, C_s, shall be determined in accordance with Eq. 12.8-2.

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \qquad \text{(Eq. 12.8-2)}$$

where

S_{DS} = the design spectral response acceleration parameter in the short period range as determined from Section 11.4.4 or 11.4.7

R = the response modification factor in Table 12.2-1

I_e = the importance factor determined in accordance with Section 11.5.1

The value of C_s computed in accordance with Eq. 12.8-2 need not exceed the following:

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \quad \text{for } T \le T_L \qquad \text{(Eq. 12.8-3)}$$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \quad \text{for } T > T_L \qquad \text{(Eq. 12.8-4)}$$

C_s shall not be less than

$$C_s = \cancel{0.01}\ \underline{0.044S_{DS}I_e \ge 0.01} \qquad \text{(Eq. 12.8-5)}$$

In addition, for structures located where S_1 is equal to or greater than 0.6*g*, C_s shall not be less than

$$C_S = 0.5S_1/(R/I_e) \qquad \text{(Eq. 12.8-6)}$$

where I_e and R are as defined in Section 12.8.1.1 and

S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4 or 11.4.7

T = the fundamental period of the structure (s) determined in Section 12.8.2

T_L = long-period transition period (s) determined in Section 11.4.5

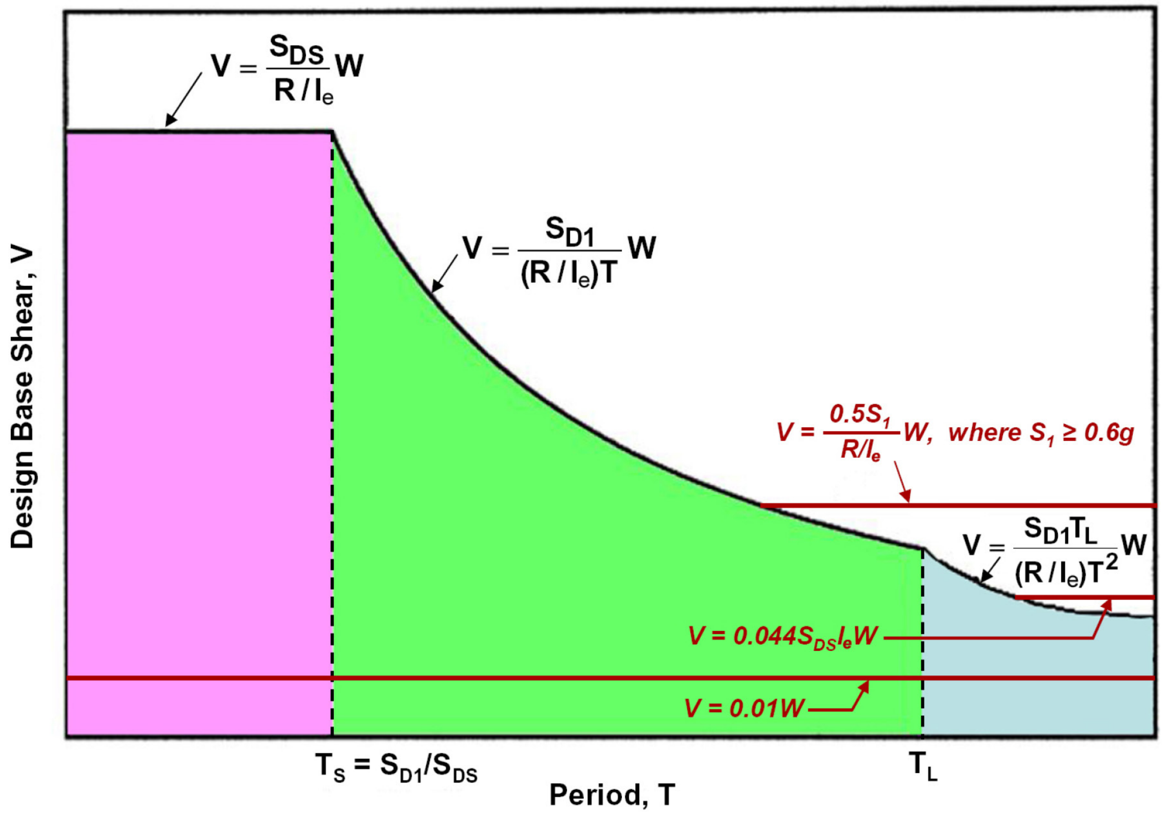
S_1 = the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 11.4.1 or 11.4.7

Analysis and Commentary

The minimum design base shear of $0.044S_{DS}I_eW$, applicable for Seismic Design Categories (SDC) B through F, was part of ASCE 7-02 and the 2000 and 2003 IBC. However, when the third (constant-displacement) branch, starting at T_L, was added to ASCE 7-05, this minimum base shear was deleted in favor of just 1% of weight, which is a structural integrity minimum, applicable irrespective of SDC. The basis was that the long-period structure was now being directly addressed by the constant-displacement branch of the design spectrum, so that there was no need for an arbitrary minimum value.

In the course of the ATC-63 project (*Quantification of Building System Performance and Response Parameters*), a large number of ordinary as well as special moment frames of concrete were analyzed by state-of-the-art dynamic analysis procedures – each frame under a large number of pairs of earthquake ground motions. The analyses disturbingly showed story mechanisms forming even in the special moment frames, which satisfied the strong column – weak beam requirement, early into earthquake excitations. After considerable deliberations, the suggestion emerged that these frames, designed by ASCE 7-05, be designed instead by ASCE 7-02, in effect reinstating the minimum design base shear requirement of $0.044S_{DS}I_eW$. When this was done, the problem went away, leading to the inescapable conclusion that the removal of this minimum base shear had been a mistake. ASCE processed a Supplement No. 2 to ASCE 7-05, reinstating this minimum design base shear. Supplement No. 2 has been adopted by the 2009 IBC. ASCE 7-10 has now incorporated Supplement No. 2 in its body.

The minimum design base shear of $0.044S_{DS}I_e$ is also added to Equations 15.4-1 and 15.4-2 for nonbuilding structures not similar to buildings. Because nonbuilding structures not similar to buildings have low R-values compared to the special reinforced concrete moment frames studied in ATC-63, the ASCE 7 standards committee chose not to restore the high minimum base shears for nonbuilding structures not similar to buildings found in ASCE 7-02. In many cases, these previous minimum base shears gave many nonbuilding structures not similar to buildings effective R-values less than 1.0. Therefore, the committee believed that the minimum base shear equation of $0.044S_{DS}I_e$ used for buildings should also be applied to nonbuilding structures not similar to buildings.



ASCE 7-10 Design Response Spectrum for Equivalent Lateral Force Procedure

Courtesy: S.K. Ghosh Associates Inc.

12.8.1.1

Clarification

S_{DS}, S_{D1} in Seismic Response Coefficient Computation

At a Glance

Clarification is provided that S_{DS} and S_{D1} for computation of seismic design base shear can be obtained from site-specific ground motion procedures.

2010 Standard

12.8.1.1 Calculation of Seismic Response Coefficient

The seismic response coefficient, C_s, shall be determined in accordance with Eq. 12.8-2.

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \qquad (12.8\text{-}2)$$

where

S_{DS} = the design spectral response acceleration parameter in the short period range as determined from Section 11.4.4 <u>or 11.4.7</u>

R = the response modification factor in Table 12.2-1

I_e = the importance factor determined in accordance with Section 11.5.1

The value of C_s computed in accordance with Eq. 12.8-2 need not exceed the following:

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \quad \text{for } T \le T_L \qquad (12.8\text{-}3)$$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \quad \text{for } T > T_L \qquad (12.8\text{-}4)$$

C_s shall not be less than

$$C_s = 0.044 S_{DS} I_e \ge 0.01 \qquad (12.8\text{-}5)$$

In addition, for structures located where S_1 is equal to or greater than 0.6*g*, C_s shall not be less than

$$C_S = 0.5S_1/(R/I_e) \qquad (12.8\text{-}6)$$

Where I_e and R are as defined in Section 12.8.1.1 and

S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4 or 11.4.7

T = the fundamental period of the structure (s) determined in Section 12.8.2

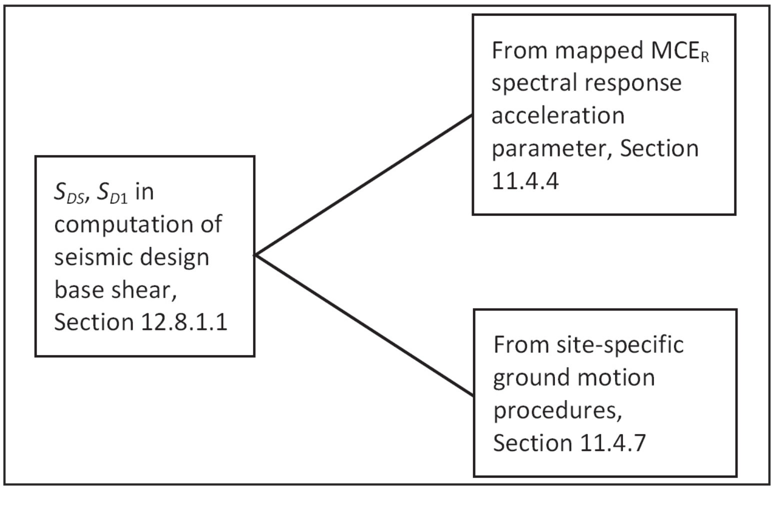
T_L = long-period transition period (s) determined in Section 11.4.5

S_1 = the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 11.4.1 or 11.4.7

Analysis and Commentary

In the event that a site-specific response spectrum is used to characterize the design ground motion, S_{DS} and S_{D1} are determined in accordance with Section 11.4.7 rather than 11.4.1. Although it may already be implied that the values obtained using the site-specific procedure (when applicable) may be used, the ASCE 7-10 text makes this more explicit.

The potential for confusion exists particularly if the site-specific response spectrum is used to perform a dynamic analysis using the modal response spectrum procedure of Section 12.9. In this case, S_{DS} and S_{D1} may not be computed when the response spectrum is developed because they are not needed to perform the analysis. However, in accordance with Section 12.9.4, the designer must compare the base shear obtained from the dynamic analysis with that given by the equivalent lateral force procedure and scale the results if necessary. As currently written, the code leads the user back to the general ground acceleration parameters of Section 11.4.4 for computation of the base shear from the equivalent static procedure rather than the site-specific values.



Either Section 11.4.4 or 11.4.7 may be used to determine the seismic design base shear when using the equivalent lateral force procedure of Section 12.8.

12.8.2.1, Table 12.8-2

Clarification and Addition

Approximate Fundamental Period

At a Glance

Determination of approximate fundamental period of steel eccentrically braced frames is clarified and steel buckling-restrained braced frames are added to Table 12.8-2.

2010 Standard

12.8.2.1 Approximate Fundamental Period

The approximate fundamental period (T_a), in s, shall be determined from the following equation:

$$T_a = C_t h_n^x \qquad \text{(Eq. 12.8-7)}$$

where h_n is the structural height as defined in Section 11.2 and the coefficients C_t and x are determined from Table 12.8-2

Table 12.8- 2 Values of Approximate Period Parameters C_t and x

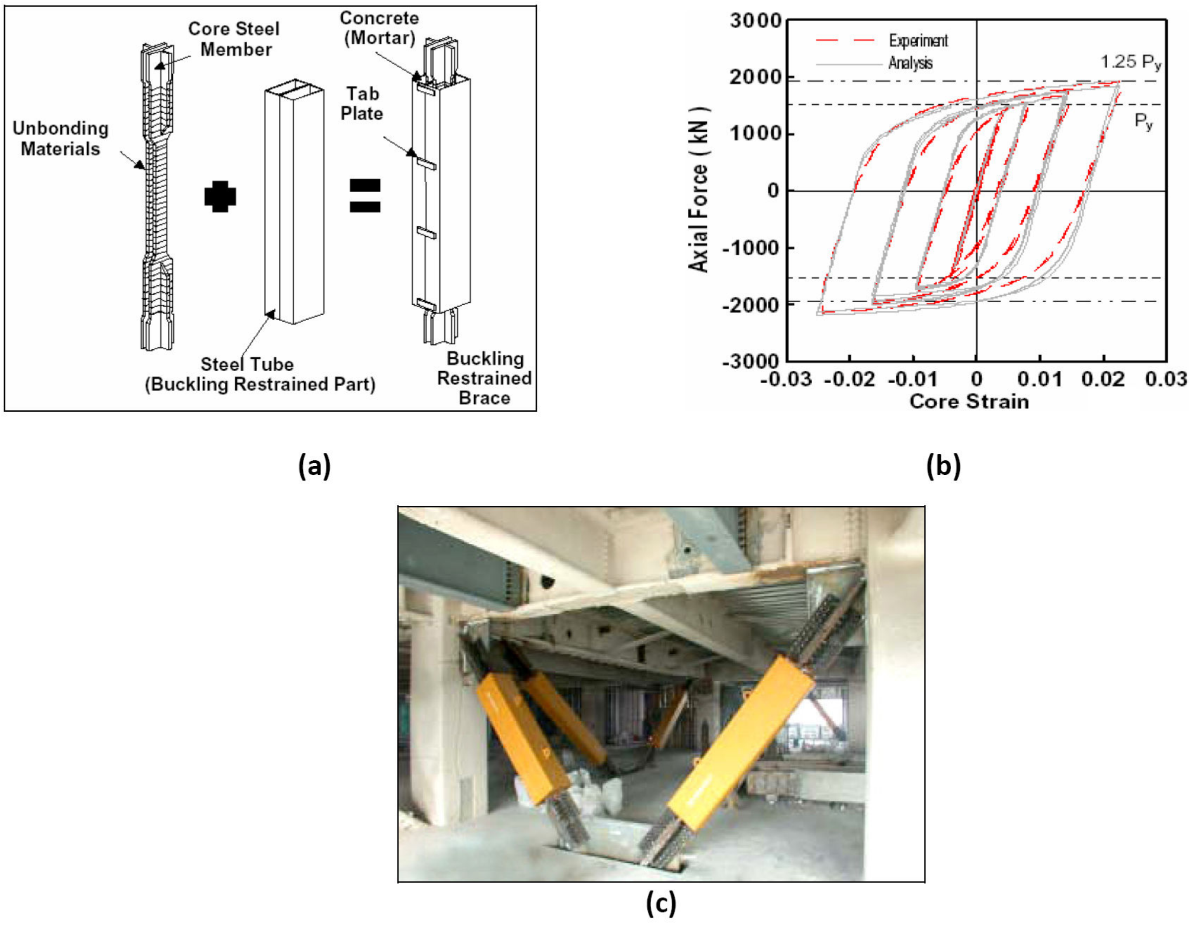
Structure Type	C_t	x
Moment resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel Moment Resisting Frames	0.028 (0.0724)[a]	0.8
Concrete Moment Resisting Frames	0.016 (0.0466)[a]	0.9
Steel e~~E~~ccentrically braced ~~steel~~ frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731)[a]	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a] Metric equivalents are shown in parentheses.

Analysis and Commentary

The longer predicted periods represented by the coefficient C_t = 0.03 for steel eccentrically braced frames (EBFs) are appropriate where significant eccentricities exist such as is the case when designing eccentrically braced frames in accordance with AISC 341. Without the added language in Table 12.8-2, minimal eccentricities could be used to justify the longer period, which would unconservatively result in a reduced base shear. The added language provides clarification and ensures that significant EBF eccentricities exist, which are consistent with longer periods.

The steel buckling restrained braced frame (BRBF) system was first approved for the 2003 NEHRP *Provisions*. The values for the approximate period parameters C_t and x were also approved as part of that original BSSC proposal. It appears to have been an oversight that these parameters were not carried forward into ASCE 7-05. Currently, these two factors can be found in Appendix R of AISC 341-05. These will be removed in the next edition of AISC 341 in view of this change in ASCE 7-10.



(a) Typical Configuration of Buckling Restrained Brace (BRB), (b) Typical Load-Displacement Behavior of a BRB, and (c) BRBs in Taipei County Administration Building, Taiwan
(reprinted with permission from Dr. K. C. Tsai, National Taiwan University, Taipei)

12.8.2.1 Modification

Approximate Period Equation Based on Number of Stories

At a Glance

The word "average" is added to story height so that the approximate period equation based on number of stories can be used if the average story height is at least 10 feet.

2010 Standard

12.8.2.1 Approximate Fundamental Period

The approximate fundamental period (T_a), in s, shall be determined from the following equation:

$$T_a = C_t h_n^x \qquad (12.8\text{-}7)$$

where h_n is the structural height as defined in Section 11.2 and the coefficients C_t and x are determined from Table 12.8-2.

Alternatively, it is permitted to determine the approximate fundamental period (T_a), in s, from the following equation for structures not exceeding 12 stories above the base as defined in Section 11.2 where the seismic force-resisting system consists entirely of concrete or steel moment resisting frames and the average story height~~, h_{sx},~~ is at least 10 ft (3 m):

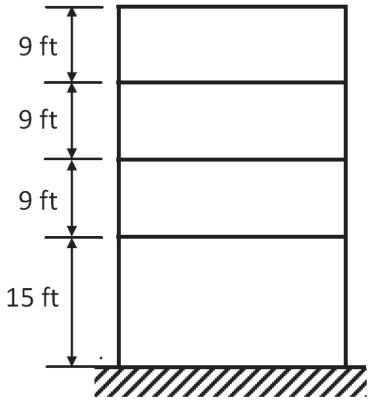
$$T_a = 0.1\,N \qquad (12.8\text{-}8)$$

where N = number of stories above the base.

(No change in remainder of section)

Analysis and Commentary

In defining the applicability of Eq. (12.8-8), the 10-ft minimum story height is revised such that it is an average story height.



For illustration,
Minimum Story Height = 9 ft
Average Story Height = 10.5 ft
ASCE 7-05 Equation 12.8-8 does not apply
ASCE 7-10 Equation 12.8-8 applies

12.8.4.3

Amplification of Accidental Torsion

Modification and Deletion

At a Glance

The exemption for structures of light frame construction from the amplification of accidental torsion requirement is discontinued.

2010 Standard

12.8.4.3 Amplification of Accidental Torsional Moment

Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) as illustrated in Fig. 12.8-1 and determined from the following equation:

$$A_x = (\delta_{max} / 1.2\delta_{avg})^2 \qquad (12.8\text{-}14)$$

where

δ_{max} = the maximum displacement at Level x computed assuming A_x = 1 (in. or mm)

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x computed assuming A_x = 1 (in. or mm)

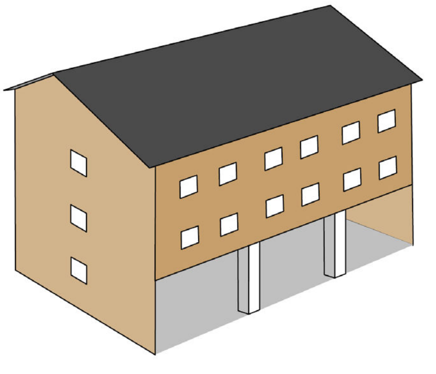
~~EXCEPTION: The accidental torsional moment need not be amplified for structures of light-frame construction.~~

The torsional amplification factor (A_x) shall not be less than 1 and is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

Analysis and Commentary

Where wood-frame diaphragms are designed as rigid diaphragms (one example is diaphragms in open-front structures), amplification of torsion should apply. Commentary to AF&PA's *Special Design Provisions for Wind and Seismic* indicates application of provisions of ASCE 7 for increased forces due to presence of irregularities, in addition to special prescriptive limits on diaphragm aspect ratio for open front structures.

Because it is possible for A_x to be less than 1 per Eq. 12.8-14, text is added to clearly indicate that A_x shall not be less than 1.



Light-frame Construction with Torsional Irregularity

12.8.6, 12.12.1 Modification

Story Drift Determination

At a Glance

Aspects of story drift determination are clarified.

2010 Standard

12.8.6 Story Drift Determination

The design story drift (Δ) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration. See Fig. 12.8-2. Where centers of mass do not align vertically, it is permitted to compute the deflection at the bottom of the story based on the vertical projection of the center of mass at the top of the story. Where allowable stress design is used, Δ shall be computed using the strength level seismic forces specified in Section 12.8 without reduction for allowable stress design.

For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity Type 1a or 1b of Table 12.3-1, the design story drift, Δ , shall be computed as the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure.

The deflections at ~~of~~ Level x (δ_x) (in. or mm) used to compute the design story drift, Δ, ~~at the center of the mass (δ_x) (in. or mm)~~ shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \qquad (12.8\text{-}15)$$

where

C_d = the deflection amplification factor in Table 12.2-1

δ_{xe} = the deflection~~s~~ at the location required in this section determined by an elastic analysis

I_e = the importance factor determined in accordance with Section 11.5.1

12.12.1 Story Drift Limit

The design story drift (Δ) as determined in Sections 12.8.6, 12.9.2, or 16.1, shall not exceed the allowable story drift (Δ_a) as obtained from Table 12.12-1 for any story. ~~For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity Types 1a or 1b of Table 12.3-1, the design story drift, Δ , shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.~~

Analysis and Commentary

Section 12.12.1 focuses on story drift limits and not computation of the story drift demand, Δ. Determination of story drift demand is outlined in Section 12.8.6. Therefore, to consolidate and provide distinct separation between limit and demand, the last sentence in Section 12.12.1 that discusses determination of story drift when horizontal irregularity type 1a or 1b is present is moved to Section