Seismic Force-Resisting Systems

Part 1: Seismic Design Factors

SEAOC Seismology Committee

This is a two-part series of articles discussing the SEAOC Seismology Committee's comments and recommendations on the current design considerations of Seismic Force-Resisting Systems. Part 1 discusses Design Factors used for each system and Height Limitations. Part 2, to be published in a future issue of STRUCTURE® magazine, discusses System Attributes and recommendations to simplify Design Parameters for structural systems.

The structural systems most commonly used to resist earthquakes are listed in ASCE 7-05 Table 12.2-1. While the ASCE 7 table refers to them as "Seismic Force-Resisting Systems" (SFRSs), it is important to note that these are distinct elements within a possible combination of systems to form the overall lateral force-resisting system of a building, which would include diaphragms, foundations, and other load path components. The text of ASCE 7-05, however, is not careful about maintaining these distinctions, so the more common term "Seismic Force-Resisting System," or SFRS, is used here. That is, the SFRS is the "basic" system contemplated by ASCE 7-05. While SFRS is difficult to define, it is commonly understood to mean the set of vertically oriented structural elements above the foundation that are expected to act together to resist inter-story drifts and to resist design earthquake forces between levels of the structure.

System Selection

The selection of an SFRS for a specific building is clearly a design decision of fundamental importance, yet there is no system that is best for all buildings. Factors to consider when selecting a seismic force-resisting system include:

- *Performance*. All of the code-prescriptive systems are expected to meet the performance objective of the code. For enhanced performance objectives, however, some systems might be better than others. For example, some systems are better able to meet tight drift limits or assure speedy repair.
- *Architectural and nonstructural coordination*. A moment-resisting frame system can accommodate open spaces and unrestricted bays between columns. Braced frame and shear wall systems generally offer less flexibility in space planning and fenestration. The spacing of gravity columns, fire-rated partitions, and utility cores can also affect the relative efficiency of certain systems.
- *Construction cost*. The project budget might dictate systems of certain materials, or cost-effective fabrication and erection procedures.
- *Design budget*. Some systems can be analyzed and designed effectively with simple hand calculations, while others require more expensive and time-consuming procedures.

Acceptable earthquake performance is a function of more than the selected structural system. Configuration of the SFRS within the building is fundamental to good design, concerning such issues as structural irregularities, torsion, redundancy, and the combination of systems.

Design Parameters

The three design parameters tabulated by ASCE 7-05 in Table 12.2-1 are:

1) A response modification coefficient, R
2) An overstrength factor, Ω_o
3) A deflection amplification factor, C_d.

In addition, limitations on allowable applications of systems to different Seismic Design Categories and for height of structure are listed.

The theoretical basis of these three parameters, and the relationships between them, were discussed in the September 2008 issue of *Structure® Magazine*. This article discusses only the relative values assigned to the various seismic force-resisting systems. Parameters assigned to a given system might be subject to code limits based on how the system is configured or combined with other systems within a building.

In general, design parameters are intended to provide equivalent confidence that each listed SFRS will meet the implied performance objectives of the code. The current parameters began as broad distinctions among four basic structural system types. Parameters for specific systems were assigned considering the characteristic performance expected of each main type, and parameters for newer systems have generally been added by judgmental comparison with older, more established systems.

Response Modification Coefficient

R is the parameter that most succinctly represents the potential capacity of a system for ductile response and energy dissipation. For the systems permitted in the highest seismic areas (represented by seismic design categories D, E, and F), the R values in ASCE 7-05 are essentially identical to those assigned by judgment in the late 1970s (ATC 1978). However, while the R values have remained the same, design and detailing provisions have changed to reflect research and seismic performance. Special moment-resisting steel frames, for example, were assigned the highest R value, 8, in 1978. They still have an R value of 8 in ASCE 7- 05, though design requirements have changed substantially based on an updated record of performance and testing.

R values for dozens of systems added since 1978 have been based in part on test data but primarily on judgment of expected performance compared to established systems. For example, ATC 3-06 assigned an R of 3.5 to reinforced masonry shear walls in a bearing wall system. Since then, details with more ductile behavior have been developed. ASCE 7- 05 retains the 3.5 value but classifies the traditional detailing as an "intermediate" system and limits its use. In seismic design categories D through F, special reinforced masonry shear walls are permitted instead, with an assigned R-value of 5.

Prior to ATC 3-06 (ATC 1978), the SEAOC *Blue Book* and the *Uniform Building Code* had identified four basic structure types (These will be described in Part 2 of this series). Each was assigned a value of the coefficient K based originally on the judgment of the SEAOC Seismology Committee. The relative K values, which remained unchanged through the 1985 UBC, were intended to give all types of structural systems an equal probability of acceptable performance under a design earthquake.

Still, the Committee acknowledged that K was largely a "judgment factor" and recognized that the record of earthquake performance for each system type would certainly grow, necessitating modifications. The first major modifications came with ATC 3-06. While the new R values were based on the existing K coefficients of the Blue Book, ATC-3-06 expanded the system table from four basic types to 21 material-specific systems, making distinctions among them to reflect expected differences in "toughness" and "damping." ATC-3-06 clearly notes that its R values were "based on the best judgment and data available … and need to be reviewed periodically."

R values of the 1997 UBC, which differs somewhat from ASCE 7-05, were converted from earlier K values with the intent of leaving design base shear levels unchanged. Roughly, $R = 8/1.4K$, where the 1.4 factor accounts for the shift from allowable stress design to strength design. With this conversion, the ductile moment frames, which had been assigned a K factor of 0.67, would have qualified for R of about 8.5. Bearing wall systems, called "box systems" in earlier UBC editions, had been assigned a K of 1.33, which converted to an R factor of about 4.3. Thus, some of the differences between 1997 UBC and ASCE 7-05 (for example, 8.5 vs. 8 for special steel moment frames, and 4.5 vs. 5 for special concrete shear walls in bearing wall systems) do not represent technical disagreements so much as different genealogies of code development. In both cases, the code values continue to reflect the judgment of previous generations.

Deflection Amplification Factor

The C_d values in ASCE 7-05 also trace back to ATC 3-06. Low C_d values indicate relatively brittle systems.

The 1997 UBC is similar in its drift provisions to ASCE 7-05, but it does not give system-specific C_d values. Instead, the UBC amplifies deflections by the value $0.7R$. The $0.7R$ amplifier effectively requires design for 70% of the drift of the theoretically elastic structure. The corresponding value in ASCE 7-05 would be represented by the ratio C_d/R. To the extent that this ratio differs from 0.7, the two codes will lead to different calculated drifts and could result in different designs.

The issue is most relevant for structures likely to be governed by drift rather than strength; that is, tall flexible structures with high R values, such as special moment-resisting frames. ASCE 7-05 Table 12.2-1 includes three moment frame systems permitted for use in buildings taller than 65 feet in seismic design categories D through F. For each of these, $C_d/R = 5.5/8 = 0.69$, so the design requirement is consistent with the UBC's 0.7 value.

However, for the seven other similarly permitted systems in ASCE 7-05 with R of 8, the ratio of C_d/R ranges from 0.50 to 0.81. (If systems with R as low as 5 are included, the ratio ranges from 0.5 to 1.0.) There are also some clear inconsistencies. For example, dual systems with special concentrically braced frames are to be designed for 81% of the elastic drift, but dual systems with eccentrically braced frames need design for only 50%. It is not clear why the C_d/R ratios for these two systems should be so different, and on opposite sides of the 70% value for special moment frames.

Height Limits

The height limits imposed by ASCE 7-05 Table 12.2-1, particularly in high seismic areas, are intended as checks on the judgmentally assigned design parameters. That is, since the design parameters were based largely on judgment and supported by limited post-earthquake observations, in lieu of exhaustive analysis or testing, it was important to the code developers to restrict certain systems to the range of application with which they were familiar. For example, ATC 3-06 notes that "the lack of reliable data on the behavior of high-rise buildings whose structural systems involve shear walls or braced frames makes it convenient at present to establish some limits." The current 65-foot limits on light-framed and timber systems also reflect the limits of practical experience.

The basic limit for non-moment-resisting frame systems in ASCE 7-05 is 160 feet, a value established by the first Blue Book to supplement an earlier Los Angeles code requirement for buildings taller than 13 stories. A height limit of 13 stories, approximately 150 or 160 feet, was imposed by Los Angeles zoning regulations since approximately the early-1900s. A variance was granted for the 1928 Los Angeles City Hall, at 454 feet, the tallest building in California until 1964. Several buildings in downtown Los Angeles were constructed to the 13-story zoning cap in the first half of the twentieth century and called "limit-height" buildings. In 1943, "the City of Los Angeles introduced an innovative seismic code that related the seismic coefficient to the flexibility of the building – the first such code in the United States and among the first anywhere." (Berg 1983, p. 26) This "dynamic" equivalent static lateral force formula was written in such a way that the calculation only worked for a building up to 13 stories in height – because none taller was allowed in that jurisdiction. When the zoning-based height limit of Los Angeles was abolished in the 1950s, California structural engineers faced a dilemma in the development of seismic requirements for taller high-rises.

Some engineers favored requiring moment-frame structures in buildings less than 160-feet tall; but others felt older buildings of a given height not conforming to the revised seismic regulations could be automatically considered deficient and obsolete.

Thus, the 160-foot limit has its origins in this Los Angeles city planning experience rather than an explicit seismic design rationale. The first Blue Book commentary noted clearly "the limitations of 13 stories and 160 feet have been established arbitrarily and are subject to further study." When ATC 3-06 revisited the system table, it retained the 160-foot limit for high seismic areas, as well as the note that its limiting values were "arbitrary, and considerable disagreement exists … regarding [their] adequacy." (ATC 1978).

The provision introduced into the UBC after Los Angeles eliminated its height limit required any building taller than 160 feet to have a "ductile" moment-resisting frame, ostensibly because shear wall and braced frame systems were perceived to lack "multiple lines of defense" While that limit has largely been retained for high-seismic areas, codes now permit other specific systems for heights up to 240 feet. As noted above, this is also an arbitrary value, but it does recognize other systems that, if not equivalent to special moment-resisting frames, are at least expected to provide ductility and reliability sufficient for acceptable performance.

Footnote d of ASCE 7-05 Table 12.2-1 refers to section 12.2.5.4, which permits systems with "steel braced frames" and "special reinforced concrete cast-in-place shear walls" up to 240 feet in seismic design categories D and E if they are configured for favorable resistance to torsion. The text of the provision is inconsistently worded, however, and therefore potentially unclear. Specific clarifications to section 12.2.5.4 and to Table 12.2-1 footnotes d and e are recommended below. It is the position of the Seismology Committee that the increased height limits in ASCE 7-05 Section 12.2.5.4 should apply only to:

- Cast-in-place (but not precast) special reinforced concrete shear walls in bearing wall systems, building frame systems, or dual systems with intermediate moment frames.
- Steel eccentrically braced frames with either type of connection at columns away from links.
- Special (not ordinary) steel concentrically braced frames in building frame systems, but not in dual systems with intermediate moment frames.

ASCE 7- 05 does not explicitly allow special steel truss moment frames (STMF) up to 240 feet, while the 1997 UBC allow STMF up to 240 feet. This discrepancy may be the result of an oversight during the development of ASCE 7- 05. Without a technical basis for specifically limiting the height of the STMF, the Seismology Committee supports the height limit as posed in the 1997 UBC.

In ASCE 7-05 Table 12.2.1, the height limits for some dual systems with intermediate moment frames are less than for a building having the same bearing wall or building frame system without the intermediate moment frame. This is inconsistent. The dual system maximum height should be equal to or greater than the limit tabulated for the case where the bearing wall or building frame system is used alone.

For Height Limits of SRFS in ASCE 7-05 Section 12.2.5.4, the Seismology Committee recommends the following clarifications (proposed additions underlined and deletions with ~~strikethrough~~):

12.2.5.4 Increased Building Height Limit for Steel Braced Frames, Special Steel Truss Moment Frames, and Special Reinforced Concrete Walls. The height limits in Table 12.2-1 are permitted to be increased from 160 feet (50 meters) to 240 feet (75 meters) for structures assigned to Seismic Design Categories D or E and from 100 feet (30 meters) to 160 feet (5 meters) for structures assigned to Seismic Design Category F that have steel eccentrically braced frames, special steel concentrically braced frames, special steel truss moment frames, or special reinforced concrete cast-in-place shear walls and that meet both of the following requirements in each story:

1) The structure shall not have an extreme torsional irregularity as defined in Table ~~12.2-1~~ 12.3-1 (horizontal structural irregularity Type 1b).
2) The braced frames, moment frames, or shear walls in any one plane shall resist no more than 60 percent of the total design seismic ~~forces~~ story shear in each direction, neglecting accidental torsional effects.
3) The design seismic shear in any braced frame, moment frame, or shear wall in any one plane resulting from torsional effects shall not exceed 20% of the total design seismic shear in that element.▪

The Structural Engineers Association of California (SEAOC) is a professional association of four member organizations representing the structural engineering community in California.

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