CHALLENGE AND BENEFITS OF UBC97
- Case studies of a series of high rise and mid rise building

Nabih Youssef
Kenny Lee
Nabih Youssef & Associates

Abstract

Many innovative concepts have been introduced in the UBC97, such as, redundancy, force level, connection design and other considerations. These new requirements influence the decision process of a designer, as well as imposing new norms in building design. These challenges require a designer to be creative in striving balance of structural performance and cost effectiveness. By studying a series of buildings ranging from 2 to 25 stories tall designed based on UBC97, a comparison of various lateral systems in terms of peculiarities of the UBC 97, construction detailing and issues and construction costs was studied.

Introduction

A number of new concepts have been introduced in UBC97. In a summary, the main items are as follows:

- New Soil Profile Categories
- Near-Source Factors (N_a and N_v)
- Revised Lateral Force Equations
- Strength Design (Revised R)
- Redundancy Factor (ρ)
- Revised Design for Elements & Components

The new soil profile categories are essentially defined in the 1992 Workshop on site response during earthquakes and seismic code provisions co-sponsored by NCEER, SEAOC and BSSC. The recommendations are basically derived from two-dimensional non-linear computer analysis of site responses using the Loma Prieta ground motions. Comparing with the UBC94 classifications, the new classifications are more quantitative and well defined. In general, the amplification factor for a soil site is slightly lower than UBC94 in the short period range and higher than that of UBC94 in the longer period range.

The near source factors accounts for the near source effects observed in the recent earthquakes, including Loma Prieta, Northridge, and Kobe. The near source effect is measured by the distance to the dominant nearby source and the seismic activity of the source. The table is used in conjunction with the seismic maps published by California Department of Conservation, Division of Mines and Geology.

The lateral force equation is re-cast such that the relationship between the period and response are slightly higher in the constant-velocity range. The system ductility factors (R Factors) are essentially the same, except they are scaled to the strength level to encourage the use of LRFD method.

A new parameter, Redundancy Factor (ρ) is added. The effect of this factor will be discussed more in detail in the following paragraph.

In general, based on UBC97, the design algorithm may be summarized as follows:

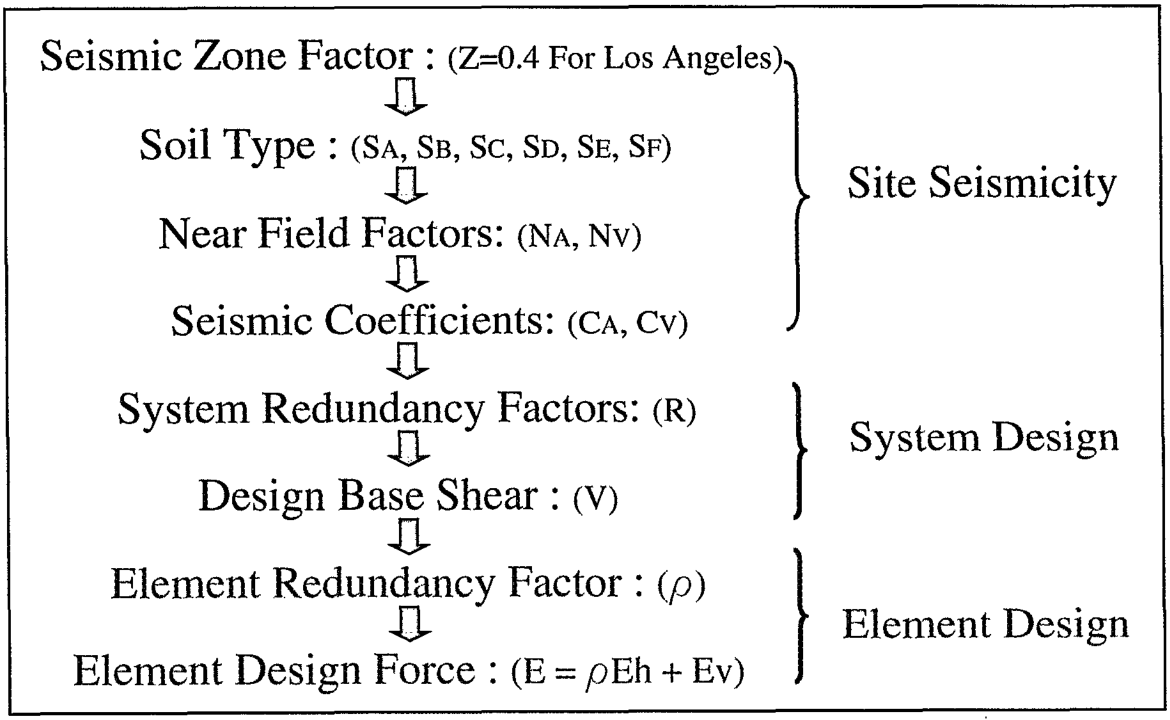


Figure 1 – UBC97 Design Algorithm

Recently, a number of buildings located in sites with similar seismicity were designed based on UBC97. The lateral systems of these buildings include SMRF, EBF, SCBC, OCBC and Dual System. Below is the general information of these buildings.

Project	**No. of Stories**	**Lateral System**		**Site Seismicity**	
		Long. Dir.	**Tran. Dir.**	**Na, Ca**	**Nv, Cv**
Glendale – 1	25 + 1 level of Basement	Steel SMRF	Steel SMRF + OCBF (Dual)	1.0, 0.44	1.2, 0.77
Burbank – 1	10 + 5 levels of Basement	Steel SMRF	Steel SMRF + EBF (Dual)	1.0, 0.44	1.2, 0.77
Burbank – 2	4	Steel SCBF	Steel SCBF	1.2, 0.53	1.5, 0.94
Del Mar	12	Concrete MF + Shear Wall (Dual)	Concrete MF + Shear Wall (Dual)	1.0, 0.40	1.2, 0.67
Irvine	8	Steel SCBF	Steel SCBF	1.0, 0.44	1.0, 0.64
Cerritos	7	Steel OCBF	Steel OCBF	1.0, 0.44	1.0, 0.64
Pasadena	6	Steel SMRF	Steel SMRF + EBF (Dual)	1.3, 0.57	1.6, 1.02
Santa Monica	4 + 2 levels of Basement	Concrete MF	Concrete MF		
Riverside	3	Concrete SW	Concrete SW	1.0, 0.4	1.2,0.67
Roseville	2	Steel EBF	Steel EBF	NA/0.3	NA/0.3

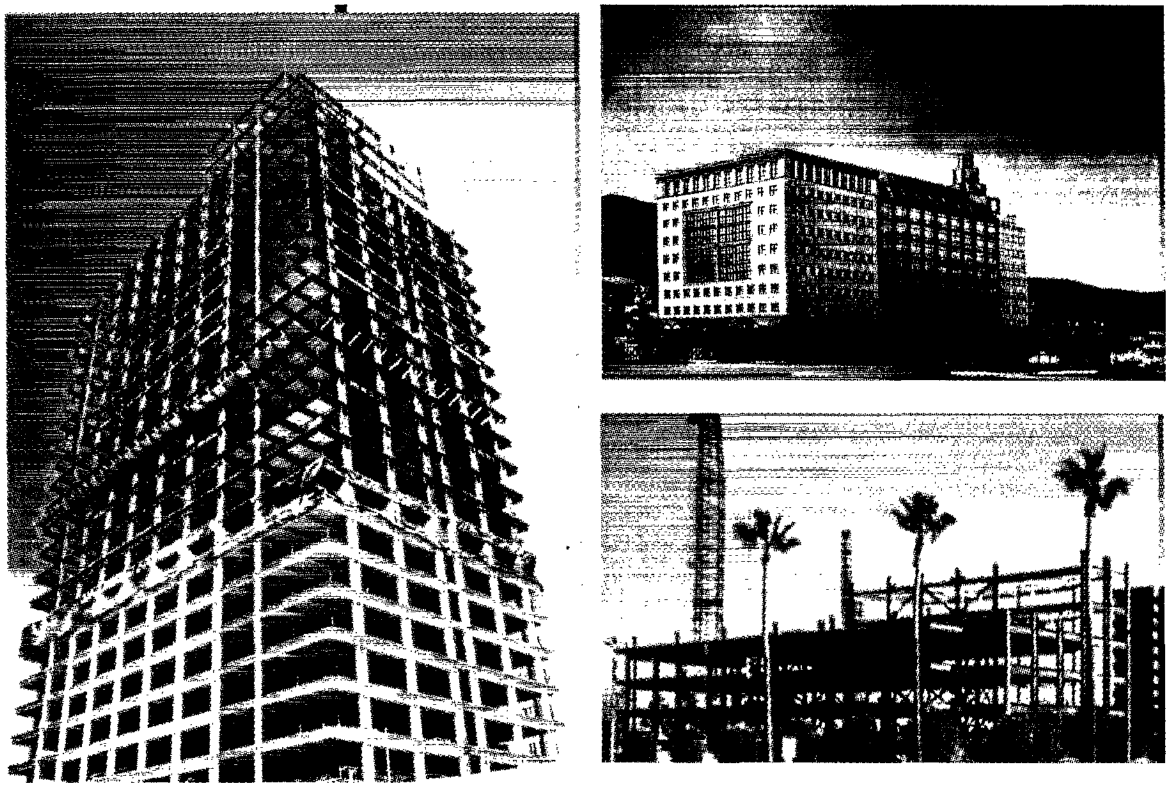


Figure 2 – Buildings used in the case studies

This series of similar buildings forms a rich data base for comparison of various systems in terms of peculiarities of the UBC 97, impact of bracing systems, design effort such as member /connection capacity design, construction detailing and issues and construction costs. This paper also discusses how the requirements of UBC97 influence the decision process of a designer. The main issues are as follows:

- Redundancy
- Force Level
- Connection Design

REDUNDANCY

In the 1980s and early 1990s, the development of rapid welding processes and the availability of large wide flange sections lead the design of steel buildings with ever fewer frames and bigger beam sections. The 1994 Northridge Earthquake gave structural engineers a wake up call. For the first time, UBC97 introduced equations for defining "Redundancy". Penalty in the form of increased design load is given when the number of frames is less than the specified number.

The Redundancy factor is arranged in the following form:

$$\rho = 2 - (20 / (r_{max} \sqrt{A}) \qquad \text{Eq - 1}$$

As a result, assuming the stiffness of individual frames is approximately equal, the number of frame requires is directly proportional to the square root of the area of the building. When the aspect ratio of the floor plan is close to one, (i.e. the floor plan is close to a square in shape) the number of frames required is directly proportional to the length of the building. Based on this relationship, one could derive a simple guideline to establish the number of frames required.

In SMRF buildings, assuming the length of the bays is approximately the same, shear forces are evenly divided between individual frame columns. The shear carried by an exterior column at the end bay may be approximately half of that of an interior column. Thus, the "r" value may be approximate by the following equation:

$$r = 0.7 \times 2 \times (V / N) \ / V \qquad \text{Eq - 2}$$

where
0.7 is the coefficient allowed in the code for interior columns
"V" denotes the design base shear
"N" denotes number of bays

By re-casting of the above equation, the number of bays required to achieve a redundancy factor of one may be taken as:

$$N = 0.07 \sqrt{A} \qquad \text{Eq - 3}$$

where
"A" denotes the area of the floor plate
"N" denotes number of bays

In the case of a Braced Frame Building, the shear is likely to be distributed among the brace members. With a similar approach, the number of braces required may be estimated by the following equation:

$$N = 0.05 \sqrt{A} \qquad \text{Eq - 4}$$

where
"A" denotes the area of the floor plate
"N" denotes number of braces

One must note that the above calculation neglected the effect of torsion and the variation in stiffness of individual frame line. The actual number of frames required to achieve a redundancy factor of one is likely to be higher. The following database presents the appropriateness of the above estimation.

Project	Floor Area "A"	Frame Direction	Estimated No. of Bays/ Braces	Actual No. of Bays/ Braces	Actual ρ factor
Glendale	22,000	Long.	10	10	1.0
		Trans.	10	16	1.0
Burbank – 1	20,000	Long.	10	12	1.0
		Trans.	10	17	1.0
Irvine	23,000	Long.	8	12	1.0
		Trans.	8	10	1.4
Cerritos	33,000	Long.	9	8	1.5
		Trans.	9	8	1.5
Pasadena	39,000	Long.	14	12	1.2
		Trans.	14	16	1.3
Santa Monica	28,000	Long.	12	24	1.0
		Trans.	12	20	1.0
Roseville	91,000	Long.	15	16	1.1
		Trans.	15	16	1.1

A more detailed study of the above table reveals that the simplified equations give fairly good estimate for buildings without significant torsion and abrupt change in stiffness. Where significant torsion occurs, the number of bays/braces requires are approximately 50% higher. When abrupt change of stiffness occurs, special attention should be given.

Traditionally, there had been a concern that a redundant building will tend to have higher unit price of steel, due to large number of connections. The following table provides some data of current steel prices of the above projects.

Project	No. of Stories	Redundancy		Steel Quantity and Price	
		Long. Dir.	Tran. Dir.	Tonnage (PSF)	Unit Price (Per Ton)
Glendale - 1	26 + 1 level of Basement	$\rho = 1$	$\rho = 1$	15	$1,450
Burbank - 1	10 + 5 levels of Basement	$\rho = 1$	$\rho = 1$	14	$1,400
Irvine	8	$\rho = 1$	$\rho = 1.4$	18	$1,200
Cerritos	7	$\rho = 1.5$	$\rho = 1.5$	13	$1,200
Pasadena	6	$\rho = 1.2$	$\rho = 1.3$	13	$1,350
Roseville	2	$\rho = 1.1$	$\rho = 1.1$	10	$1,200

COMPARISON OF SPECIAL CONCENTRIC BRACED FRAME AND ORDINARY CONCENTRIC BRACED FRAME

UBC94 introduced a new frame classification called Special Concentric Braced Frame, SCBF. Compared with "Ordinary Concentric Braced Frame", the main requirements include new width-thickness ratio for tube members, new gusset plate strength and stability requirements and new proportioning requirements for chevron braces.

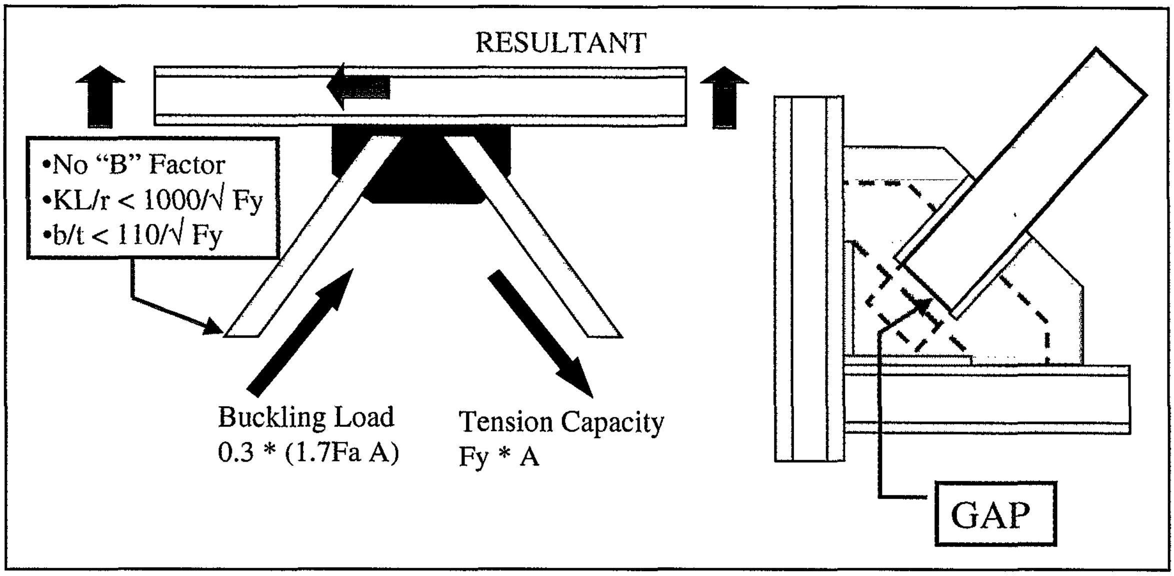


Figure 3 – SCBF Vs CBF

To qualify as a Special Concentric Braced Frame, the frame must be proportioned such that the yielding of the frame is limited to the brace elements. Thus, for a chevron-braced frame, the beam where the braces intersect must be designed for the unbalanced forces. Such requirement tends to require very large beam sections.

In addition, the brace elements must be compact sections to allow a stable post-yield behavior. Further, the gusset plate must be design to compressive strength of the braces without buckling. In the case of chevron braced frames, the beam intersecting the chevron have to resist the unbalance brace force. When these requirements are complied with, a 16% reduction of design strength is allowed, compared with a similar OCBF building. There are no benefit or penalty in drift requirement.

In the cases where all the braces are from column to column, the main consideration becomes the availability of brace sections. The maximum width-thickness ratio for standard tube sections (fy=46ksi) is 7.6. As a result, the largest common standard tube member one could use is TS10x10x5/8. Beyond that, built-up sections or WF members would be required. In our experience, for building five stories tall or less, standard tube sections typically are adequate. SCBF is typically the most economical system.

For taller building, however, when building drift governs the design, there is little benefit to design the building as SCBF from an economic point of view. On the contrary, the slightly larger connections required by SCBF might actually results in an increase of structural cost. Further, when the design base shear exceeds the threshold, one would have to use either built-up brace members or wide flange brace members. Both solutions result in a premium in erection costs.

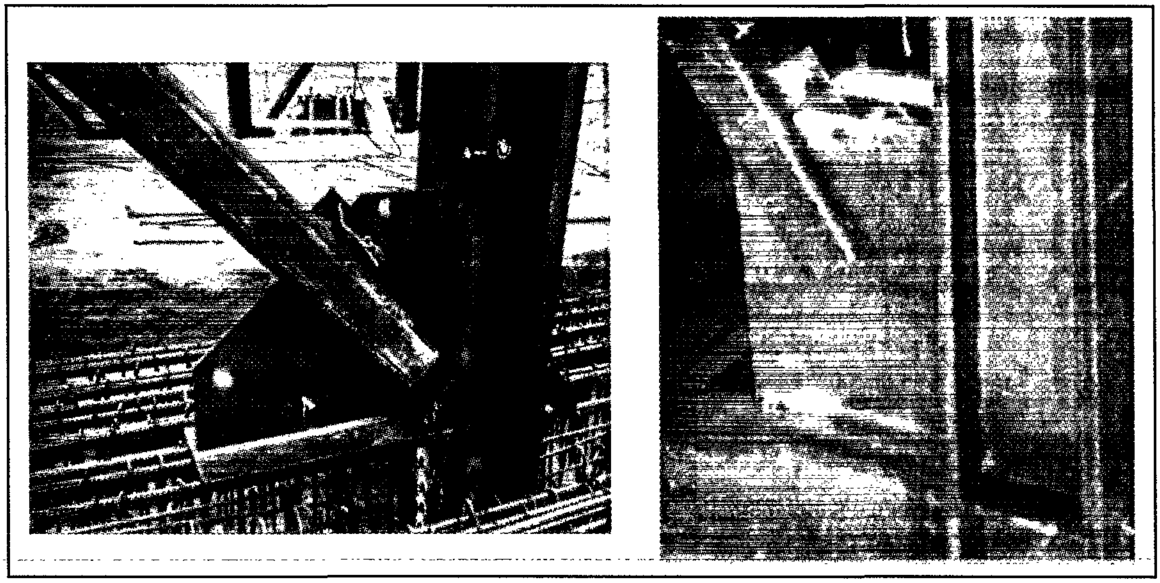


Figure 4 – Gusset Plate design for OCBF (Left) and SCBF (Right)

COMPARISON OF ECCENTRIC BRACED FRAME AND CONCENTRIC BRACED FRAME

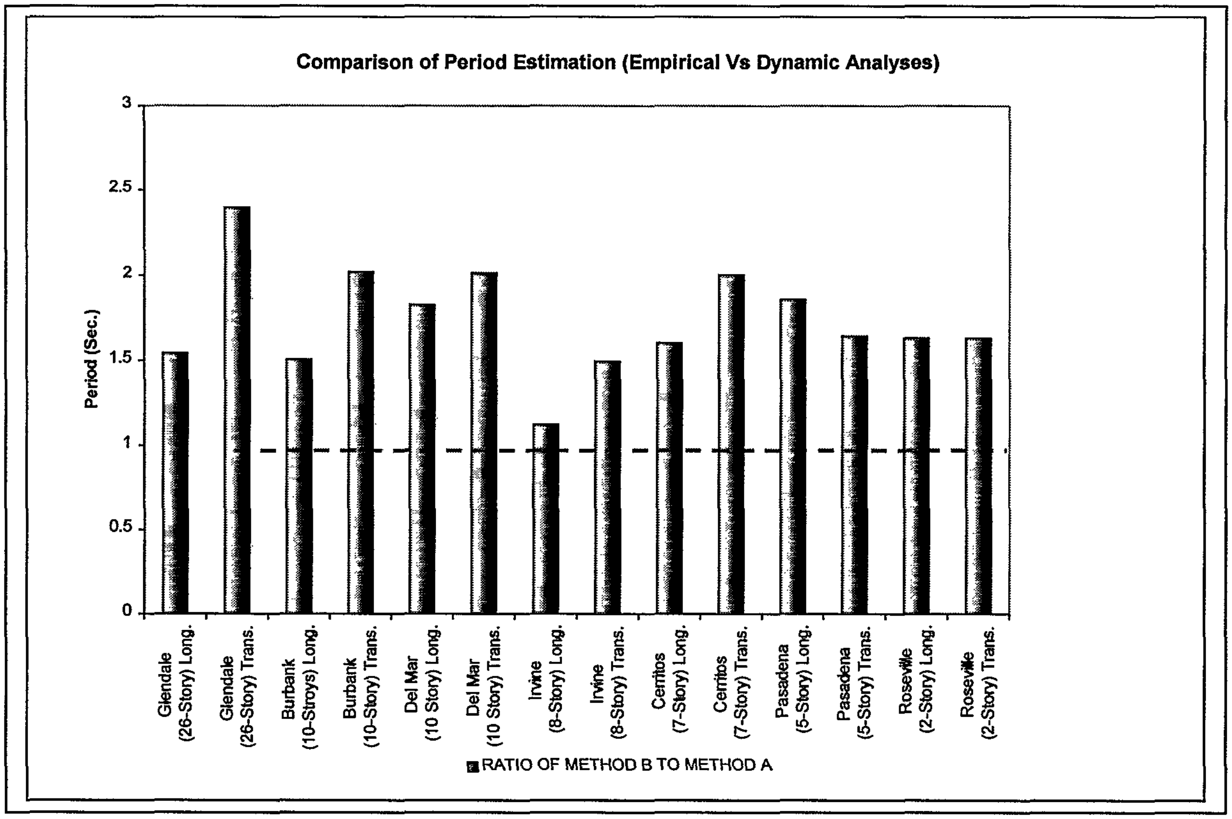
In the case of SCBF involving chevron braces, the beam intersected by chevron braces must be designed for the unbalanced brace force. In our experience, to satisfy this requirement, the intersecting beams usually are fairly large, rendering this option economically less competitive compared with the EBF system.

First of all, the EBF system has an "R" factor of 7 versus 6.4 for SCBF. This translates to a 10% reduction in design base shear for comparable buildings. The primary hidden penalty is the "Method A" period calculation. Equation (30-8) of UBC97 provides an empirical formula to estimate the period of a building of certain frame type. A design may also perform a dynamic analysis to calculate the period of the structure. However, the calculated period may not be greater than 30% of the empirical one. In our experience, the empirical building period often governs the design base shear for mid-rise (4-10 story) buildings. However, the empirical formula allows a much more liberal period for EBF buildings, compared with SCBF buildings. As a result, the design base shear formula favors EBF over SCBF for mid-rise building. The following table illustrates how the empirical formula impact the design forces:

Building	No. Of Stories	Frame System	"Method A" Period (Sec)	"Method B" Period (Sec)	Design Base Shear (9)
Burbank	10	SCBF	0.87	2.1	0.08
		EBF	1.31	2.3	0.053 (33% Red.)
Cerritos	8	SCBF	0.6	1.2	0.12
		EBF	0.9	1.3	0.08 (33% Red.)
Roseville	2	SCBF	0.29	0.42	0.12
		EBF	0.43	0.70	0.06 (50% Red.)

EMPIRICAL PERIOD CALCULATION

It seems that the earliest source of the period estimation empirical formula in its current form appears in ATC-3. Compared with recently designed standard use buildings, the empirical formula tends to give periods much lower than the one calculated based on dynamic analyses. The following chart presents the differences of the two methods for the above buildings.



CONCLUSION REMARKS

The above observations/studies presented the general implications of various code requirements from a designer's point of view. The intent of the paper is to help designers to identify implications and pit-falls of a design decision. It also points out how prescriptive building codes may unintentionally bias towards some building systems. To make a final design decision, however, a designer should also understand the difference in performance of these lateral systems and judge how the seismic behavior of the system meet the needs of the intended use of the building.