SEISMIC ANALYSIS MODELING TO SATISFY BUILDING CODES

The Current Building Codes Use the Terminology Principal Direction without A Unique Definition

17.1. INTRODUCTION

Currently a three-dimensional dynamic analysis is required for a large number of different types of structural systems that are constructed in Seismic Zones 2, 3 and 4 [1]. The lateral force requirements suggest several methods that can be used to determine the distribution of seismic forces within a structure. However, these guidelines are not unique and need further interpretations.

The major advantage of using the forces obtained from a dynamic analysis as the basis for a structural design is that the vertical distribution of forces may be significantly different from the forces obtained from an equivalent static load analysis. Consequently, the use of dynamic analysis will produce structural designs that are more earthquake resistant than structures designed using static loads.

For many years, approximate two-dimensional static load was acceptable as the basis for seismic design in many geographical areas and for most types of structural systems. During the past twenty years, due to the increasing availability of modern digital computers, most engineers have had experience with the static load analysis of three dimensional structures. However, few engineers, and the writers of the current building code, have had experience with the three dimensional dynamic response analysis. Therefore, the interpretation of the dynamic analysis requirement of the current code represents a new challenge to most structural engineers.

The current code allows the results obtained from a dynamic analysis to be normalized so that the maximum dynamic base shear is equal to the base shear obtained from a simple two-dimensional static load analysis. Most members of the profession realize that there is no theoretical foundation for this approach. However, for the purpose of selecting the magnitude of the dynamic loading that will satisfy the code requirements, this approach can be accepted, in a modified form, until a more rational method is adopted.

The calculation of the "design base shears" is simple and the variables are defined in the code. It is of interest to note, however, that the basic magnitude of the seismic loads has not changed significantly from previous codes. The major change is that "dynamic methods of analysis" must be used in the "principal directions" of the structure. The present code does not state how to define the principal directions for a three dimensional structure of arbitrary geometric shape. Since the design base shear can be different in each direction, this "scaled spectra" approach can produce a different input motion for each direction, for both regular and irregular structures. Therefore, ***the current code dynamic analysis approach can result in a structural design which is relatively "weak" in one direction***. The method of dynamic analysis proposed in this chapter results in a structural design that has equal resistance in all directions.

In addition, the maximum possible design base shear, which is defined by the present code, is approximately 35 percent of the weight of the structure. For many structures, it is less than 10 percent. It is generally recognized that this force level is small when compared to measured earthquake forces. Therefore, the use of this design base shear requires that substantial ductility be designed into the structure.

The definition of an irregular structure, the scaling of the dynamic base shears to the static base shears for each direction, the application of accidental torsional loads and the treatment of orthogonal loading effects are areas which are not clearly defined in the current building code. The purpose of this section is to present one method of three dimensional seismic analysis that will satisfy the Lateral Force Requirements of the code. The method is based on the response spectral shapes defined in the code and previously published and accepted computational procedures.

17.2. THREE DIMENSIONAL COMPUTER MODEL

Real and accidental torsional effects must be considered for all structures. Therefore, all structures must be treated as three dimensional systems. Structures with irregular plans, vertical setbacks or soft stories will cause no additional problems if a realistic three dimensional computer model is created. This model should be developed in the very early stages of design since it can be used for static wind and vertical loads, as well as dynamic seismic loads.

Only structural elements with significant stiffness and ductility should be modeled. Non-structural brittle components can be neglected. However, shearing, axial deformations and non-center line dimensions can be considered in all members without a significant increase in computational effort by most modern computer programs. The rigid, in-plane approximation of floor systems has been shown to be acceptable for most buildings. For the purpose of elastic dynamic analysis, gross concrete sections, neglecting the stiffness of the steel, are normally used. A cracked section mode should be used to check the final design.

The P-Delta effects should be included in all structural models. It has been shown in Chapter 11 that these second order effects can be considered, without iteration, for both static and dynamic loads. The effect of including P-Delta displacements in a dynamic analysis results in a small increase in the period of all modes. In addition to being more accurate, an additional advantage of automatically including P-Delta effects is that the moment magnification factor for all members can be taken as unity in all subsequent stress checks.

The mass of the structure can be estimated with a high degree of accuracy. The major assumption required is to estimate the amount of live load to be included as added mass. For certain types of structures it may be necessary to conduct several analyses with different values of mass. The lumped mass approximation has proven to be accurate. In the case of the rigid diaphragm approximation, the rotational mass moment of inertia must be calculated.

The stiffness of the foundation region of most structures can be modeled by massless structural elements. It is particularly important to model the stiffness of piles and the rotational stiffness at the base of shear walls.

The computer model for static loads only should be executed prior to conducting a dynamic analysis. Equilibrium can be checked and various modeling approximations can be verified with simple static load patterns. The results of a dynamic analysis are generally very complex and the forces obtained from a response spectra analysis are always positive. Therefore, dynamic equilibrium is almost impossible to check. However, it is relatively simple to check energy balances in both linear and nonlinear analysis.

17.3. THREE DIMENSIONAL MODE SHAPES AND FREQUENCIES

The first step in the dynamic analysis of a structural model is the calculation of the three dimensional mode shapes and natural frequencies of vibration. Within the past several years, very efficient computational methods have been developed which have greatly decreased the computational requirements associated with the calculation of orthogonal shape functions as presented in Chapter 14. It has been demonstrated that load-dependent Ritz vectors, which can be generated with a minimum of numerical effort, produce more accurate results when used for a seismic dynamic analysis than if the exact free-vibration mode shapes are used.

Therefore, a dynamic response spectra analysis can be conducted with approximately twice the computer time requirements of a static load analysis. Since systems with over 60,000 dynamic degrees-of-freedom can be solved within a few hours on personal computers, there is not a significant increase in cost between a static and a dynamic analysis. The major cost is the “man hours” required to produce the three dimensional computer model that is necessary for a static or a dynamic analysis.

In order to illustrate the dynamic properties of the three dimensional structure, the mode shapes and frequencies are calculated for the irregular, eight story, 80 foot tall building shown in Figure 17.1. This building is a concrete structure with several hundred degrees-of-freedom. However, the three components of mass are lumped at each of the eight floor levels. Therefore, only 24 three dimensional mode shapes are possible.

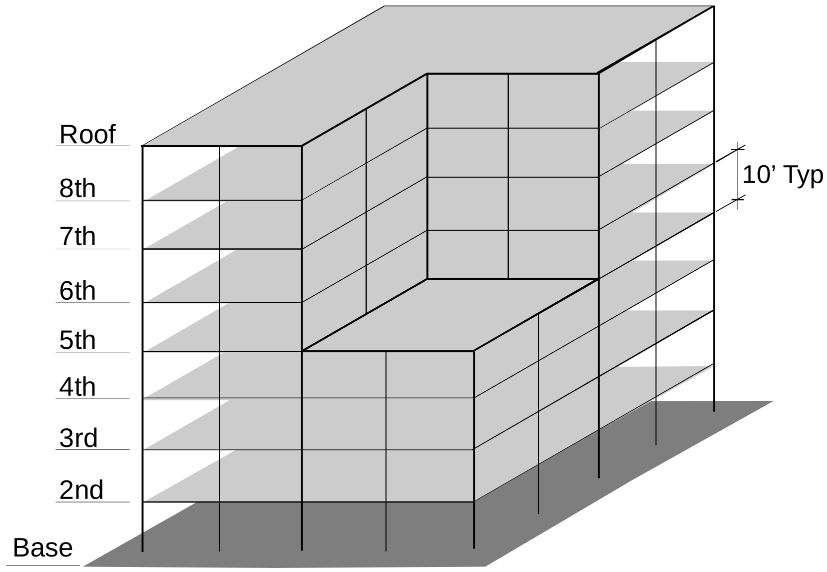

Figure 17.1. Example of Eight Story Irregular Building

Each three dimensional mode shape of a structure may have displacement components in all directions. For the special case of a symmetrical structure, the mode shapes are uncoupled and will have displacement in one direction only. Since each mode can be considered to be a deflection due to a set of static loads, six base reaction forces can be calculated for each mode shape. For the structure shown in Figure 17.1, Table 17.1 summarizes the two base reactions and three overturning moments associated with each mode shape. Since vertical mass has been neglected there is no vertical reaction. The magnitudes of the forces and moments have no meaning since the amplitude of a mode shape can be normalized to any value. However, the relative values of the different components of the shears and moments associated with each mode are of considerable value. The modes with a large torsional component are highlighted in **bold**.

Table 17.1. Three Dimensional Base Forces and Moments

MODE	PERIOD	MODAL BASE SHEAR REACTIONS			MODAL OVERTURNING MOMENTS		
	Seconds	X-DIR	Y-DIR	Angle Deg.	X-AXIS	Y-AXIS	Z-AXIS
1	.6315	.781	.624	38.64	-37.3	46.6	-18.9
2	.6034	-.624	.781	-51.37	-46.3	-37.0	38.3
3	**.3501**	**.785**	**.620**	**38.30**	**-31.9**	**40.2**	**85.6**
4	.1144	-.753	-.658	41.12	12.0	-13.7	7.2
5	.1135	.657	-.754	-48.89	13.6	11.9	-38.7
6	**.0706**	**.989**	**.147**	**8.43**	**-33.5**	**51.9**	**2438.3**
7	.0394	-.191	.982	-79.01	-10.4	-2.0	29.4
8	.0394	-.983	-.185	10.67	1.9	-10.4	26.9
9	**.0242**	**.848**	**.530**	**32.01**	**-5.6**	**8.5**	**277.9**
10	.0210	.739	.673	42.32	-5.3	5.8	-3.8
11	.0209	.672	-.740	-47.76	5.8	5.2	-39.0
12	**.0130**	**-.579**	**.815**	**-54.63**	**-.8**	**-8.8**	**-1391.9**
13	.0122	.683	.730	46.89	-4.4	4.1	-6.1
14	.0122	.730	-.683	-43.10	4.1	4.4	-40.2
15	.0087	-.132	-.991	82.40	5.2	-.7	-22.8
16	.0087	-.991	.135	-7.76	-.7	-5.2	30.8
17	**.0074**	**-.724**	**-.690**	**43.64**	**4.0**	**-4.2**	**-252.4**
18	.0063	-.745	-.667	41.86	3.1	-3.5	7.8
19	.0062	-.667	.745	-48.14	-3.5	-3.1	38.5
20	.0056	-.776	-.630	39.09	2.8	-3.4	54.1
21	.0055	-.630	.777	-50.96	-3.4	-2.8	38.6
22	.0052	.776	.631	39.15	-2.9	3.5	66.9
23	**.0038**	**-.766**	**-.643**	**40.02**	**3.0**	**-3.6**	**-323.4**
24	**.0034**	**-.771**	**-.637**	**39.58**	**2.9**	**-3.5**	**-436.7**

A careful examination of the directional properties of the three dimensional mode shapes at the early stages of a preliminary design can give a structural engineer additional information which can be used to improve the earthquake resistant design of a structure. The current code defines an "irregular structure" as one which has a certain geometric shape or in which stiffness and mass discontinuities exist. A far more rational definition is that a "regular structure" is one in which there is a minimum coupling between the lateral displacements and the torsional rotations for the mode shapes associated with the lower frequencies of the system. Therefore, if the model is modified and "tuned" by studying the three dimensional mode shapes during the preliminary design phase, it may be possible to convert a "geometrically irregular" structure to a "dynamically regular" structure from an earthquake-resistant design standpoint.

Table 17.2. Three Dimensional Participating Mass - (percent)

MODE	X-DIR	Y-DIR	Z-DIR	X-SUM	Y-SUM	Z-SUM
1	34.224	21.875	.000	34.224	21.875	.000
2	23.126	36.212	.000	57.350	58.087	.000
3	2.003	1.249	.000	59.354	59.336	.000
4	13.106	9.987	.000	72.460	69.323	.000
5	9.974	13.102	.000	82.434	82.425	.000
6	.002	.000	.000	82.436	82.425	.000
7	.293	17.770	.000	82.729	90.194	.000
8	7.726	.274	.000	90.455	90.469	.000
9	.039	.015	.000	90.494	90.484	.000
10	2.382	1.974	.000	92.876	92.458	.000
11	1.955	2.370	.000	94.831	94.828	.000
12	.000	.001	.000	94.831	94.829	.000
13	1.113	1.271	.000	95.945	96.100	.000
14	1.276	1.117	.000	97.220	97.217	.000
15	.028	1.556	.000	97.248	98.773	.000
16	1.555	.029	.000	98.803	98.802	.000
17	.011	.010	.000	98.814	98.812	.000
18	.503	.403	.000	99.316	99.215	.000
19	.405	.505	.000	99.722	99.720	.000
20	.102	.067	.000	99.824	99.787	.000
21	.111	.169	.000	99.935	99.957	.000
22	.062	.041	.000	99.997	99.998	.000
23	.003	.002	.000	100.000	100.000	.000
24	.001	.000	.000	100.000	100.000	.000

For this building, it is of interest to note that the mode shapes, which tend to have directions that are 90 degrees apart, have almost the same value for their period. This is typical of three dimensional mode shapes for both regular and irregular buildings. For regular symmetric structures, which have equal stiffness in all directions, the periods associated with the lateral displacements will result in pairs of identical periods. However, the directions associated with the pair of three dimensional mode shapes are not mathematically unique. For identical periods, most computer programs allow round-off errors to produce two mode shapes with directions which differ by 90 degrees. Therefore, the SRSS method should not be used to combine modal maximums in three dimensional dynamic analysis. The CQC method eliminates problems associated with closely spaced periods.

For a response spectrum analysis, the current code states that "at least 90 percent of the participating mass of the structure must be included in the calculation of response for each principal direction." Therefore, the number of modes to be evaluated must satisfy this requirement. Most computer programs automatically calculate the participating mass in all directions using the equations presented in Chapter 13. This requirement can be easily satisfied using LDR vectors. For the structure shown in Figure 17.1, the participating mass for each mode and for each direction is shown in Table 17.2. For this building, only eight modes are required to satisfy the 90 percent specification in both the x and y directions.

17.4. THREE DIMENSIONAL DYNAMIC ANALYSIS

It is possible to conduct a dynamic, time-history, response analysis by either the mode superposition or step-by-step methods of analysis. However, a standard time-history ground motion, for the purpose of design, has not been defined. Therefore, most engineers use the response spectrum method of analysis as the basic approach. The first step in a response spectrum analysis is the calculation of the three dimensional mode shapes and frequencies as indicated in the previous section.

17.4.1. Dynamic Design Base Shear

For dynamic analysis, the 1994 UBC requires that the "design base shear", **V**, is to be evaluated from the following formula:

$$\mathbf{V} = [\,\mathbf{Z\,I\,C} / \mathbf{R_W}\,]\,\mathbf{W} \tag{17.1}$$

Where

Z = Seismic zone factor given in Table 16-I.

I = Importance factor given in Table 16-K.

$\mathbf{R_W}$ = Numerical coefficient given in Table 16-N or 16-P.

W = The total seismic weight of the structure.

C = Numerical coefficient (2.75 maximum value) determined from:

$$\mathbf{C = 1.25\,S/\,T^{2/3}} \tag{1-2}$$

Where

S = Site coefficient for soil characteristics given in Table 16-J.

T = Fundamental period of vibration (seconds).

The period, **T**, determined from the three dimensional computer model, can be used for most cases. This is essentially Method B of the code.

Since the computer model often neglects nonstructural stiffness, the code requires that Method A be used under certain conditions. Method A defines the period, **T**, as follows:

$$\mathbf{T = C_t\,h^{3/4}} \tag{1-3}$$

where **h** is the height of the structure in feet and $\mathbf{C_t}$ is defined by the code for various types of structural systems.

The Period calculated by Method B cannot be taken as more than 30% longer than that computed using Method A in Seismic Zone 4 and more than 40% longer in Seismic Zones 1, 2 and 3.

For a structure that is defined by the code as “regular”, the design base shear may be reduced by an additional 10 percent. However, it must not be less than 80 percent of the shear calculated using Method A. For an “irregular” structure this reduction is not allowed.

17.4.2. Definition of Principal Directions

A weakness in the current code is the lack of definition of the "principal horizontal directions" for a general three dimensional structure. If each engineer is allowed to select an arbitrary reference system, the "dynamic base shear" will not be unique and each reference system could result in a different design. One solution to this problem, that will result in a unique design base shear, is to use the direction of the base shear associated with the fundamental mode of vibration as the definition of the "major principal direction" for the structure. The "minor principal direction" will be, by definition, ninety degrees from the major axis. This approach has some rational basis since it is valid for regular structures. Therefore, this definition of the principal directions will be used for the method of analysis presented in this chapter.

17.4.3. Directional and Orthogonal Effects

The required design seismic forces may come from any horizontal direction and, for the purpose of design, they may be assumed to act non-concurrently in the direction of each principal axis of the structure. In addition, for the purpose of member design, the effects of seismic loading in two orthogonal directions may be combined on a square-root-of-the-sum-of-the-squares (SRSS) basis. (Also, it is allowable to design members for 100 percent of the seismic forces in one direction plus 30 percent of the forces produced by the loading in the other direction. We will not use this approach in the procedure suggested here for reasons presented in Chapter 15.)

17.4.4. Basic Method of Seismic Analysis

In order to satisfy the current requirements, it is necessary to conduct two separate spectrum analyses in the major and minor principal directions (as defined above). Within each of these analyses, the Complete Quadratic Combination (CQC) method is used to accurately account for modal interaction effects in the estimation of the maximum response values. The spectra used in both of these analyses can be obtained directly from the Normalized Response Spectra Shapes given by the Uniform Building Code.

17.4.5. Scaling of Results

Each of these analyses will produce a base shear in the major principal direction. A single value for the "dynamic base shear" is calculated by the SRSS method. Also, a "dynamic base shear" can be calculated in the minor principal direction. The next step is to scale the previously used spectra shapes by the ratio of "design base shear" to the minimum value of the "dynamic base shear". This approach is more conservative than proposed by the current requirements, since only the scaling factor that produces the largest response is used. However, this approach is far more rational since it results in the same design earthquake in all directions.

17.4.6. Dynamic Displacements and Member Forces

The displacement and force distribution are calculated using the basic SRSS method to combine the results from 100 percent of the scaled spectra applied in each direction. If two analyses are conducted in any two orthogonal directions, in which the CQC method is used to combine the modal maximums for each analysis, and the results are combined by the SRSS method, exactly the same results will be obtained regardless of the orientation of the orthogonal reference system. Therefore, the direction of the base shear of the first mode defines a reference system for the building.

If site-specific spectra are given, for which scaling is not required, any orthogonal reference system can be used. In either case, only one computer run is necessary to calculate all member forces to be used for design.

17.4.7. Torsional Effects

Possible torsional ground motion, the unpredictable distribution of live load mass and the variations of structural properties are three reasons why both regular and irregular structures must be designed for accidental torsional loads. Also, for a regular structure lateral loads do not excite torsional modes. One method suggested in the Code is to conduct several different dynamic analyses with the mass at different locations. This approach is not practical since the basic dynamic properties of the structure (and the dynamic base shears) would be different for each analysis. In addition, the selection of the maximum member design forces would be a monumental post-processing problem.

The current Code allows the use of pure static torsional loads to predict the additional design forces caused by accidental torsion. The basic vertical distribution of lateral static loads is given by the Code equations. The static torsional moment at any level is calculated by the multiplication of the static load at that level by 5 percent of the maximum dimension at that level. In this book it is recommended that these pure torsional static loads, applied at the center of mass at each level, be used as the basic approach to account for accidental torsional loads. This static torsional load is treated as a separate load condition so that it can be appropriately combined with the other static and dynamic loads.

17.5. NUMERICAL EXAMPLE

To illustrate the base-shear scaling method recommended here, a static seismic analysis is conducted on the building shown in Figure 17.1. The eight-story building has 10 feet story heights. The seismic dead load is 238.3 kips for the top four stories and 363.9 kips for the lower four stories. For I = 1, Z = 0.4, S = 1.0, and R_W = 6.0, the evaluation of Equation 17.1 yields the design base forces given in Table 17.3.Table 17.3. Static Design Base Forces Using The Uniform Building Code

Period (sec)	Angle (deg)	Base Shear	Overturning Moment
0.631	38.64	279.9	14,533
0.603	-51.36	281.2	14,979

The normalized response spectra shape for soil type 1, which is defined in the Uniform Building Code, is used as the basic loading for the three dimensional dynamic analyses. Using eight modes only and the SRSS method of combining modal maxima, the base shears and overturning moments are summarized in Table 17.4 for various directions of loading.

Table 17.4. Dynamic Base Forces Using The SRSS Method

	BASE SHEARS		OVERTURNING MOMENTS	
Angle -deg	**V_1**	**V_2**	**M_1**	**M_2**
0	58.0	55.9	2982	3073
90	59.8	55.9	2983	3185
38.64	70.1	5.4	66	4135
-51.36	83.9	5.4	66	4500

The 1-axis is in the direction of the seismic input and the 2-axis is normal to the direction of the loading. This example clearly illustrates the major weakness of the SRSS method of modal combination. Unless the input is in the direction of the fundamental mode shapes, a large base shear is developed normal to the direction of the input and the dynamic base shear in the direction of the input is significantly underestimated as illustrated in Chapter 15.

As indicated by Table 17.5, the CQC method of modal combination eliminates problems associated with the SRSS method. Also, it clearly illustrates that the directions of 38.64 and -51.36 degrees are a good definition of the principal directions for this structure. Note that the directions of the base shears of the first two modes differ by 90.00 degrees.

Table 17.5. Dynamic Base Forces Using The CQC Method

	BASE SHEARS		OVERTURNING MOMENTS	
Angle -deg	**V_1**	**V_2**	**M_1**	**M_2**
0	78.1	20.4	1202	4116
90	79.4	20.4	1202	4199
38.64	78.5	0.2	3.4	4145
-51.36	84.2	0.2	3.4	4503

Table 17.6 summarizes the scaled dynamic base forces to be used as the basis for design by two different methods.

Table 17.6 Normalized Base Forces In Principal Directions

	38.64 Degrees		-51.36 Degrees	
	V (kips)	M(ft-kips)	V (kips)	M(ft-kips)
Static Code Forces	279.9	14,533	281.2	14,979
Dynamic Design Forces Scaled by Base Shear 279.9/78.5 = 3.57	279.9	14,732	299.2	16,004

For this case, the input spectra scale factor of 3.57 should be used for all directions and is based on the fact that both the dynamic base shears and the dynamic overturning moments must not be less than the static code forces. This approach is clearly more conservative than the approach suggested by the current Uniform Building Code. It is apparent that the use of different scale factors for a design spectra in the two different directions, as allowed by the code, results in a design that has a weak direction relative to the other principle direction.

17.6. DYNAMIC ANALYSIS METHOD SUMMARY

In this section, a dynamic analysis method is summarized that produces unique design displacements and member forces which will satisfy the current Uniform Building Code. It can be used for both regular and irregular structures. The major steps in the approach are as follows:

1. A three dimensional computer model must be created in which all significant structural elements are modeled. This model should be used in the early phases of design since it can be used for both static and dynamic loads.

2. The three dimensional mode shapes should be repeatedly evaluated during the design of the structure. The directional and torsional properties of the mode shapes can be used to improve the design. A well-designed structure should have a minimum amount of torsion in the mode shapes associated with the lower frequencies of the structure.

3. The direction of the base reaction of the mode shape associated with the fundamental frequency of the system is used to define the principal directions of the three dimensional structure.

4. The "design base shear" is based on the longest period obtained from the computer model, except when limited to 1.3 or 1.4 times the Method A calculated period.

5. Using the CQC method, the "dynamic base shears" are calculated in each principal direction due to 100 percent of the Normalized Spectra Shapes. Use the minimum value of the base shear in the principal directions to produce one "scaled design spectra".

6. The dynamic displacements and member forces are calculated using the SRSS value of 100 percent of the scaled design spectra applied non-concurrently in any two orthogonal directions as presented in Chapter 15.

7. A pure torsion static load condition is produced using the suggested vertical lateral load distribution defined in the code.

8. The member design forces are calculated using the following load combination rule:

$$\mathbf{F_{DESIGN} = F_{DEAD\ LOAD} \pm [\, F_{DYNAMIC} + |\, F_{TORSION}\,|\,] + F_{OTHER}}$$

The dynamic forces are always positive and the accidental torsional forces must always increase the value of force. If vertical dynamic loads are to be considered, a dead load factor can be applied.

One can justify many other methods of analyses that will satisfy the current code. The approach presented in this chapter can be used directly with the computer programs ETABS and SAP2000 with their steel and concrete post-processors. Since these programs have very large capacities and operate on personal computers, it is possible for a structural engineer to investigate a large number of different designs very rapidly with a minimum expenditure of manpower and computer time.

17.7. SUMMARY

After being associated with the three dimensional dynamic analysis and design of a large number of structures during the past 40 years, the author would like to take this opportunity to offer some constructive comments on the lateral load requirements of the current code.

First: *the use of the "dynamic base shear" as a significant indication of the response of a structure may not be conservative*. An examination of the modal base shears and overturning moments in Tables 17.1 and 17.2 clearly indicates that base shears associated with the shorter periods produce relatively small overturning moments. Therefore, a dynamic analysis, which will contain higher mode response, will always produce a larger dynamic base shear relative to the dynamic overturning moment. Since the code allows all results to be scaled by the ratio of dynamic base shear to the static design base shear, the dynamic overturning moments can be significantly less than the results of a simple static code analysis. A scale factor based on the ratio of the "static design overturning moment" to the "dynamic overturning moment" would be far more logical. The static overturning moment can be calculated by using the static vertical distribution of the design base shear which is currently suggested in the code.

Second: *for irregular structures, the use of the terminology "period (or mode shape) in the direction under consideration" must be discontinued.* The stiffness and mass properties of the structure define the directions of all three dimensional mode shapes. The term "principal direction" should not be used unless it is clearly and uniquely defined.

Third: *the scaling of the results of a dynamic analysis should be re-examined.* The use of site-dependent spectra is encouraged.

Finally: *it is not necessary to distinguish between regular and irregular structures when a three dimensional dynamic analysis is conducted*. If an accurate three dimensional computer model is created, the vertical and horizontal irregularities and known eccentricities of stiffness and mass will cause the displacement and rotational components of the mode shapes to be coupled. A three dimensional dynamic analysis, based on these coupled mode shapes, will produce a far more complex response with larger forces than the response of a regular structure. It is possible to predict the dynamic force distribution in a very irregular structure with the same degree of accuracy and reliability as the evaluation of the force distribution in a very regular structure. Consequently, if the design of an irregular structure is based on a realistic dynamic force distribution, there is no logical reason to expect that it will be any less earthquake resistant than a regular structure which was designed using the same dynamic loading. A reason why many irregular structures have a documented record of poor performance during earthquakes is that their designs were often based on approximate two dimensional static analyses.

One major advantage of the modeling method presented in this chapter is that one set of dynamic design forces, including the effects of accidental torsion, is produced with one computer run. Of greater significance, however, is the resulting structural design has equal resistance to seismic motions from all possible directions.

17.8. REFERENCES

1. *Recommended Lateral Force Requirements and Commentary, 1996 Sixth Edition*, Seismology Committee, Structural Engineers Association of California, Tel. 916-427-3647.