SEISMIC DESIGN GUIDELINES FOR ESSENTIAL BUILDINGS

Sigmund A. Freeman (I)
Joseph P. Nicoletti (II)
George Matsumura (III)
Presenting Author: S. A. Freeman

SUMMARY

This paper describes the development of a supplement to the 1982 edition of the U. S. Army, Navy and Air Force manual "Seismic Design for Buildings." The proposed guidelines, which are being developed for essential and critical facilities, provide a dynamic analysis approach that is also applicable to irregular buildings and other non-critical buildings as an alternative to static force procedures. Two levels of earthquake motion are considered. At the first level, the structure is designed to remain elastic to provide damage control for a moderate earthquake. At the second level, the criterion requires that the structure remains functional after a major earthquake.

INTRODUCTION

During the 1971 earthquake in San Fernando, California, a substantial amount of structural and nonstructural damage occurred to facilities deemed essential to public welfare for post-earthquake operations. This led to a demand that codes be modified to provide for better earthquake resistive performance. In addition to modifying the lateral force formulas, the I-coefficient was introduced into building codes (Ref. 1) in the mid-1970's to provide for an assignment of higher force levels to structures housing certain facilities, such as hospitals, communication centers, firefighting stations and buildings intended to house other disaster-related services. However, it has been acknowledged by the Seismology Committee of the Structural Engineers Association of California (SEAOC) "that better performance of buildings is not totally equated to increasing the design force level. The basic concepts and configurations of the lateral force resisting system may be more important than the design force level, but consideratons impractical to codify are involved. The lateral force resisting systems in whole, and in part, must be well detailed to accommodate seismic demands."(Ref. 2).

In the 1982 edition of "Seismic Design for Buildings" (Ref. 3), the Tri-Services Committee of the Departments of the Army, the Navy and the Air Force incorporated the latest SEAOC "Recommended Lateral Force Requirements and Commentary" into their seismic design provisions (Ref. 2). These static force seismic design provisions include the I-coefficient for essential facilities; however, alternative procedures to effect a more realistic design concept may

---

(I) Senior Consultant and Branch Manager, Wiss, Janney, Elstner Associates, Emeryville, California, USA

(II) Executive Vice President, URS/Blume and Associates, San Francisco, California, USA

(III) Office of the Chief of Engineers (OCE), U. S. Army (DAEN-ECE-D), Washington, D.C., USA

be used upon approval of properly substantiated technical data for establishing lateral forces and force distribution by dynamic analysis.

To provide guidelines for dynamic analysis procedures of seismic design for essential buildings, the Tri-Services Committee is developing a supplementary manual, "Seismic Design Guidelines for Essential Buildings" (Ref. 4), that is scheduled for completon in 1984.

CURRENT CRITERIA FOR SEISMIC DESIGN

The basic seismic criteria for the design of military construction are established by the Tri-Services manual (Ref. 3). This manual furnishes guidance for the design of buildings, some structures other than buildings, mechanical and electrical equipment supports and utility systems in areas subject to damaging earthquakes. The seismic design and detail requirements are based on SEAOC Recommendations (Ref. 2). The seismic provisions of the Uniform Building Code (Ref. 1) are also adopted from the SEAOC Recommendations. The basic purpose of the building code is to provide for public safety. The intent of these seismic code provisions is to have buildings perform as follows:

1. Respond to minor earthquakes within the elastic limits of the structural elements and without damage.

2. Withstand moderate earthquakes with damage to nonstructural elements, but little or no damage to the structural elements.

3. Resist major or severe earthquakes without major failure of the building or its component members and equipment, and to maintain life safety.

Performance criteria for essential buildings that are vitally needed for post-disaster recovery and continuous operation during and after an earthquake are not expressly stated in these provisions.

The seismic provisions of the building codes (e.g., Ref. 1, 2, 3, 5) prescribe lateral forces that are in the general range of forces associated with minor earthquakes. Building codes attempt to provide the additional capacity required for major earthquakes by means of ductile detailing requirements, restrictions on irregular structures, and higher coefficients on susceptible types of structural systems. However, the designer is not required to evaluate the structure for its overall lateral resistance capacity to resist major earthquake motion. Therefore, there is some uncertainty on how code designed buildings will actually perform during a major earthquake. Although some buildings may not perform as well as intended, in general, buildings can survive earthquakes substantially greater than those represented by building code forces (Ref. 6).

DYNAMIC ANALYSIS PROCEDURE FOR SEISMIC DESIGN

The proposed "Seismic Design Guidelines for Essential Buildings" (Ref. 4) provides a two-level dynamic analysis approach to design buildings. First, the building is designed to resist the lower level of earthquake motion by elastic behavior. Then the building is evaluated for its ability to resist the higher level earthquake with allowances for inelastic behavior.

Design Earthquakes

The first level of earthquake motion, designated EQ-I, is specified as having a 50 percent probability of being exceeded in 50 years. This represents a moderately large earthquake for the particular site of the building. The second level of earthquake motion, designated EQ-II, is specified as having a 10 percent probability of being exceeded in 100 years. This represents the major earthquake for the building site. The principal method of describing these ground motions is in the form of acceleration response spectra. Methodologies for determining the site dependent earthquake ground motions are given in the guideline publication (Ref. 4). For earthquake areas with certain soil profiles, epicentral distances, and magnitude conditions (Ref. 4), the design response spectra may be determined from an adaptation of data in the ATC 3-06 publication (Ref. 7).

First Level Earthquake

The structure is to be designed to resist the forces of EQ-I within the elastic range of the capacity of the lateral force resisting system. The general procedure requires a trial-and-error process because the magnitude and distribution of seismic forces depend on the weight, periods of vibration and mode shapes of the structure. Thus, an approximation of the building characteristics is required before the design forces can be calculated. The selection of trial structural member sizes can be made on a manner similar to that of conventional static design procedures. Structural member forces are calculated by means of a modal analysis using the EQ-I response spectrum with the damping prescribed in Table 1. The member forces are compared to the elastic capacities of the structural elements. All building components are designed to provide yield strength capacities sufficient to resist the combined effects of the seismic forces and applicable gravity loads. A load factor of 1.2 is placed on the dead load to account for possible vertical components of seismic force. Live and seismic loads are given a load factor of 1.0. Some slight flexural yielding of a limited number of structural components may be acceptable on the condition that the elastic-linear behavior of the overall structure is not substantially altered. An interstory drift limit not to exceed 0.005 times the story height is specified. Upon completion of the first level seismic design, the structure is evaluated for the second level earthquake.

Second Level Earthquake

The structure that was designed to resist the forces of EQ-I elastically is now evaluated to determine its performance characteristics when subjected to the demands of EQ-II. Two acceptable procedures are presented. One is an elastic analysis procedure that evaluates overstress ratios and the other is an approximate inelastic analysis procedure that evaluates lateral distortion limits. Load factors for the second level earthquake are equal to unity and live loads may be reduced to realistic actual conditons, which can be as low as 25 percent of the design live load. Interstory drifts are limited to 0.010 times the story height.

Method 1 - Elastic Analysis Procedure: The elastic analysis procedure is acceptable when the equivalent forces of EQ-II are less than about double the equivalent forces of EQ-I and when the structural system is redundant so that a limited number of members form yield hinges and the overall structure responds in an elastic manner. The structural member forces are calculated by means of a modal analysis using the EQ-II response spectrum with the damping prescribed in Table 1. The damping value for the EQ-II spectrum is generally higher than the damping value used with EQ-I. Also, the mathematical model of the structure may be revised to account for some inelastic distortions associated with EQ-II, thus resulting in longer natural periods of vibration. The calculated elastic structural member forces (demand) are compared to yield capacities of the structural members. Inelastic demand ratios, a ratio of the demand forces to the yield capacity, are calculated for all structural elements of the lateral force resisting system. The inelastic demand ratios are evaluated for the following conditions: exceeding the prescribed maximum values shown in Table 2, unsymmetrical yielding on a horizontal plane, forming column mechanisms that cause instability, and unusual distributions. If all the conditions are within prescribed limits, the structure is considered to satisfy the provisions of the seismic design criteria. If the conditions are not met, structural modifications are required or the approximate inelastic analysis procedure must be used.

Method 2 - Approximate Inelastic Analysis Procedure: A step-by-step approach is used to approximate the inelastic capacity of the structure. First, the structure is analyzed to determine the lateral force level that is required to cause first major yielding of the structure. Next, the stiffness characteristics of all structural elements that are within 10 percent of their yield capacities are revised to represent plastic hinges. Then, additional lateral forces are applied to the structure until an additional group of structural elements reaches its yield capacities. The process is repeated until the combined results reach an ultimate limit governed by a mechanism, instability or excessive distortion. The results are converted to a capacity curve based on the periods and spectral accelerations for the fundamental mode of vibration. A graphical solution is used to compare the demand of EQ-II with the capacity of the structure.

An example of the graphical solution is shown in Figure 1 based on the results of Table 3. Table 3 shows the procedure to convert the base shear (V) and displacements (d_R) to values of spectral accelerations (S_a), spectral displacements (S_d), and fundamental periods (T). Figure 1a shows the capacity in terms of base shear and lateral roof displacement. The curve goes beyond the elastic limit into the inelastic capacity of the overall structure. Figure 1b shows the capacity curve of Figure 1a in terms of S_a and S_d. The dashed lines show the transformation of the response spectra of Figure 1c. Figure 1c shows the response spectra for EQ-I and EQ-II. The dashed line is the transformation of the capacity curve. Figures 1b and 1c both show that the structure remains elastic for EQ-I and that inelastic response is required for EQ-II. This procedure was first developed for an evaluation of the Naval Facilities at Bremerton, Washington (Ref. 8). Examples of how the procedure is implemented for existing structures are available in References 6 and 9.

CONCLUSIONS

As seismic design codes presently stand, the designer is unaware of how the structure will perform if subjected to severe earthquake type motion. A two-level approach to seismic design will force the designer to evaluate the building performance characteristics and discover possible weak spots that are susceptible to severe damage. The approximate procedures described in the

"Seismic Design Guidelines for Essential Buildings" (Ref. 4) can be practicably applied by design engineers. Several hospitals have been designed by the Army and Navy based on similar two-level design procedures. This type of approach to seismic design can give the practicing engineer more confidence in the design of structures and can reduce the risk of catastrophic damage in the event of a major earthquake.

At the time of this writing (October, 1983), the draft of the "Seismic Design Guidelines for Essential Buildings" is going through its final review and is scheduled to be completed in early 1984. URS/John A. Blume & Associates, Engineers, is under contract to the Department of the Army, Huntsville Division, Corps of Engineers, to develop the guidelines. Joseph Nicoletti is principal-in-charge; Sigmund Freeman of Wiss, Janney, Elstner Associates is project manager; George Matsumura has been the technical monitor for OCE; and Haresh Shah of Stanford University and Theodore Zsutty of San Jose State University are consultants for the development of the ground motion specifications.

REFERENCES

1. "Uniform Building Code", 1976 Edition, International Conference of Building Officials, Whittier, California.

2. "Recommended Lateral Force Requirements and Commentary", Fourth Edition, Seismology Committee, Structural Engineers Association of California, San Francisco, California, 1980.

3. "Seismic Design for Buildings" (Tri-Services Manual), Departments of the Army (TM 5-809-10), Navy (NAVFAC P-355), and Air Force (AFM 88-3, Chapter 13), February, 1982.

4. "Seismic Design Guidelines for Essential Buildings", Departments of the Army, the Navy, and the Air Force. Intermediate Draft, June 30, 1983.

5. "American National Standard Minimum Design Loads for Buildings and Other Structures", ANSI A58.1-1982, American National Standards Institute, New York, New York, 1982.

6. "An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance", ATC-10, Applied Technology Council, Palo Alto, California, November 1982.

7. "Tentative Provisions for the Development of Seismic Regulations for Buildings", ATC 3-06, Applied Technology Council, Palo Alto, California, 1978 (also, National Bureau of Standards, Special Publication 510).

8. Freeman, S. A., Nicoletti, J. P., and Tyrrell, J. V., "Evaluation of Existing Buildings for Seismic Risk - A Case Study of Puget Sound Naval Shipyard, Bremerton, Washington", Proceeding of the U. S. National Conference on Earthquake Engineering - 1975, Earthquake Engineering Research Institute, Berkeley, California, pp. 113-122.

9. Freeman, S. A., "Prediction of Response of Concrete Buildings to Severe Earthquake Motion", Douglas McHenry International Symposium on Concrete and Concrete Structures, SP-55, American Concrete Institute, Detroit, Michigan, 1978. pp. 589-605.

TABLE 1: DAMPING VALUES FOR STRUCTURAL SYSTEMS

Structural System	Elastic-Linear	Post Yield
Structural Steel	3%	7%
Reinforced Concrete	5%	10%
Masonry Shear Walls	7%	12%
Wood	10%	15%
Dual Systems	(1)	(2)

1. Use the value of the primary, or more rigid system. If both systems are participating significantly, a weighted value, proportionate to the relative participation of each system, may be used.
2. The value for the system with the higher damping value may be used.

TABLE 2: MAXIMUM ALLOWABLE INELASTIC DEMAND RATIOS

Building System	Element	Critical Facilities		Others
		Essential (i.e.,I=1.5)	High Risk (i.e.,I=1.25)	(i.e.;I=1.0)
Steel DMRSF	Beams	2.5	3.0	4.5
	Columns[1]	1.5	1.75	2.0
Braced Frames	Bracing[2]	1.5	1.75	1.75
	Connections	1.25	1.50	1.50
Concrete DMSF	Beams	2.5	3.0	4.5
	Columns[1]	1.5	1.75	2.0
Concrete Walls	In Shear	1.5	1.75	2.0
	In Flexure	2.5	3.0	4.0
Masonry Walls	Shear	1.25	1.5	1.5
	Flexure	1.75	2.0	2.0
Wood	Trusses	1.5	1.75	2.0
	Columns[1]	1.5	1.75	2.0
	Shear Walls	2.0	2.50	3.0
	Connections[3]	1.25	1.50	2.0

[1] In no case will axial loads exceed the elastic buckling capacity.

[2] Allowable values reduced for K-bracing and other concentric bracing systems that depend on compression diagonals without an equal number of tension diagonals for each direction of applied lateral loads.

[3] Other than nails.

NOTE: VALUES IN TABLES 1 AND 2 ARE TENTATIVE, SUBJECT TO REVISION PRIOR TO FINAL PUBLICATION OF DOCUMENT (REF. 4).

TABLE 3: CONVERT BASE SHEAR - ROOF DISPLACEMENT
TO
SPECTRAL ACCELERATION - SPECTRAL DISPLACEMENT - PERIOD

V (kips)	d_R (in.)	C_B	$\frac{d_R}{S_d}$	$\frac{C_B}{S_a}$	S_a (g)	S_d (in.)	T (sec)
2200	2.28	0.22	1.30	0.78	0.280	1.75	0.80
2600	3.05	0.26	1.28	0.80	0.325	2.38	0.86
2800	4.10	0.28	1.28	0.80	0.350	3.20	0.97
3000	8.69	0.30	1.26	0.83	0.361	6.90	1.40

$C_B = V/W$: V = Base Shear, W = Weight = 10,000 Kips

d_R = Lateral roof displacement due to V

$d_R/S_d = (\Sigma m\phi)(\phi_R)/(\Sigma m\phi^2)$, modal roof participation factor

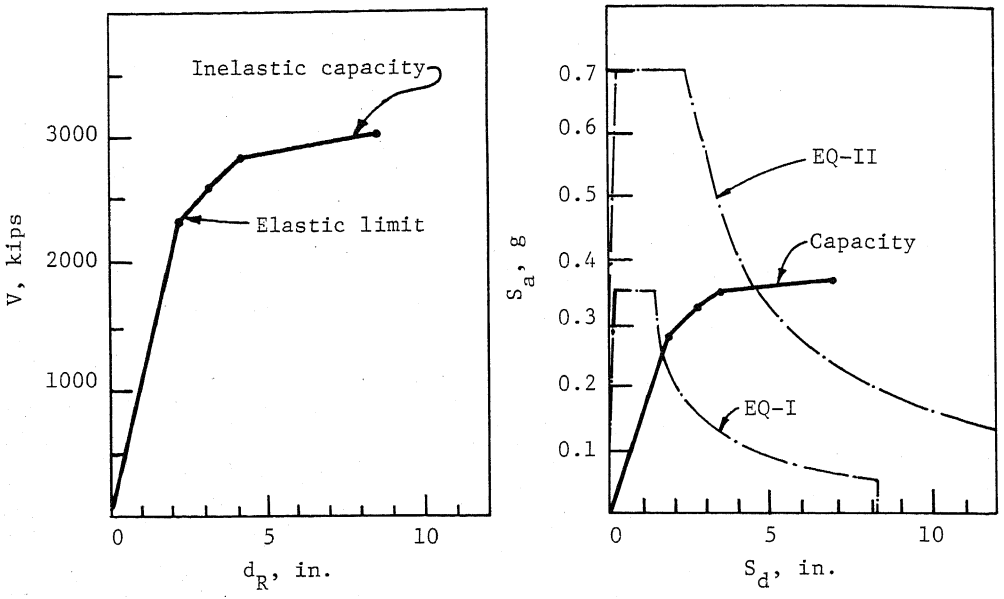
$C_B/S_a = (\Sigma m\phi)^2/(\Sigma m)(\Sigma m\phi^2)$, effective modal weight

S_a = Spectral acceleration

S_d = Spectral displacement

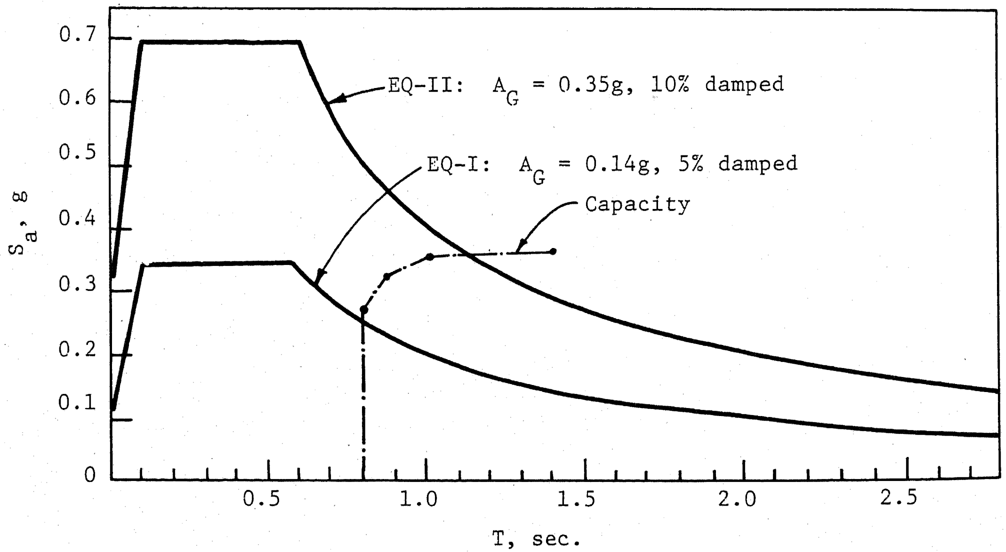
$T = 2\pi\sqrt{S_d/(S_a)(g)}$, fundamental period of vibration

$\Sigma m\phi$ = Summation of story mass time mode shape factor from the roof to the base of the building



(a) V vs. d_R

(b) S_a vs. S_d



(c) S_a vs, T

FIGURE 1: CAPACITY AND DEMAND CURVES (see Table 3 for definitions)