1084

STRUCTURAL DESIGN FOR EARTHQUAKE PROTECTION

by

Warren F. Rosebraugh

B.S., Civil Engineering, 1972

Kansas State University

A MASTER'S REPORT

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY

Manhattan, Kansas

1977

Approved by:

Wayne W. Williams

Major Professor

TABLE OF CONTENTS

THIS BOOK CONTAINS NUMEROUS PAGES WITH DIAGRAMS THAT ARE CROOKED COMPARED TO THE REST OF THE INFORMATION ON THE PAGE.

THIS IS AS RECEIVED FROM CUSTOMER.

LIST OF FIGURES

Figure

PART I

INTRODUCTION

Purpose of the Report

The purpose of this report is to provide an overview of the procedures currently being used to deal with seismic risk in the design of buildings. Primary emphasis will be placed on the general approach taken in the determination of earthquake forces, building design for earthquake resistance, and on the use of the Uniform Building Code. (1)

Scope of the Report

This report covers the important aspects of earthquake force determination, the design of earthquake resistant, multistory buildings, and the current building code requirements.

The report is not intended to provide detailed design procedures, as this information is readily available in design manuals.

PART II

LITERATURE REVIEW

Earthquake Forces

The causes of earthquakes are not completely understood, but the evidence that has been gathered tends to support the elastic rebound theory developed by Harry Fielding Reid around the time of the San Fransisco earthquake in 1906. According to this theory, distortions occur gradually in the earth's crust. As a result, stresses "...build up with the passage of time until the stress at some location becomes great enough to fracture the rock or to cause it to slip along some previously existing fault plane. Slippage at one location causes an increase in the stress in the adjacent rock, so that the slippage propagates rapidly along the fault plane. The result is a sudden rebound of the elastic strain, and the strain energy that had accumulated in the rock is suddenly released and propagated in all directions from the source in a series of shock waves." (2)

Three distinctive wave forms are generated by an earthquake. (3) The first wave to arrive at a given location is the P, or pressure wave. It is characterized by a push-pull type of particle motion parallel to the direction of the wave front. After a period of relative quiet, the S, or shear wave arrives. It results in a transverse particle displacement normal to the direction of the wave front. These first two events are referred to by Lamb (4) as the minor tremor. The major tremor then occurs with the arrival of the R, or Rayleigh wave. It is characterized by both vertical and horizontal particle motion. The effects of these waves on a particle on the surface of an elastic half-space are shown in Figure 1, below, from Richart (3).

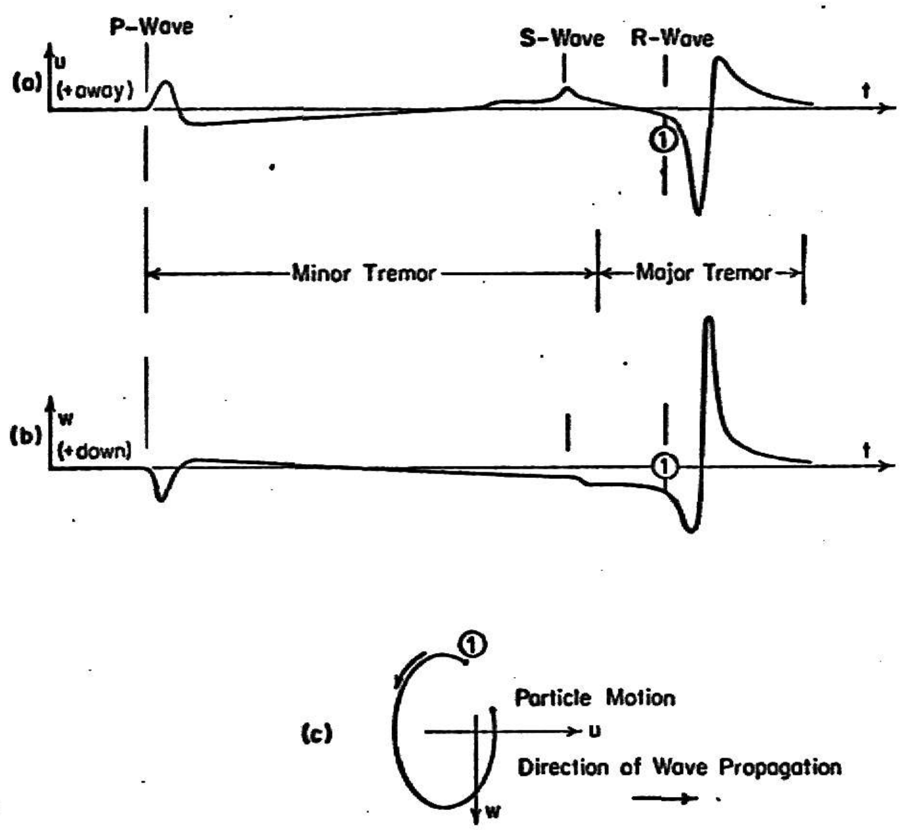

Fig. 1 Effects of Rayleigh wave on particle in ideal medium. (a) Horizontal component of particle motion. (b) Vertical component of particle motion. (c) Path of particle motion.

The time interval between wave arrivals increases and the amplitude of the oscillations decreases with increasing distance from the source. Also, the minor tremor decays more rapidly than the major tremor. As a result, the Rayleigh wave has the most significant effect on buildings and other surface objects. In fact, at large distances from the source, it may be the only clearly distinguishable wave (3).

Earthquake Measurement

The Modified Mercalli Intensity scale is the most commonly used system for describing the intensity of earthquakes in the United States. It provides an arbitrary measure of earthquake effects, as described in Figure 2, from Wiegel (5).

The magnitude of an earthquake is measured on a seismograph using the scale originally developed by Charles F. Richter. Because the scale is logarithmic, a one unit increase in magnitude corresponds to about a thirtyfold increase in energy (5).

Richter has correlated Modified Mercalli Intensity with the Richter magnitude scale as shown in Figure 3, from Wiegel (5).

I. Detected only by sensitive instruments.
II. Felt by a few persons at rest, especially on upper floors; delicate suspended objects may swing.
III. Felt noticeably indoors, but not always recognized as a quake; standing autos rock slightly, vibration like passing truck.
IV. Felt indoors by many, outdoors by a few; at night some awaken; dishes, windows, doors disturbed; motor cars rock noticeably.
V. Felt by most people; some breakage of dishes, windows, and plaster; disturbance of tall objects.
VI. Felt by all; many are frightened and run outdoors; falling plaster and chimneys; damage small.
VII. Everybody runs outdoors; damage to buildings varies, depending on quality of construction; noticed by drivers of autos.
VIII. Panel walls thrown out of frames; fall of walls, monuments, chimneys; sand and mud ejected; drivers of autos disturbed.
IX. Buildings shifted off foundations, cracked, thrown out of plumb; ground cracked; underground pipes broken.
X. Most masonry and frame structures destroyed; ground cracked; rails bent; landslides.
XI. New structures remain standing; bridges destroyed; fissures in ground; pipes broken; landslides; rails bent.
XII. Damage total; waves seen on ground surface; lines of sight and level distorted; objects thrown up into air.

Fig. 2 Abridged Modified-Mercalli Intensity Scale.

Richter Magnitude	Mercalli Intensity
2	I-II
3	III
4	V
5	VI-VII
6	VII-VIII
7	IX-X
8	XI

Fig. 3 Correlation of Richter Magnitude and Modified Mercalli Intensity.

<u>Seismic Risk</u>

In the United States, California is often thought of as being the only area where major earthquakes are likely to occur. However, a map showing the known major earthquakes gives a considerably different picture, as shown in Figure 4, from Earthquake Engineering (5).

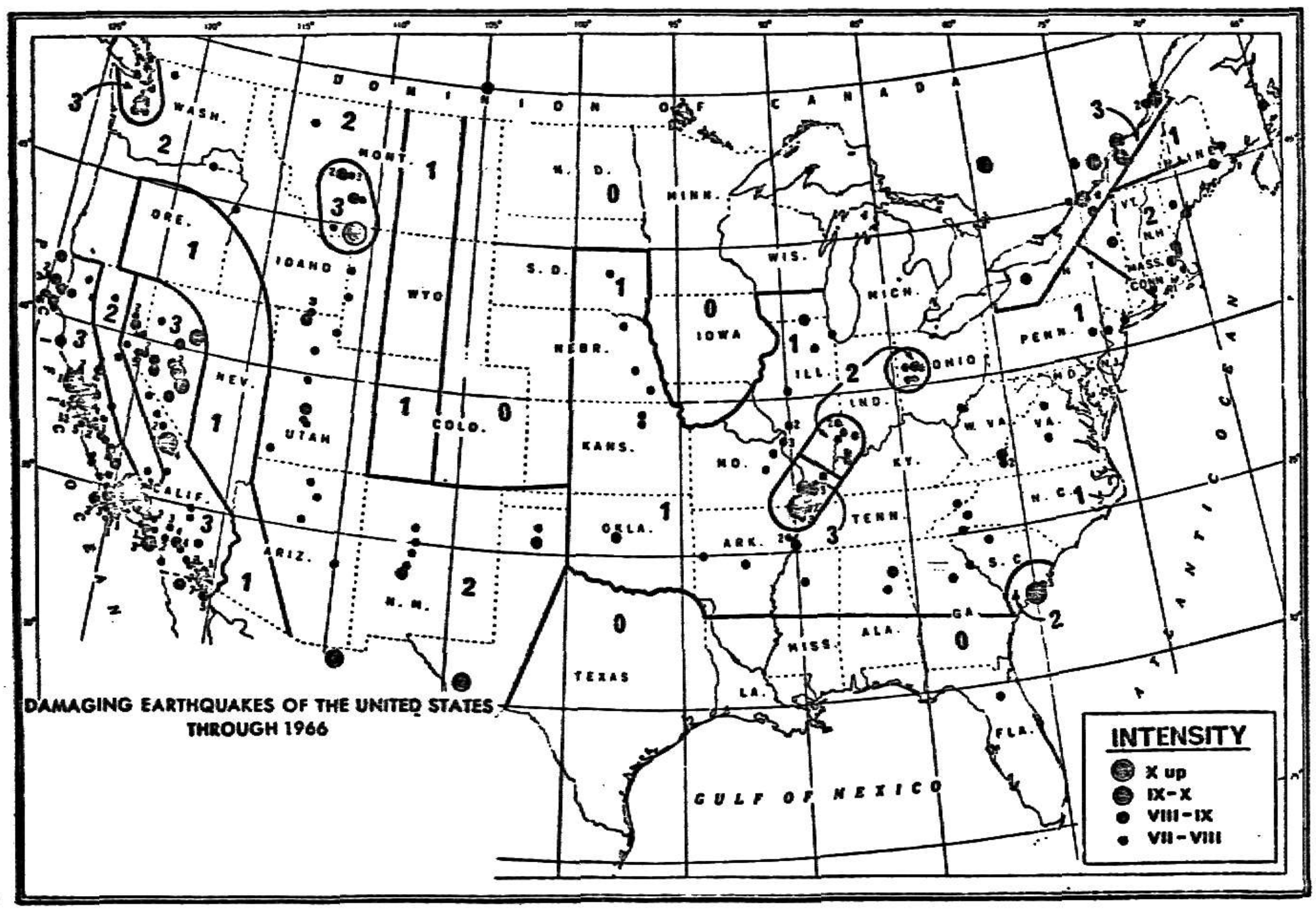

Fig. 4 Locations of damaging historic earthquakes through 1966.

Earthquakes have occurred in nearly all of the states and, in fact, some of the largest earthquakes recorded in the United States occurred near New Madrid, Missouri, and near Charleston, South Carolina (2). The absence of recorded destructive earthquakes in a given area is no reason to assume that it is immune, since major earthquakes can occur 100 or more years apart (5). This suggests that even in regions that appear to be seismically inactive, one should not ignore earthquake forces entirely in the design of structures where failure would be a major catastrophe.

The Uniform Building Code (1) contains a seismic zone map, shown in Figure 5, that is based on the recorded distribution of earthquakes in the United States as well as other geological and statistical considerations. The use of this map in determining earthquake loadings for buildings will be explained in a later section of this report.

Causes of Damage

Structural damage may result from any of several different effects of an earthquake such as tsunamis, soil liquefaction, or landslides. However, "... the principal loading mechanism recognized by seismic design requirements in building codes is the response to the earthquake ground motions applied at the base of the structure." (5)

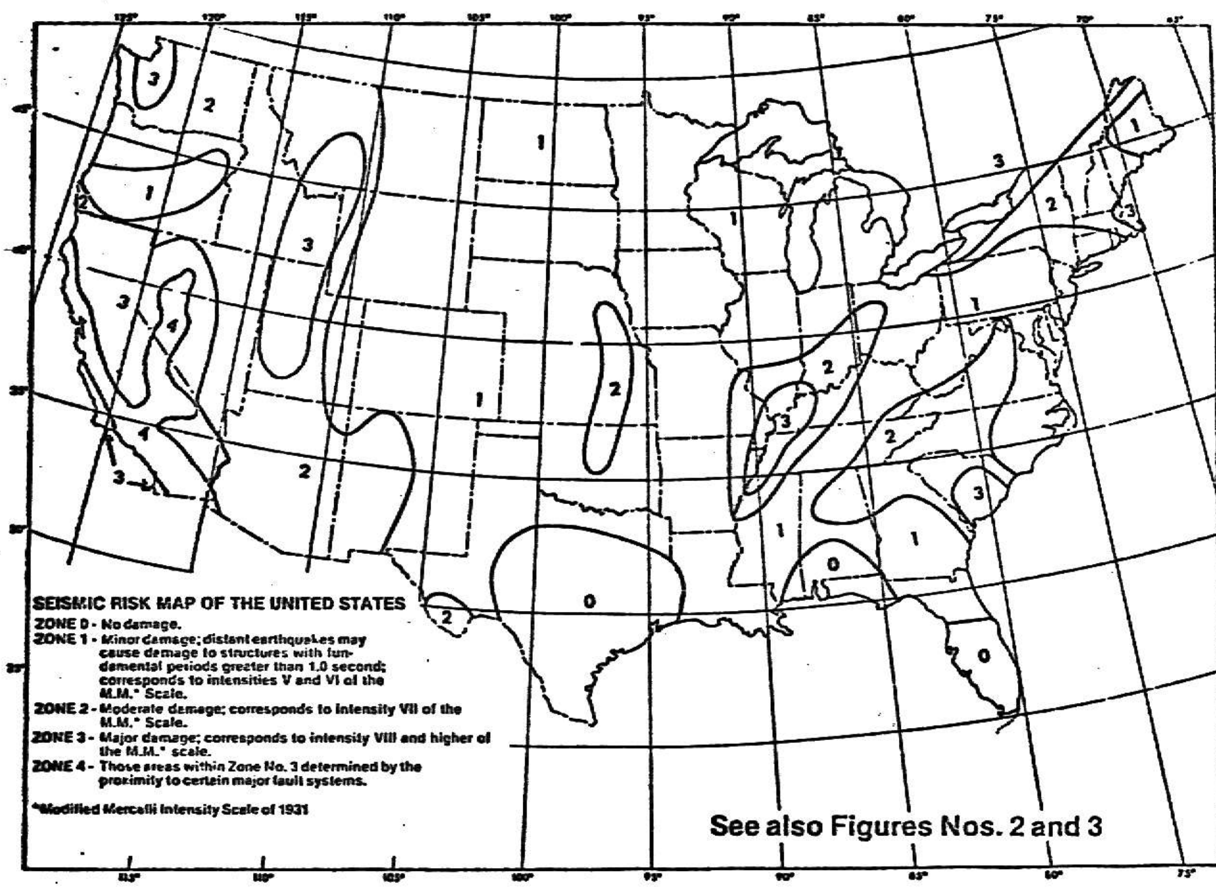

Fig. 5 Seismic zone map of the United States.

The ground motions will be both vertical and horizontal, "... but it is customary to neglect vertical components since most structures have considerable excess strength in the vertical direction through the effect of safety factor requirements." (6)

Structural Forces

The magnitude of the force transmitted to the building depends on both the mass and the acceleration of the building. If the structure and its foundation were completely rigid, the acceleration of the building would equal the acceleration of the ground. Because structures are never completely rigid, the acceleration of a building depends on both the ground motion and the vibrational characteristics of the structure and its foundation (2).

Current building codes covering earthquake design specify forces that are based primarily on the performance of tall buildings that have been subjected to major earthquake loadings. This approach is still necessary because the theoretical design forces obtained from the available ground motion measurements are much too high for the usual elastic methods of analysis. "While great progress has been made on analyses based on elasto-plastic performances, these studies have not progressed to the point where they can be effectively used in the design office for the average building." (5)

Most early studies of vibration effects were concerned primarily with steady state harmonic motion. The results of these studies were impractical, for the most part, because of the nature of the required assumptions (6).

Force Determination

In order to avoid these inherent assumptions, a spectrum approach is now used to evaluate the effects of earthquake ground motions on simplified structures. The spectrum can be demonstrated by the use of a series of cantilever pendulums attached to a movable base, as in Figure 6, from Degenkolb (6).

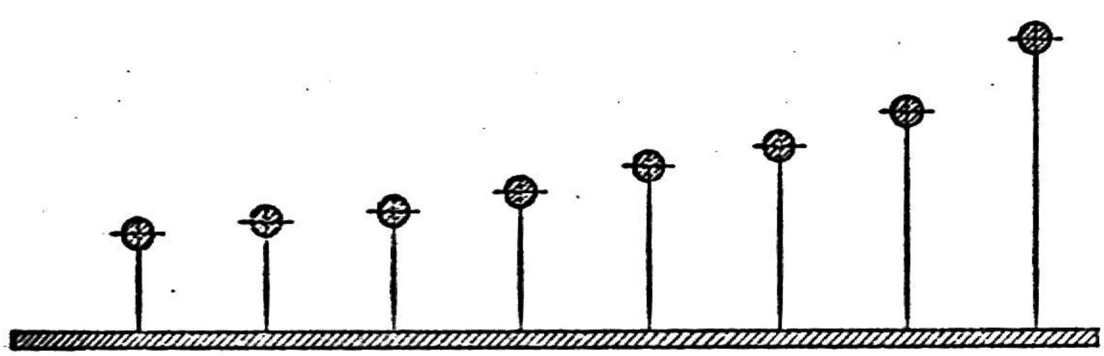

Fig. 6 Pendulums of varying periods.

If the base is moved horizontally in a manner that simulates the ground motion caused by an earthquake, the maximum response of each pendulum can be recorded. If this response is then plotted against the corresponding period for each pendulum, a curve will develop, similar to the one indicated by the solid line in Figure 7, below, from Degenkolb (6).

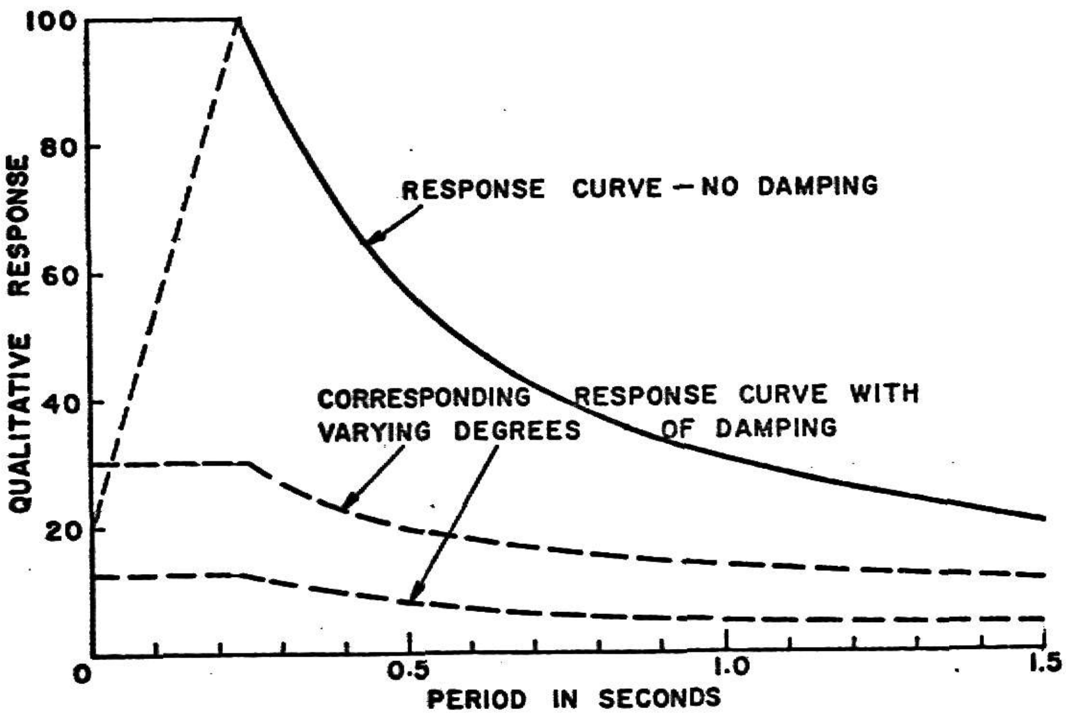

Fig. 7 Effect of damping on qualitative response of a pendulum.

The response can be recorded in terms of deflection or shear, however, equivalent acceleration is used most often. As indicated in Figure 7, damping can greatly reduce the magnitude of the response curve. It can also be seen that a long period of vibration yields a relatively low response value. These concepts, in addition to other factors were used to develop the earthquake code requirements that are now being used.

Several different approximations of the response spectrum curve have been used to yield values of the earthquake coefficient C. The first of these approximations to be used in a code resulted in the Los Angeles Formula which was adopted in 1943. Figure 8, from Degenkolb (6), shows the base shear that results from the use of this formula. The shape of this curve was designed to approximate the shape of the response curve in Figure 7.

Several variations of this same approach have been tried in the ensuing years as new information regarding the performance of tall buildings in moderate to strong earthquakes has become available. The Los Angeles Formula is no longer used because it was not intended to be valid for structures of more than 13 stories in height (6). Most of the subsequent work concerning earthquake coefficients has carried out by the Structural Engineers Association of California.

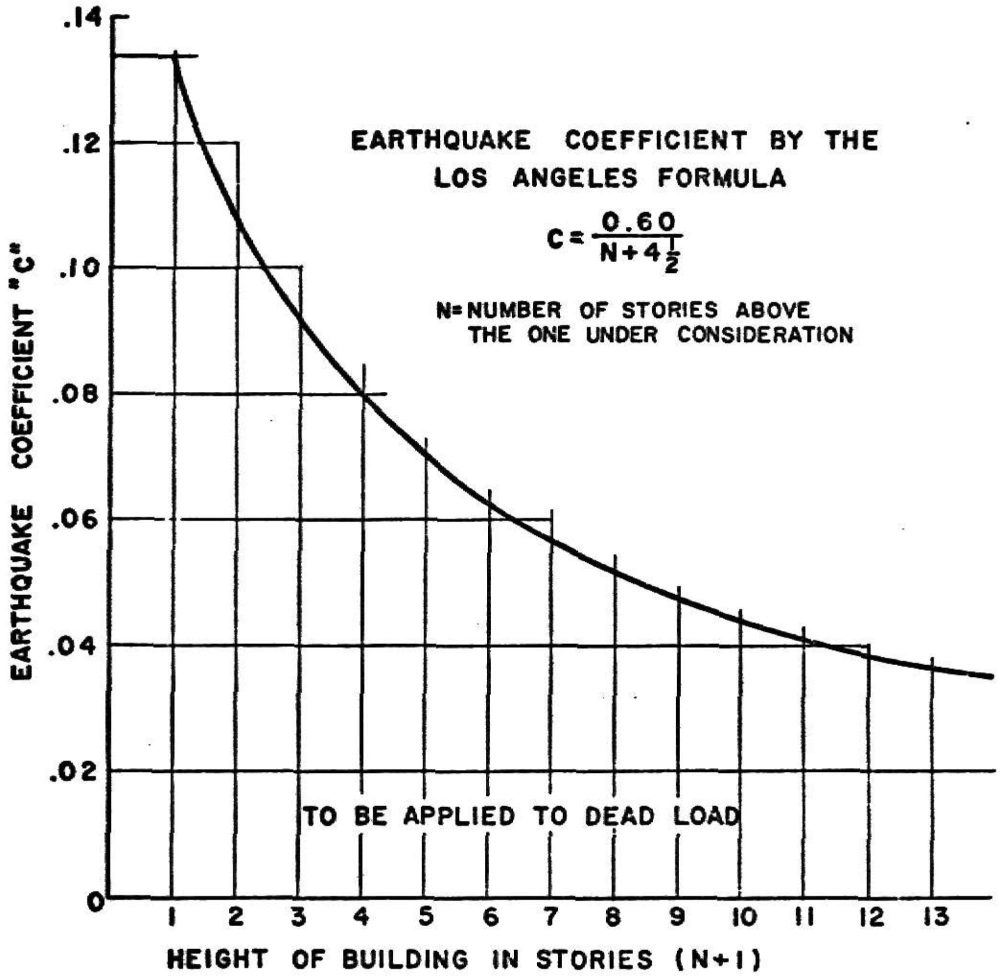

Fig. 8 Earthquake Coefficient resulting from the Los Angeles Formula.

Uniform Building Code Forces

The 1976 edition of the Uniform Building Code (1) specifies the use of the following formula to determine the earthquake coefficient C:

$$C = \frac{1}{15\sqrt{T}} \qquad (12\text{-}2)$$

The numbers used to identify formulas in this report are identical to those in the Uniform Building Code for the purpose of avoiding confusion.

From Figure 9, below, it can be seen that values of C obtained from this new formula do not differ greatly from those of the older formulas.

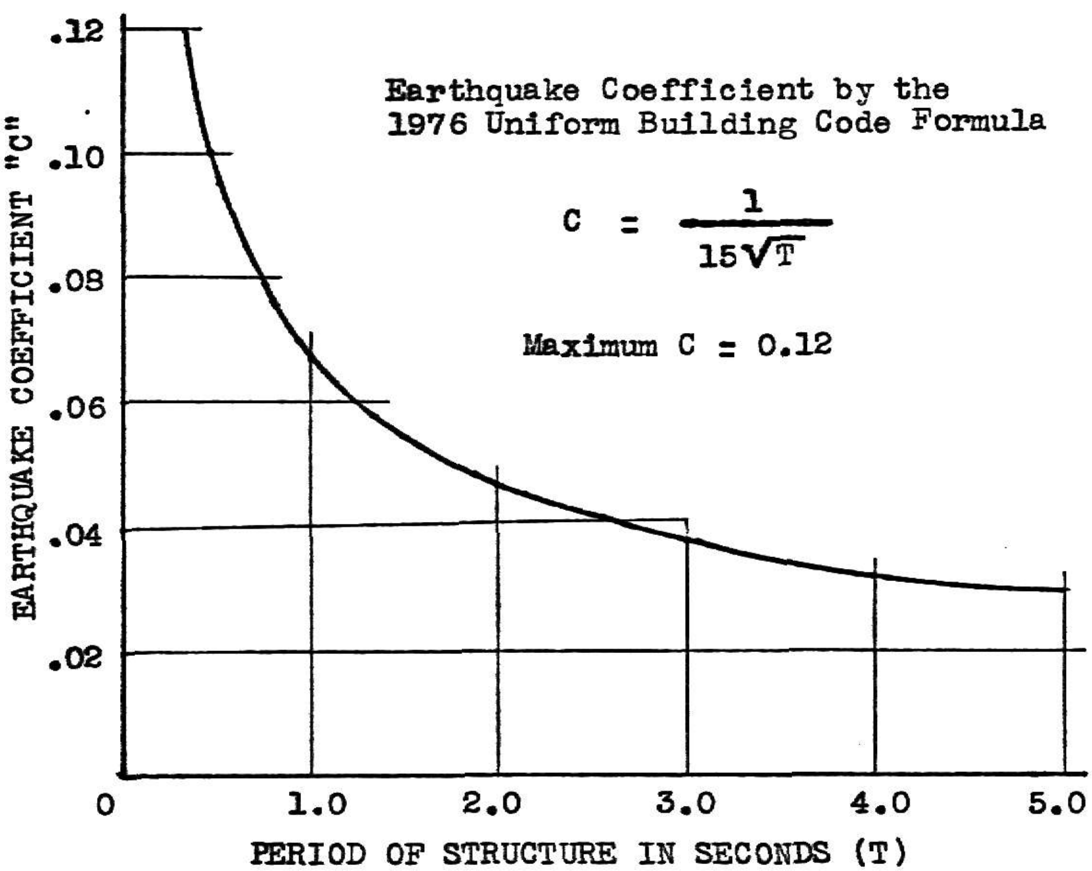

Fig. 9 Earthquake Coefficient resulting from the 1976 Uniform Building Code Formula 12-2.

The value of T, the fundamental elastic period of vibration of the structure in the direction under consideration, can be determined in one of three ways. It can be "... established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis ..." (1).

If this means of determination is not used, T can be calculated using the following formula, where h_n is the height in feet above the base to the n^{th} level, and D is the width of the building, in feet, in the direction parallel to the applied forces:

$$T = \frac{0.05\, h_n}{\sqrt{D}} \qquad (12\text{-}3A)$$

In buildings where "... the lateral force resisting system consists of ductile moment-resisting space frames capable of resisting 100 percent of the required lateral forces and such system is not enclosed by or adjoined by more rigid elements tending to prevent the frame from resisting lateral forces ..." (1), the following formula can be used:

$$T = 0.10\, N \qquad (12\text{-}3B)$$

The total, minimum base shear V that must be resisted by the structure is determined using the following formula:

$$V = ZIKCSW \qquad (12\text{-}1)$$

It is assumed that this force can act in the direction of either of the main axes of the structure, though not in both directions simultaneously. This equation is derived from the formula force equals mass times acceleration, where the acceleration is expressed as a percentage of gravity, modified by various coefficients.

The Z term is the zone coefficient which takes into account the estimated probability of a strong earthquake occuring during the life of the building. The value of Z to be used is found by first determining the zone in which the building under consideration is located, using the map in Figure 5. The zones for Alaska and Hawaii are given in Figures 2 and 3 of the code. For locations in Zone No. 1, $Z = 3/16$; for Zone No. 2, $Z = 3/8$; for Zone No. 3, $Z = 3/4$; for Zone No. 4, $Z = 1$.

The I term in Formula 12-1 is the occupancy importance factor which is determined using table number 23-K of the code. That table is shown in Figure 10, below.

TABLE NO. 23-K
VALUES FOR OCCUPANCY IMPORTANCE FACTOR I

TYPE OF OCCUPANCY	I
Essential Facilities[1]	1.5
Any building where the primary occupancy is for assembly use for more than 300 persons (in one room)	1.25
All others	1.0

Fig. 10 Occupancy Importance Factor.

The essential facilities category refers to "... structures or buildings which must be safe and usable for emergency purposes after an earthquake in order to preserve the health and safety of the general public." (1) This would include hospitals and other medical facilities, fire and police stations, and government disaster operation and communication centers.

The K in Formula 12-1 is the Horizontal Force Factor. This coefficient is used to take into account the observed differences in performance of various types of structures that have been subjected to significant earthquake loads.

TABLE NO. 23-I—HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES[1]

TYPE OR ARRANGEMENT OF RESISTING ELEMENTS	VALUE[2] OF K
1. All building framing systems except as hereinafter classified	1.00
2. Buildings with a box system as specified in Section 2312 (b)	1.33
3. Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls or braced frames using the following design criteria: a. The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames b. The shear walls acting independently of the ductile moment resisting portions of the space frame shall resist the total required lateral forces c. The ductile moment resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force	0.80
4. Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: The ductile moment resisting space frame shall have the capacity to resist the total required lateral force	0.67
5. Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building	2.5[3]
6. Structures other than buildings and other than those set forth in Table No. 23-J	2.00

Fig. 11 Horizontal Force Factor.

As the table in Figure 11 indicates, buildings with ductile space frames designed to resist the total lateral force have performed much better than other types of structures, and as a result they are assigned the lowest value of K.

The S term in Formula 12-1 is a numerical coefficient for site-structure resonance. It is determined using Formulas 12-4 and 12-4A. If T/T_s is 1.0 or less, then

$$S = 1.0 + T/T_s - 0.5(T/T_s)^2. \qquad (12\text{-}4)$$

If T/T_s is greater than 1.0, then

$$S = 1.2 + 0.6(T/T_s) - 0.3(T/T_s)^2. \qquad (12\text{-}4A)$$

The resulting value of S should not be less than 1.0, and the product of CS need not exceed 0.14.

The value of T used in Formulas 12-4 and 12-4A should be determined as discussed previously, but with a minimum value of 0.3 second.

"The range of values of T_s may be established from properly substantiated geotechnical data, in accordance with U. B. C. Standard No. 23-1, except that T_s shall not be taken as less than 0.5 second nor more than 2.5 seconds. T_s shall be that value within the range of site periods, as determined above, that is nearest to T." (1) If T_s is not established in this manner, the value of S should be 1.5. If the value of T is found to be greater than 2.5

seconds, the value of S is determined using a value of 2.5 seconds for T_s.

Force Distribution

Once the proper base shear has been determined, then the shear must be resolved into equivalent static forces that are assumed to act as point loads on each floor of the building. During an earthquake, a building would deflect in one or more possible modes of vibration. Examples of the first three modes are shown in Figure 12, from Degenkolb (6).

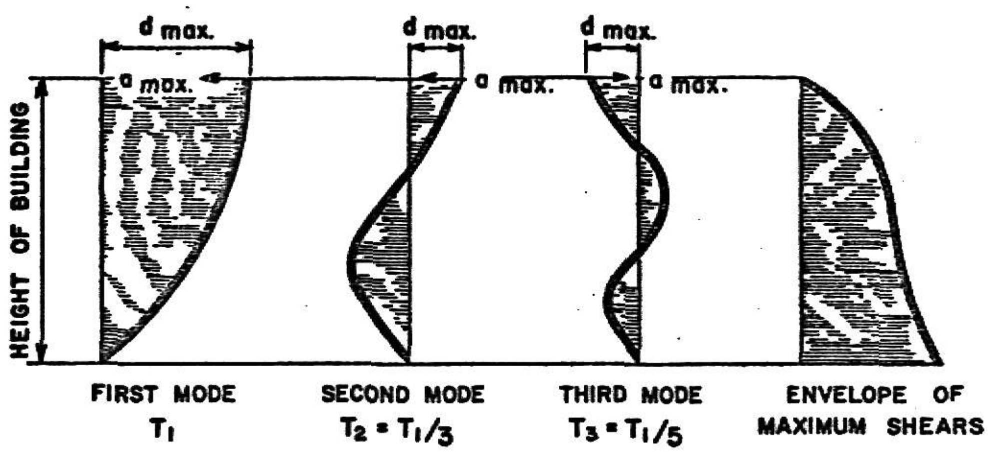

Fig. 12 Natural modes of vibration.

The lateral forces in a structure have "... a distribution whose shape is related to the shape of that mode. This means that during an earthquake the lateral forces at any instant are given by the superposition of the forces associated with each mode provided that there are no large inelastic deformations. These may sometimes add and sometimes cancel. The worst case is the simultaneous occurrence of the maximum vibration of all modes. These cannot be additive at all points at the same time so each point must be considered separately." (6) Shown at the right of Figure 12 is the envelope of maximum shears. It is developed by plotting the maximum dynamic shear for all modes of vibration at each point throughout the height of the building.

Uniform Building Code Force Distribution

The Uniform Building Code (1) specifies the following formula for determining the distribution of the base shear V over the various floors of a building:

$$F_x = \frac{(V - F_t)w_x h_x}{\Sigma\, w_i h_i} \qquad (12\text{-}7)$$

For a building with a uniform section and weight, this formula gives a triangular load distribution as shown at the right of Figure 13, from Degenkolb (6).

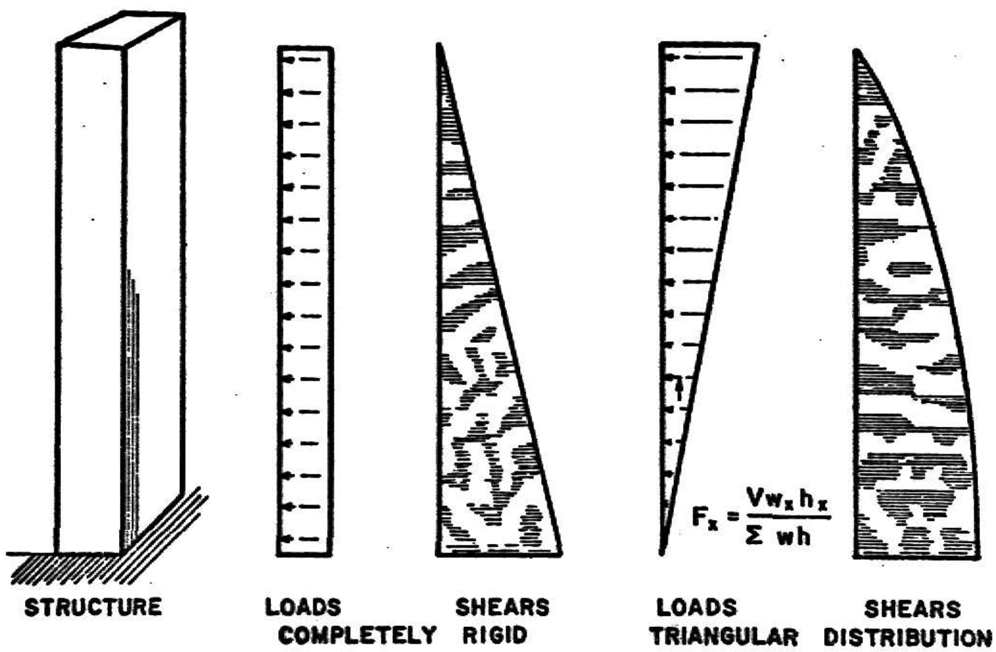

Fig. 13 Load and shear diagrams for a building.

The triangular load distribution of Formula 12-7 is designed to yield a shear distribution that approximates the shape of the envelope of maximum shears shown in Figure 12.

The F_t term in Equation 12-7 is the concentrated load that is assumed to act at the top of the structure. It is calculated using the following equation:

$$F_t = 0.07TV \qquad (12\text{-}6)$$

"F_t need not exceed 0.25V and may be considered as 0 where T is 0.7 second or less." (1)

The terms w_x and h_x in the numerator of Formula 12-7 represent the weight and height, respectively, of the floor under consideration. The denominator of equation 12-7 is the summation of the products of the weights and heights of all the floors of the building.

Earthquake Resistant Design

The engineering profession has tried two basic approaches to the problem of earthquake design. The first approach is to assume a ground motion, then run a dynamic analysis to determine the forces. The loads are then assigned to the resisting elements and the members are proportioned to resist them. This can be done on either an elastic or an elasto-plastic basis. While much progress has been made on this design approach, it is not yet practical for use on a complex structure (5).

The second approach is trial and error. The basis for most of the current earthquake code requirements is the performance of the tall buildings that survived the 1906 San Francisco earthquake. The result of using this second approach is, in effect, an accumulation of desirable and undesirable features for earthquake resistant buildings.

The goal that is most often sought in earthquake protective design is that a building should suffer minimal damage in earthquakes of the intensity that would be expected to occur in the life of the building. The structure should also have sufficient reserve strength to prevent collapse in the event of a severe earthquake.

Flexibility

Flexibility is probably one of the most important considerations in the design of earthquake-resistant buildings. The tall buildings that performed well in the 1906 San Francisco earthquake had flexible steel frames with masonry shear walls that tended to crush and absorb energy. If a structure is ductile enough so that it can dissipate energy in cycles of inelastic deformation, then it can survive an earthquake that would destroy a less flexible building (2).

The favorable effect of ductility on structural performance has been accounted for in the Uniform Building Code with the use of the K term in Equation 12-1.

Although a very flexible building would perform well in a major earthquake, provided it had the strength and stability to withstand large deflections, it could cause problems in the event of high winds or moderate earthquakes. Excessive swaying can result in psychological problems for building occupants and expensive repair bills for the owner if rigid partitions crack and windows break during every wind storm.

As an example, there were several lightweight buildings constructed in Los Angeles using the provisions of the Los Angeles Building Code formula. When the 1952 Kern County earthquakes occurred, the predominant ground motion was one with a relatively long period. Most buildings suffered no damage, but some of the new buildings "... were badly shaken. So badly shaken in fact, that one new building had several hundreds of thousands of dollars worth of plaster and paint damage. There was no structural failure and no hazard to the occupants, but this is small comfort to the owner who faces the prospect of another large repair bill after every minor earthquake." (6)

The provisions of the Uniform Building Code are designed to provide a safe and economical solution to this problem.

Ductile Concrete Design

The studies of structural performance in earthquakes was originally thought to indicate that reinforced concrete was not a satisfactory material for earthquake-resistant design. It is now believed, however, that by the use of ductile concrete (abbreviated DC) and careful construction inspection reinforced concrete can compete with steel for use in earthquake-resistant buildings.

The capacity of a structure, based on its ability to absorb energy, is greatly increased by the use of a ductile rather than a brittle substance. This difference is illustrated in the diagram in Figure 14, from Wiegel (5).

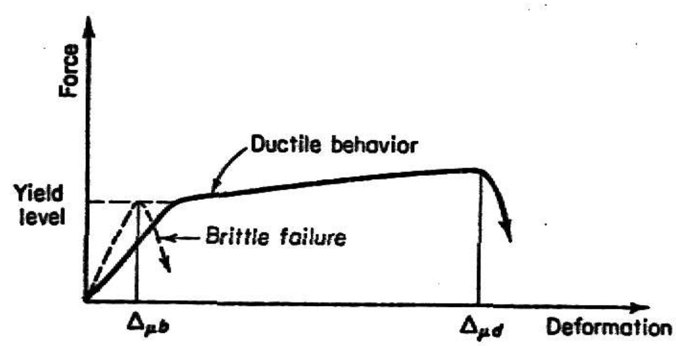

Fig. 14 Ductile versus brittle behavior.

Ductile concrete differs from ordinary reinforced concrete in the amount and location of the reinforcing steel that is required. Ductile concrete is designed "... as to ensure that in flexural members shear failure and compression failure in the concrete cannot occur prior to stretching of the tensile bars, and in compression members shear failure cannot occur and any concrete that fails in compression will be confined." (5)

The design of ductile concrete structures is covered in Sections 2630 and 2631 of the Structural Engineers Association of California code (7).

Steel Design

There are three important aspects of structural steel design that must be considered for earthquake resistance. They are beam-column connections, bending of structural shapes, and column compression and bending.(5).

The beam-column connections were the weakest elements of the steel-framed structures that have been studied following earthquake loadings. Most of these were web connections with top and bottom clip angles. In general, the connections performed well and developed considerable moment resistance at large deformations. They were found to have great ductility but very little rigidity, and were usually assumed to act as hinged connections.

The best connections for moment-resistant steel frames are now considered to be either the fully-welded or the multiple-tee types. Stiffeners are usually required for the connections in high-rise structures (5).

The second important factor affecting the performance of steel-framed structures is the bending capacity of the beams and girders. "In steel, failure is usually by instability and in order to perform satisfactorily in a major earthquake, the moment-rotation curve must develop a long plateau. Ultimate failure usually is not caused by lateral buckling but is triggered by local buckling." (5) The relationship between moment and rotation is shown in Figure 15, from Wiegel (5).

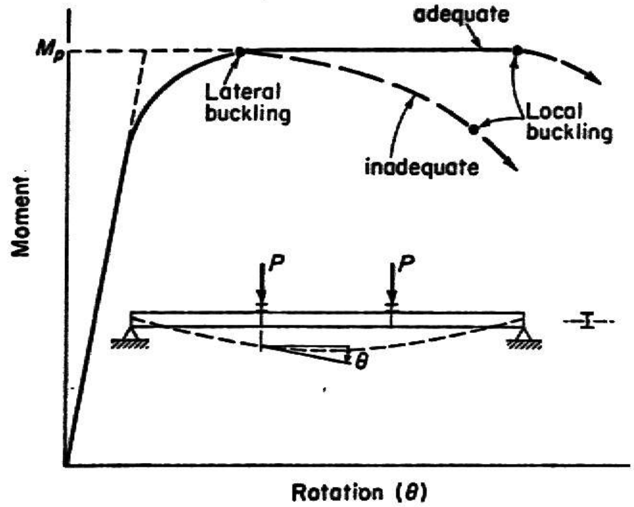

Fig. 15 Beam moment-rotation relationships.

The third basic aspect of steel design that must be considered is column behavior. When designing structures for earthquake resistance, the effect of concurrent axial and moment loadings must be considered. It has been found that "... most usable steel columns have considerable strength in the postbuckling range, sustaining loads greater than the usual design loads at very large deformations." (5) The details of designing columns for earthquake-resistant structures can be found in the work of Galambos and Lay (8).

Tying the Structure Together

"Foremost among requirements vital to earthquake resistant design of all types of buildings is the necessity of tying the building together so that it acts as a unit. Generally, this requirement cannot be stated in detail by a building code, but has to be left to the judgment of the responsible engineer." (6) One important aspect of this problem is illustrated in Figure 16, from Degenkolb (6).

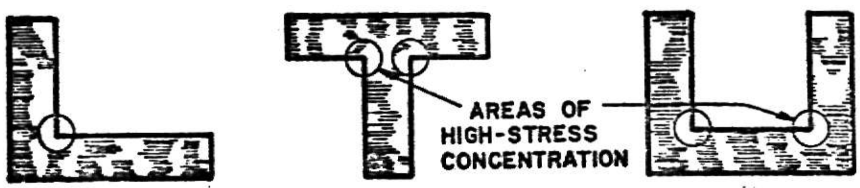

Fig. 16 Plan of irregular-shaped buildings.

Reentrant corners are areas of high stress during an earthquake and must be reinforced accordingly.

Another problem associated with the tying together of structures is pounding. Pounding can occur if two structures with different periods of vibration are built close together. A great deal of damage can occur at the point of contact in an earthquake that would not damage either of the buildings if the pounding were avoided.

It is also very important to anchor all parapets, ornamentation and similar exterior appendages to the building securely. Similarly, walls must be well anchored.

Pile caps and footings must also be tied together.

The Uniform Building Code requires that they "... shall be interconnected by ties, each of which can carry by tension and compression a minimum horizontal force equal to 10 percent of the larger ... loading." (1) If the piles are to be made of concrete, they should be reinforced.

Construction Inspection

As in all aspects of building design, the performance of the completed structure in an earthquake depends as much on the care with which the building is constructed as it does on the design. Inspection is particularly important where ductile reinforced concrete has been specified. Many of the failures of reinforced concrete-framed buildings have been attributed to faulty construction practices (5).

PART III

CONCLUSION

Earthquakes have resulted in considerable death and destruction throughout history. Fortunately, a great deal of progress has been made in recent years in devising ways of providing safe buildings for people in earthquake-prone areas.

Building code requirements for earthquake-resistant design are still empirical in nature because the theoretical approaches have not been developed to the point where they can be applied to complex structures.

Hopefully, the building code provisions will be updated as new earthquake data and analytical techniques become available.

PART IV

APPENDIX

To illustrate the use of the Uniform Building Code for the determination of earthquake forces, the following sample problem is presented.

The problem is based on an assumed building in earthquake zone number two. The building has ten stories, is 100 feet wide, 100 feet long and 120 feet tall. The assumed weights are 2,500 kips per floor, 1,500 kips for the roof and a total weight of 24,000 kips for the building.

The first step in this problem is to determine the period T of the building. Since detailed information regarding the structural properties of the building are not known, equation 12-3A is used.

$$T = 0.05\ h_n / \sqrt{D}$$

$$T = 0.05\ (120) / \sqrt{100}$$

$$T = 0.6 \text{ seconds}$$

Thus T is greater than the required minimum value of 0.3 second.

The coefficient C is then found using equation 12-2.

$$C = 1 / 15 \sqrt{T}$$

$$C = 1 / 15 \sqrt{0.6}$$

$$C = 0.086$$

The value of C is below the maximum value of 0.12.

In order to determine the numerical coefficient for site-structure resonance, it is necessary to calculate the T / T_s ratio. For this purpose, a value of 1.5 seconds has been assumed for the characteristic site period T_s.

$$T / T_s = 0.6 / 1.5$$
$$T / T_s = 0.4$$

Since this value is less than 1.0, equation 12-4 is used to find the numerical coefficient for site-structure resonance S.

$$S = 1.0 + T / T_s - 0.5 (T / T_s)^2$$
$$S = 1.0 + 0.4 - 0.5 (0.4)^2$$
$$S = 1.32$$

The value of S is greater than the required minimum value of 1.0.

The product of CS is found to be 0.113 which is less than the required maximum of 0.14.

The building is assumed to be located in earthquake zone number two. The code specifies that the earthquake coefficient Z is 3/8 for this location.

The Occupancy Importance Factor I is found to be 1.0 from table 23-K if the building is assumed to not be an essential facility.

If the building is assumed to have a ductile space frame with shear walls, the Horizontal Force Factor K is found to be 0.8 from table 23-I.

The total lateral force V at the base of the building can now be calculated using equation 12-1.

$$V = Z\ I\ K\ C\ S\ W$$

$$V = (3/8)(1.0)(0.8)(0.113)(24{,}000)$$

$$V = 817.2 \text{ kips}$$

The portion of V considered concentrated at the top of the structure F_t can now be found using equation 12-6.

$$F_t = 0.07\ T\ V$$

$$F_t = 0.07\ (0.06)(817.2)$$

$$F_t = 34.3 \text{ kips}$$

The lateral force F_x applied to any level x is found using equation 12-7.

$$F_x = (V - F_t)(w_x)(h_x) \ / \sum_{i=1}^{n} (w_i)(h_i)$$

$$F_x = (782.9)(w_x)(h_x) \ / \ 1{,}530{,}000$$

The lateral forces for all of the floors and the roof can now be determined. The results of these calculations can be found in the table on the following page.

Floor No. x	Height (ft.)	Weight (kips)	Force (kips)
2	12	2,500	15.35
3	24	2,500	30.70
4	36	2,500	46.05
5	48	2,500	61.40
6	60	2,500	76.75
7	72	2,500	92.10
8	84	2,500	107.46
9	96	2,500	122.81
10	108	2,500	138.16
Roof	120	1,500	92.11
			782.89
		F_t =	34.30
		V =	817.2

ACKNOWLEDGEMENTS

The author wishes to express his sincere appreciation and gratitude to his major advisor Professor Wayne W. Williams, committee member Dr. Edwin C. Lindly, and to his father Professor Vernon H. Rosebraugh for their advice and encouragement during the preparation of this report.

He would also like to thank Professor I. Eugene Thorson, member of the advisory committee, and Dr. Robert R. Snell, Head of the Department of Civil Engineering.

BIBLIOGRAPHY

1. International Conference of Building Officials (1976). The Uniform Building Code.

2. Berg, Glen V., Designing for Earthquakes, American Iron and Steel Institute, Vol. 2, No. 3,

3. Richart, F. E., Vibrations of Soils and Foundations, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1970.

4. Lamb, H., "On the Propagation of Tremors over the Surface of an Elastic Solid," Philosophical Transactions of the Royal Society, London, Ser. A, Vol. 203.

5. Wiegel, Robert L., Earthquake Engineering, Prentice-Hall Inc., Englewood Cliffs, N. J., 1970.

6. Degenkolb, Henry J., Earthquake Forces on Tall Structures, Bethlehem Steel Co., Booklet 2717.

7. Structural Engineers Association of California, Seismology Committee (1966). Recommended Lateral Force Requirements.

8. Galambos, T. V., and M. G. Lay (May 1962). End-Moment End-Rotation Characteristics for Beam-Columns, Lehigh University, Institute of Research, Fritz Engineering Laboratory Report No. 205A.35.

STRUCTURAL DESIGN FOR

EARTHQUAKE PROTECTION

by

Warren F. Rosebraugh

B.S., Civil Engineering, 1972

Kansas State University

AN ABSTRACT OF A MASTER'S REPORT

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY

Manhattan, Kansas

1977

ABSTRACT

Although most earthquakes in the United States occur in California, earthquake loads cannot be ignored in any state in the design of tall buildings.

The analysis and design of structures for earthquake resistance is still an empirical process even though a great deal of progress has been made in the theoretical approach to the problem. Numerous design considerations intended to reduce structural damage to buildings in the event of a strong earthquake are available in the literature. These are a direct result of the systematic study of the performance of buildings that have been exposed to earthquake loadings.