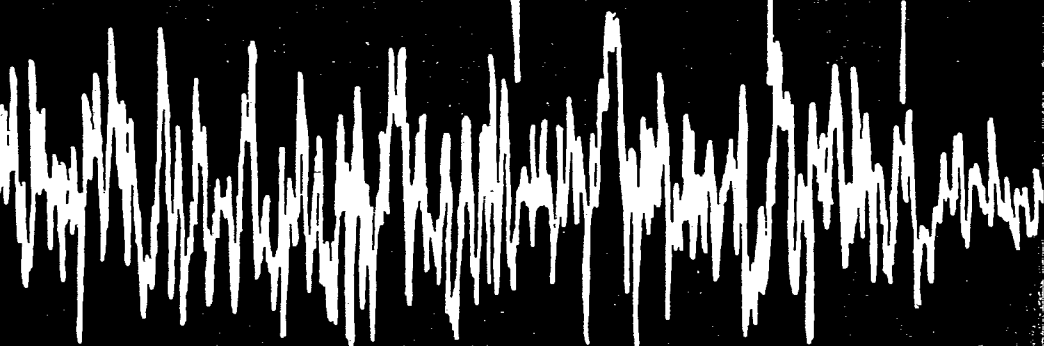
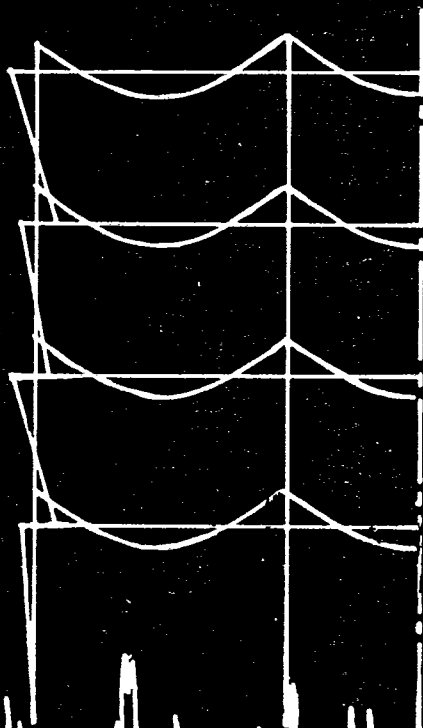


Earthquake-Resistant Concrete Structures Inelastic Response and Design

S.K. Ghosh
Editor



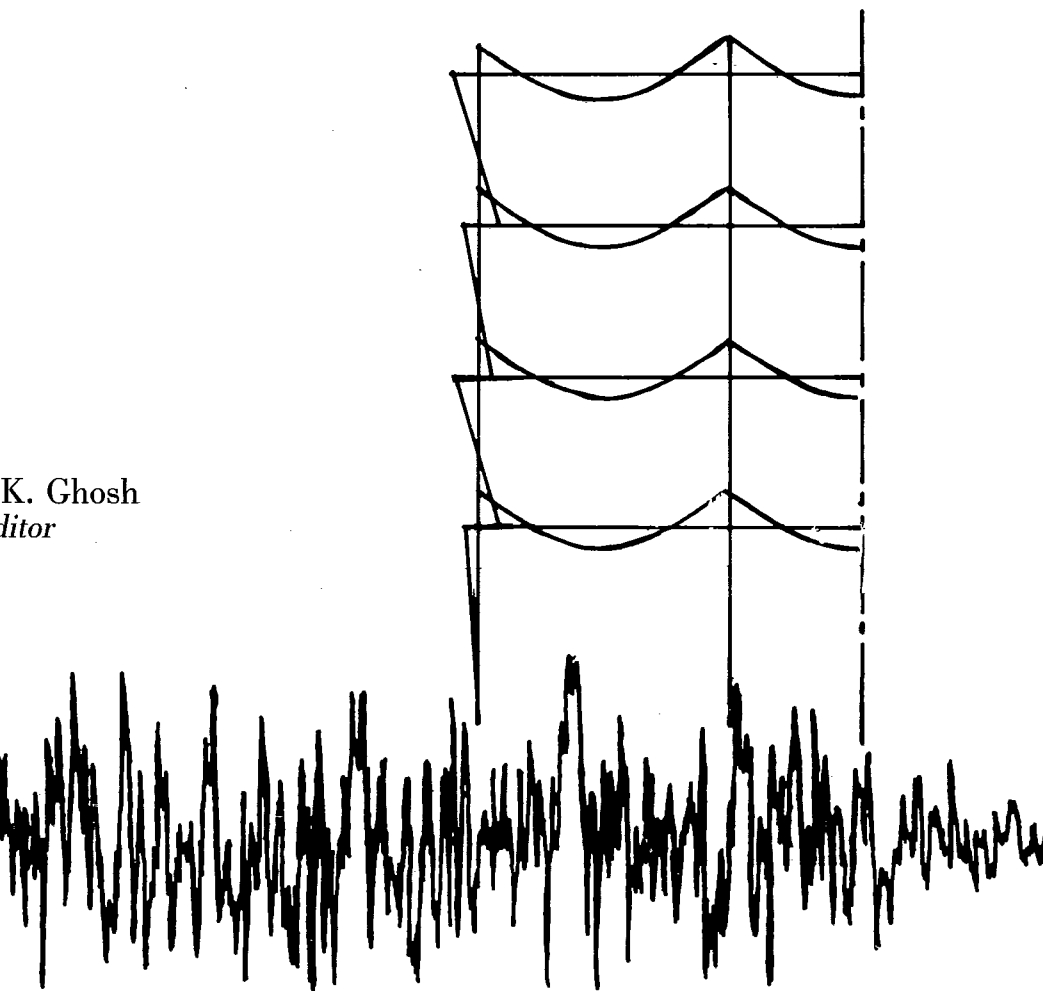
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Earthquake-Resistant Concrete Structures Inelastic Response and Design

K. Ghosh
Editor



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Printed in the United States of America

Editorial production Victoria A. Lunick

Library of Congress catalog card number 91-73196

PREFACE

Almost a decade ago, in 1982, an ad hoc subcommittee on Inelastic Response of Concrete Structures was formed within ACI-ASCE Committee 442, Response of Concrete Buildings to Lateral Forces. The charge of the subcommittee was to develop a state-of-the-art report on Inelastic Design Procedures for Reinforced Concrete Structures Subjected to Earthquakes and Extreme Winds. The scope was later narrowed to earthquakes only. The subcommittee report was developed for use as a guide by those wishing to familiarize themselves with the relatively new tools that permit the utilization of inelasticity in structural members subject to the effects of intense seismic excitation. It is believed that controlled utilization of such inelasticity can result in structures capable of superior earthquake response.

As the various parts of the document began to be assembled, it became clear that the volume was too extensive to be accommodated within a single report, unless drastic reductions were made in the text. The subcommittee felt that the various prepared sections were of such quality that it would be a disservice to the structural engineering profession not to make the material generally available. At that point, approval was sought and received from ACI headquarters to present this subcommittee report as an ACI special publication.

This special publication builds on many topics alluded to in the main committee report, ACI 442R-88. It is also intended as a comprehensive update of many topics discussed in the landmark three-volume proceedings of a workshop in Earthquake-Resistant Reinforced Concrete Building Construction (ERCBC) sponsored by the National Science Foundation, and held at the University of California, Berkeley in July 1977 with Professor V.V. Bertero as organizer. Some of the material contained herein was presented at a session on Inelastic Design of Earthquake-Resistant Concrete Structures at the 1989 ACI Annual Convention in Atlanta in February, 1989, and at another session on Inelastic Response of Concrete Structures to Lateral Forces at the ASCE Structures Congress in San Francisco in May, 1989.

This special publication will, it is hoped, be of interest to engineers not only in regions of high seismicity, but also in areas that have been assigned a higher seismic risk through recent changes in seismic zoning maps in model codes and other related documents. It makes current research results available to design professionals in a convenient form. Properly utilized, the publication will enable them to design new buildings with enhanced seismic safety at no increase in cost (sometimes at a reduced cost). It will also facilitate more accurate evaluation of the seismic safety of existing buildings in regions of moderate to high seismicity.

This special publication has been made possible by the contributions of many individuals and organizations. Mark Fintel, the founding chairman of the ad hoc subcommittee on Inelastic Response of Concrete Structures, gave his vision and early leadership to the project. All contributors to this publication deserve special thanks and gratitude, not only for the outstanding quality of their efforts and their commitment, but also for their extraordinary patience and understanding through the long years it took to finally bring this publication out. This editor is grateful to the many reviewers of the various sections who unstintingly gave of their time. He would like to express his deep gratitude to Professor Richard Klingner, Chairman of ACI-ASCE Committee 442 since 1986, for his outstanding support of the major effort leading to this publication. Finally, it must be acknowledged that this publication would not have been possible without the generous support of the Portland Cement Association.

Skokie, Illinois
April 1991

S.K. Ghosh

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Seismic Design of Reinforced Concrete Structures Considering Inelasticity

by S. K. Ghosh

Synopsis: The purpose and scope of this special publication, which is concerned primarily with departures from and improvement upon code-based seismic design practice, are outlined. The broad underlying principles of wind and seismic design by current codes are discussed. Certain major deficiencies of current practice are pointed out. Comparative features of elastic and inelastic seismic structural response

Keywords: Earthquake-resistant structures; elastic properties; reinforced concrete; standards; structural design; wind pressure.

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PURPOSE

The objective of this publication is to present the state-of-the-art in inelastic dynamic and static approaches to the analysis and design of structures subjected to earthquakes. The report is limited to the discussion of structures which, when designed by the codes, are expected to respond inelastically to intense earthquakes. The major emphasis is on building structures.

This publication is intended as a guide for those wishing to familiarize themselves with the new tools that permit the utilization of inelasticity in structural members. It is believed that controlled utilization of such inelasticity can result in superior earthquake response of structures.

Inelastic dynamic approaches for determining the earthquake response of single- and multi-degree-of-freedom systems have been used as research tools during the past two decades. For practical design, elastic dynamic analysis is used in the nuclear power stations field and occasionally for important commercial structures. Although a specific approach to inelastic dynamic analysis and design has evolved in Japan, where it is used for exceptional structures (1,2), inelastic dynamic approaches have only rarely been used for practical design of structures in North America.

Information on the amount and distribution of internal forces and deformations in yielding structures during a seismic response can be obtained only through inelastic response history analyses of structures subjected to earthquake input motions. While such analytical studies provide estimates on how much inelastic deformability may be required of structural members in earthquake situations, the inelastic deformability attainable with proper member proportioning and detailing can be estimated only through laboratory testing of structural systems, subsystems, and elements. An impressive amount of experimental research has provided much needed information on the hysteretic behavior, including attainable strength, stiffness, and deformability, of structural elements and

subassemblies. Suitable detailing must be prescribed for potential inelastic regions, so that capacity for inelastic deformation exceeds deformation demands consistent with stability.

Design utilizing inelastic response history analysis offers the possibility to control the extent of structural damage through limitations on inelastic deformations of members. Limits can also be set on interstory drift to control structural and nonstructural damage. Imposing on the structures desirable sequence of member yielding, while simultaneously taking care not to exceed the limits set on inelastic deformations, seems to be a realistic possibility in designs utilizing inelastic response history analysis, although such analysis is not a prerequisite to attainment of a desirable yielding sequence. This aim can be achieved by establishing a rational strength hierarchy between yielding and elastic members.

SCOPE

This publication consists of nine parts: introduction; description of earthquake ground motion; structural configurations, and lessons from earthquakes; inelastic behavior of system and components; computation of inelastic response; correlation of experimentally observed and analytically predicted structural response; strength, stiffness and deformability of structural components; seismic design; and proportioning and detailing.

As mentioned, the major emphasis is on building-type structures. The publication is restricted to seismic structural response, analysis and design, largely because the consideration of inelasticity is not usual in design against wind.

BACKGROUND

When a structure responds elastically to ground motions during a severe earthquake, the maximum response accelerations may be several times the maximum ground acceleration, and depend on the structure's mass, stiffness, strength and energy dissipation capacity. It is generally uneconomical and also unnecessary to design a structure to respond in the elastic range to the maximum likely earthquake-induced inertia forces. Thus, the design seismic horizontal forces recommended by codes are generally much less than the elastic response inertia forces induced by a major earthquake. The development of seismic building codes has been traced from the structural engineer's point of view in Ref. 3. A world list of earthquake-resistant design regulations has been compiled by the International Association for Earthquake Engineering (4).

Experience has shown that structures designed to the level of seismic horizontal forces recommended by codes can survive major earthquake shaking. This is due to the ability of well-designed structures to dissipate seismic energy by inelastic deformations in critical regions of certain members. Decreases in structural stiffness caused by accumulating damage and soil-structure interaction *may* also help at times. It should be evident that use of the code-recommended seismic design loads implies that the critical regions of inelastically deforming members should have sufficient inelastic deformability to enable the structure to survive without collapse when subjected to several reversing cycles of loading well into the inelastic range. This means avoiding all forms of brittle failure and achieving adequate ductility by flexural yielding of members. To quote from Park (5):

In the design of multistory moment-resisting reinforced concrete frames to resist severe earthquakes, the emphasis should be on good structural concepts and detailing of reinforcement. Poor structural concepts can lead to major damage or collapse due to column sideway mechanisms or excessive twisting as a result of soft storeys or lack of structural symmetry or uniformity. Poor detailing of reinforcement can lead to brittle connections, inadequate anchorage of reinforcement, or insufficient transverse reinforcement to prevent shear failure, premature buckling of compressed bars or crushing of compressed concrete. In the seismic provisions of the New Zealand concrete design code, special considerations are given to the ratio of column flexural strength to beam flexural strength necessary to reduce the likelihood of plastic hinges forming simultaneously in the top and bottom of columns, the ratio of shear strength to flexural strength necessary to avoid shear failures in beams and columns at large inelastic deformations, and detailing of beams and columns for adequate flexural strength and ductility, and the detailing of beams, columns, and beam-column joints for adequate shear resistance and bar anchorage.

Most of the general principles enumerated above have broad applicability to reinforced concrete structures other than just the multistory moment-resisting frame. In connection with the good structural concepts mentioned above, the need for redundancy in structures designed to accommodate inelastic behavior cannot be overemphasized. Also, what is said about New Zealand seismic provisions is generally true of U.S. seismic provisions, although, differences exist between current United States and New Zealand code provisions for detailing

beams and columns for ductility and for the design of beam-column joints. Ref. 6 summarizes the principal differences in beam-column joint design provisions.

Over the past few decades, a pattern of American seismic building code development has emerged. Provisions have been proposed first by the Structural Engineers Association of California (SEAOC) in its "Recommended Lateral Force Requirements" (commonly refined to as the "Blue Book") (7), then adopted by the International Conference of Building Officials in the Uniform Building Code (8) and finally (often with modification) by the ANSI Standard (9) and the other model codes (10,11).

A departure from the above pattern occurred in 1972, when the National Science Foundation and the National Bureau of Standards initiated a Cooperative Program in Building Practices for Disaster Mitigation. Under that program, the Applied Technology Council (a research and development subsidiary of the Structural Engineers association of California) enlisted broad participation from all the diverse specialties related to seismology and earthquake engineering, and developed a document entitled "Tentative Provisions for the Development of Seismic Regulations for Buildings. This document, commonly referred to as ATC 3-06 (12), underwent thorough review by the building community in the ensuing years. Trial designs were conducted to establish the technical validity of the new provisions and to assess their impact. All of this subsequent effort culminated in the publication in 1985 of the "NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings," (13) which essentially is an extensively revised and updated version of the earlier ATC 3-06 document. A 1988 update of the NEHRP document has since been published, and a 1991 update is in progress.

In 1980, the SEAOC Seismology Committee undertook the task of developing an ATC-based revision of their Blue Book. This extensive effort resulted in the latest edition of the SEAOC Recommendations, which has been adopted into the 1988 and 1991 editions of the Uniform Building Code.

The last editions of the NEHRP document and the Blue Book contain the latest U.S. seismic design regulations. Significantly, the reinforced concrete design and detailing requirements of both documents are slightly modified versions of Chapter 21 of ACI 318-89 (14). This Chapter thus represents the current U.S. practice in seismic detailing of reinforced concrete structures.

Current design practice is based on the premise that a building should respond elastically to the factored wind forces specified for a given location. When subjected to an earthquake of the intensity specified for the given seismic map area, the structure is expected to experience inelastic response of some members.

Figure 1 shows the idealized force-deformation relationship of a structure, designed by current codes, and subjected to wind and earthquake forces. The two basic stages of structural performance are the elastic limit, stage C_2 , at which yielding starts in some structural members, and the limit of usable inelastic deformation, stage C_3 , beyond which collapse may occur due to member failure or instability.

The strength level corresponding to the factored wind loads, W_2 , must always be below stage C_2 , to assure elastic behavior under wind loads. A stage prior to C_2 can also be considered in design, such as stage C_1 , to correspond to the specified unfactored wind loads, and the related specified drift limit. Stages C_1 and C_2 are the serviceability (drift control) and strength limit states for wind design, as specified in current codes.

It should be noted that many structures, especially if tall and slender, are these days designed on the basis of wind tunnel testing of models. It may be desirable and sometimes necessary to allow some local yielding of some members of slender structures under design wind loads based on wind tunnel testing. This has been successfully done on a number of projects.

In design by current seismic codes, stage C_2 represents capacity (strength) equal to or exceeding that required under factored code-specified seismic forces, E_1 . It is assumed that due to the specified special detailing the structure has the capacity to deform inelastically to C_3 . To avoid collapse, the actual deformation corresponding to an earthquake of the intensity specified for the given seismic map area - (represented by E_2' in Fig. 1) must always be smaller than that at stage C_3 . E_2 and E_2'' refer to seismic force and deformation due to an earthquake of intensity as specified for the given seismic map area, but correspond to fully elastic, rather than inelastic, structural response. The earthquake force E_2 is obtained from the elastic (acceleration) response spectrum implicit in the applicable code. The code specified earthquake force E_1 is derived from the earthquake force E_2 by suitable reduction to account for inelastic deformations and damping.

The earthquake force E_1 is distributed along the height of a building considering only fundamental mode behavior for stiff

structures, and accounting also for some effects of higher modes for more flexible structures. For these distributed forces, an elastic analysis of the structure is carried out. Structural members are proportioned to resist the resulting internal forces at nominal strength reduced by a strength reduction factor. In some codes significant moment and shear redistribution is encouraged. The reduced seismic force level for which a structure is designed is chosen on the basis of the structural system's configuration and material. Detailing requirements are prescribed in an attempt to insure that resistance will be sustained at the deformations imposed on a structure by an earthquake of the actual intensity anticipated by the code. Certain deflection amplification factors or multipliers are also prescribed. The computed elastic story drifts corresponding to forces E_1 , when multiplied by the prescribed factors, are assumed to give the actual story drifts at E_2 , corresponding to the code-anticipated earthquake. These amplified computed story drifts are required to be within certain allowable limits to control damage and to avoid structural instability. Further, because it is desirable to have inelasticity occur in the beams rather than in the columns of reinforced concrete frames, it is usually required that, at a beam-column connection, the moment strengths of columns (calculated for the most critical axial design force consistent with the direction of the seismic forces considered) should exceed those of the beams.

DEFICIENCIES OF CURRENT PRACTICE

The current practice of earthquake-resistant design, as outlined above, has the following deficiencies:

1. The internal forces determined from elastic analysis under code-specified static loads, E_1 , are quite different from those resulting from an actual inelastic earthquake response of the structure.
2. The distribution and magnitude of inelastic deformations in individual structural members cannot be (and are not meant to be) determined through elastic analysis under code-specified static loads. Thus, special ductile detailing must be provided in many structural elements, even though inelasticity may actually occur only in certain key elements. Also, there is no way to ascertain that the inelastic deformation capacity provided through conformance with prescribed detailing requirements will always suffice.
3. Elastic story drifts under the code-specified forces, amplified by the prescribed multiplication factors, will be quite different from the actual inelastic story drifts. Thus, the intended damage control and safety against instability may not always be achieved.

4. Manipulating the relationship between the strengths of columns and beams at a beam-column joint on the basis of results from gravity load and static lateral load analyses may be ineffective at times. Actual inelastic seismic structural response can cause axial forces in columns that may substantially alter the strength relationship intended by the designer.

ELASTIC VERSUS INELASTIC STRUCTURAL RESPONSE: NUMERICAL EXAMPLE

Some of the above points can be better appreciated through an example that emphasizes that the response of building structures in actual intense earthquakes is inelastic, and that demonstrates through numerical analyses that such response can be realistically estimated only through inelastic dynamic analyses. With this view in mind, the results of dynamic elastic as well as inelastic response history analyses of hypothetical reinforced concrete buildings having different periods and subject to a particular history of ground accelerations, as recorded at a certain Mexico City location in the September 1985 earthquake, are presented and discussed in this section.

Elastic Response of Example Buildings

The basic structure selected for analysis is a six-story building, rectangular in shape, with five 13-ft (4 m) stories, a bottom story of 16 ft (4.9 m), and a penthouse. Floors consist of 9-in.-thick (230-mm) flat plates with 32-ft (9.75 m) square bays.

Four bracing schemes are investigated, as might be done during a preliminary design. Scheme 1 combines ductile or special moment frames in one direction with load-bearing shearwalls in the orthogonal direction. Schemes 2 through 4 also utilize ductile or special moment frame in the E-W direction. The lateral load-resisting system in the N-S direction consists of a Building Frame System (an essentially complete space frame provides support for gravity loads; resistance to lateral load is provided by shearwalls or braced frames) in Scheme 2, a Dual System* in Scheme 3, and ductile or special moment frames in Scheme 4. Figure 2 shows the framing system for Scheme 4. Details of the others are reported in Ref. 15.

* An essentially complete space frame provides support for gravity loads. Resistance to lateral load is provided by (a) A specially detailed moment resisting frame which is capable of resisting at least 25% of the base shear, (b) shearwalls or braced frames. The two systems are designed to resist lateral load in proportion to their relative rigidities.

Structural members comprising the four schemes were sized in conformance with the "Tentative Lateral Force Requirements" (1985 draft) issued by the Seismology Committee of the Structural Engineers Association of California for gravity loads and Uniform Building Code Zone 4 seismic forces. An importance factor I of 1.0 (standard occupancy structure) was assumed. Note that the ductile or special moment frames in the E-W direction are differently sized in Scheme 1, Scheme 2, and in Schemes 3 and 4 because of a specific SEAOC requirement (10) concerning the combination of different lateral load resisting systems in orthogonal directions of the same building.

The periods of all six natural modes of vibration were determined for each structural scheme in each principal direction by eigenvalue analyses of two-dimensional models described in Ref. 15. Similar periods were also determined from eigenvalue analyses of three-dimensional models described in Ref. 15, by constraining them to move along one or the other of the principal directions. The results are listed in Table 1. In general, two-dimensional and three-dimensional analyses yielded periods that were in reasonable agreement.

Dynamic elastic response history analyses were performed on the seven lateral load resisting systems of the four structural schemes described above (the systems in the E-W direction of Schemes 3 and 4 are identical) under the first 44 seconds of the S60E component of the SCT, Mexico City, 1985 ground motion shown in Fig. 3. The ground acceleration record of Fig. 3 has a number of distinct characteristics that are likely to influence structural response. First, the ground motion is very regular. In fact, the accelerogram is very unlike the highly irregular, almost erratic ground acceleration histories recorded in other earthquakes, and is not unlike the response of an elastic single degree-of-freedom system to a typical earthquake ground acceleration history. Second, the predominant period of the ground motion is an uncharacteristically long two seconds. Third, the peak ground acceleration is a relatively high 0.2g. According to Resenbueth (16), there is no record anywhere in the world with a horizontal peak ground acceleration of 0.20g associated with a two-second period. Fourth, the duration is unusually long, the motion lasted perceptibly over three minutes. Records show a very large number of significant cycles. Indeed, the ground motions experienced in Mexico City in the earthquake of 1985 were unique with respect to intensity, regularity, frequency, and duration, making the earthquake "selectively devastating." This uniqueness can be attributed to the well-known soil condition of the valley of Mexico (17).

The structural systems analyzed had fundamental periods of vibration ranging from 0.55 to 2.14 seconds, using values from Table 1 obtained from two-dimensional analysis. The analyses were carried out using the computer program DRAIN-2D (18), a

general purpose program for the dynamic analysis of plane elastic or inelastic structures. The dynamic response is determined using step-by-step integration, assuming a constant response acceleration during each time step.

Viscous damping in the form of a linear combination of mass-proportional and stiffness-proportional components was used in the dynamic analyses using DRAIN-2D. Five percent of critical damping in the fundamental and second modes was assumed. The three schemes with the longest periods were also analyzed assuming 10% and 20% of critical damping.

Figure 4 shows a plot of the computed top deflections against fundamental periods. As the fundamental period of the structure approaches, the predominant two-second period of the ground motion, the elastic response increases dramatically in magnitude. While higher damping reduces response, even at a very high damping of 20% of critical, the response of the buildings have fundamental periods close to two seconds shows drift in the range of 1.5%.

Inelastic Response of Example Buildings

Although the above investigation confirms resonance of ground motion and structure, and also demonstrates the role played by viscous damping in such resonance, it does not account for inelastic structural response. Most structures are, of course, designed to respond inelastically to moderate and major earthquakes.

There are two aspects of inelastic response that are of importance. First, the period of a reinforced concrete structure progressively lengthens as it suffers inelastic deformations in certain locations while responding to an earthquake. Second, inelastic hysteresis has an effect similar to damping on structural response to an earthquake.

Dynamic inelastic response history analyses were performed on the orthogonal lateral load resisting systems of the buildings considered under the first 44 seconds of the S60E component of the SCT, Mexico City, 1985 ground motion. The program DRAIN-2D was used for these analyses also.

Program DRAIN-2D accounts for inelastic effects by allowing the formation of concentrated "point hinges" at the ends of elements where the moments equal or exceed the specified yield moments. The moment versus end rotation characteristics of elements are defined in terms of a basic bilinear relationship which develops into a hysteretic loop with unloading and reloading stiffnesses decreasing in loading cycles subsequent to yielding. The modified Takeda Model (19), developed for reinforced concrete, was utilized in the program to represent the above characteristics.

The strength levels (yield moments) assigned to the ends of beams and columns were in accordance with factored bending moments at these locations caused by Zone 4 seismic forces calculated using Ref. 7 (1985 draft). The strength of each column stack was kept uniform at the value chosen for the base of the particular column stack. The strength of all the beams at a particular floor level were also made equal to the largest factored bending moment anywhere along that line of beams. The assigned positive and negative flexural capacities were equal at every critical section. Figure 5 shows a plot of the computed top deflections against fundamental periods.

The overall elastic and inelastic responses of the lateral load resisting systems of the building in Fig. 2 to the S60E component of the SCT, Mexico City, 1985 ground motion are presented in Table 2. The table lists the lateral deflections at various floor levels for two different amounts of viscous damping. It is evident from the table that yielding at certain locations decreases the response of the building considered to the SCT ground motion in both orthogonal directions. The response remains strong, however, even for 10% damping. During the inelastic response, the calculated roof level drifts in the N-S and E-W directions are 2.0% and 2.1%, respectively, for 5% damping. The corresponding values for 10% damping are 1.7% and 1.8%, respectively.

A comparison of Figs. 4 and 5 shows the effect of the lengthening of structural period due to inelasticity on seismic response. While in Fig. 4, the response is quite low for short-period structures, and suddenly shoots up as the critical two-second period is approached, Fig. 5 shows a much more gradual increase in structural response starting at much shorter periods and continuing up to the two-second period and beyond. This is because shorter period structures, with accumulating damage and with progressive increases in period caused by such damage, gradually "ride into" the critical period range of the ground motion. The observed behavior of Mexico City structures in the 1985 earthquake was more in accord with Fig. 5 than with Fig. 4, as is only to be expected. Buildings in the 5- to 21-story height range (having initial fundamental periods of vibration between 0.5 and 2.1 seconds) were vulnerable, rather than just buildings in the 20-story height range (having initial fundamental periods of around 2 seconds).

CONCLUSION

This chapter emphasizes that the response of building structures in actual intense earthquakes is inelastic, and that such response can be realistically estimated only through inelastic dynamic analysis. Much of the discussion is focused on the controlled utilization of inelastic response in earthquake-resistant design of concrete structures. The purpose and scope of this ACI Special Publication are outlined.

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Table 1 - Periods from eigenvalue analysis, seconds

East West								
	Scheme 1		Scheme 2		Scheme 3		Scheme 4	
	3-D	2-D	3-D	2-D	3-D	2-D	3-D	2-D
T1	1.30	1.42	1.41	1.68	1.64	2.14	1.94	2.14
T2	0.40	0.43	0.43	0.54	0.35	0.68	0.58	0.68
T3	0.19	0.23	0.21	0.30	0.14	0.37	0.30	0.37
T4	0.11	0.14	0.13	0.20	0.08	2.24	0.19	0.24
T5	0.08	0.10	0.08	0.15	0.05	0.17	0.13	0.17
T6	0.06	0.07	0.06	0.12	0.04	0.13	0.10	0.13
North-South								
T1	0.59	0.59	0.57	0.64	0.60	0.55	1.75	2.07
T2	0.11	0.10	0.11	0.11	0.13	0.12	0.54	0.66
T3	0.05	0.04	0.05	0.04	0.06	0.05	0.29	0.37
T4	0.03	0.02	0.03	0.02	0.04	0.02	0.19	0.26
T5	0.02	0.01	0.03	0.01	0.03	0.01	0.14	0.21
T6	0.02	0.01	0.02	0.01	0.03	0.01	0.11	0.17

Table 2 - Comparative elastic and inelastic displacements of concrete building subject to the S60E component of the SCT, Mexico City, 1985 ground motion.

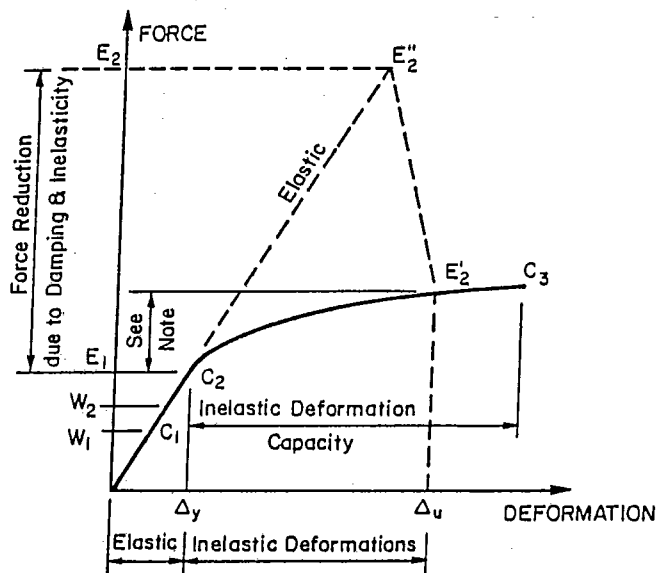
(a) Elastic Displacement, inches

Scheme	4 N-S		4 E-W	
	5% of Cr.	10% of Cr.	5% of Cr.	10% of Cr.
Level				
1	6.10	4.19	6.87	4.11
2	13.71	9.41	15.67	9.33
3	21.28	14.57	25.18	14.95
4	27.59	18.86	33.30	19.73
5	32.19	21.97	39.26	23.23
6	34.95	23.84	42.83	25.33

(b) Inelastic Displacement, inches

Scheme	4 N-S		4 E-W	
	5% of Cr.	10% of Cr.	5% of Cr.	10% of Cr.
Level				
1	4.23	4.19	4.04	3.41
2	8.42	7.43	8.08	6.88
3	12.44	10.95	12.14	10.39
4	15.20	13.67	15.60	13.36
5	17.72	15.61	18.26	15.68
6	19.24	16.91	20.16	17.31

Note: Fundamental periods of Schemes 4N-S and 4E-W are 2.07 and 2.14 seconds, respectively.



Loads:

- W_1 = design wind forces, unfactored
- W_2 = design wind forces, factored
- E_1 = design earthquake forces from code, factored
- E_2 = design earthquake forces from response spectrum

Response to Loads:

- E'_2 = represents inelastic response to an earthquake of code anticipated intensity
- E''_2 = represents elastic response to an earthquake of code anticipated intensity
- E'_2 and E''_2 are related by the equal displacement or the equal energy concept.

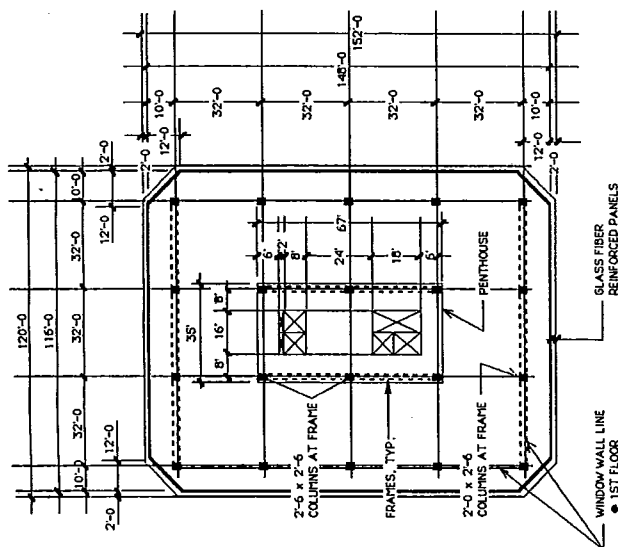
Stages:

- C_1 corresponds to specified wind drift limit
- C_2 corresponds to initiation of yielding
- C_3 corresponds to deformation capacity

Note:

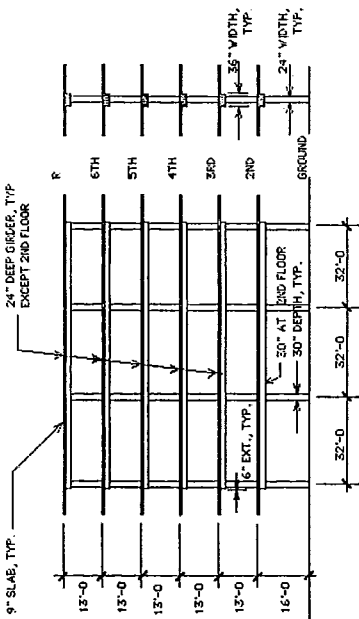
- Strength increase caused by
 - (1) strength of section mobilized during large displacements being possibly larger than nominal strength, and
 - (2) gradual formation of hinge mechanism with moment redistribution.

Fig. 1--Idealized force-deformation relationship of code-designed structure subjected to wind and earthquake forces

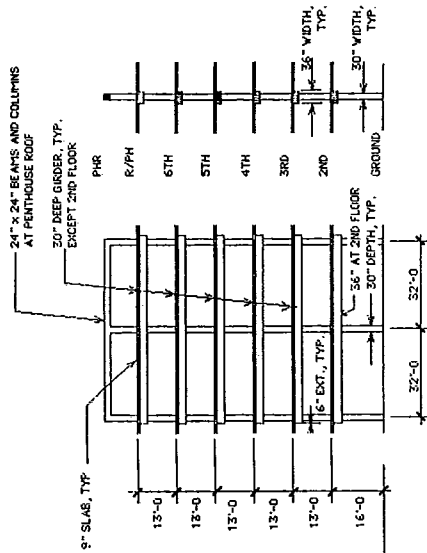


a) Plan

Fig. 2--Example building



b) E-W frame member sizes



c) N-S frame member sizes

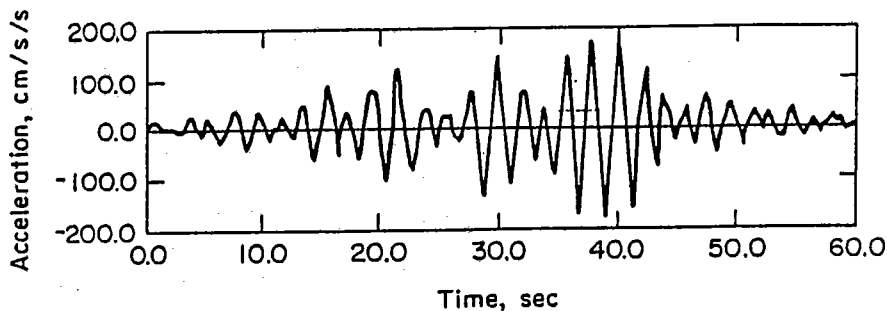


Fig. 3--S60E component of ground accelerations determined from N and E components measured at SCT in Mexico City

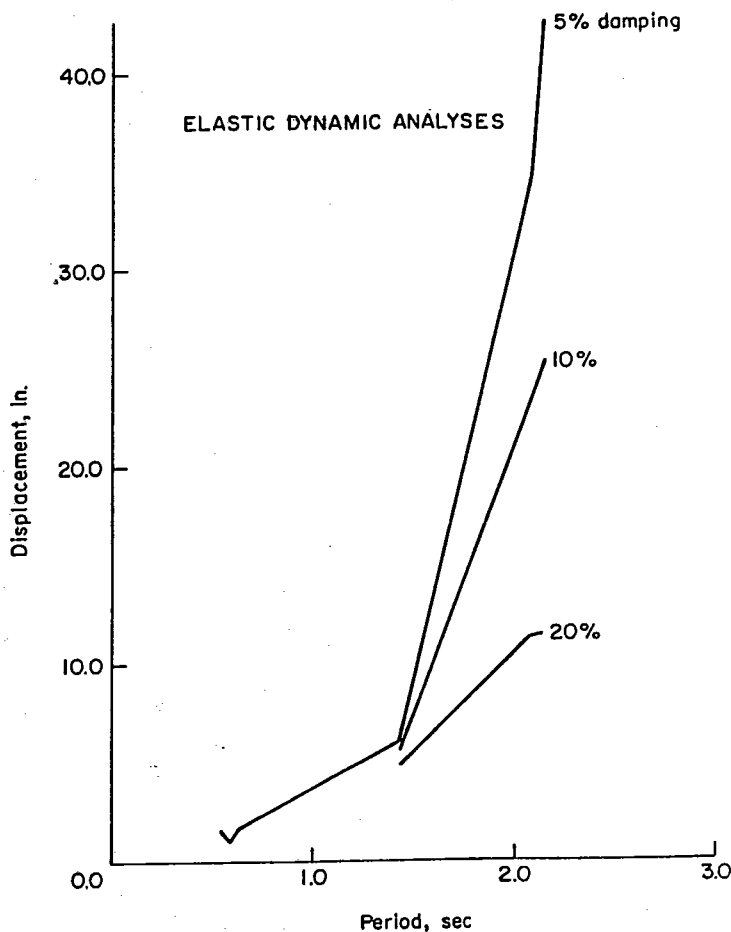


Fig. 4--Calculated elastic displacements of multi-degree-of-freedom systems subjected to ground motion of Fig. 3

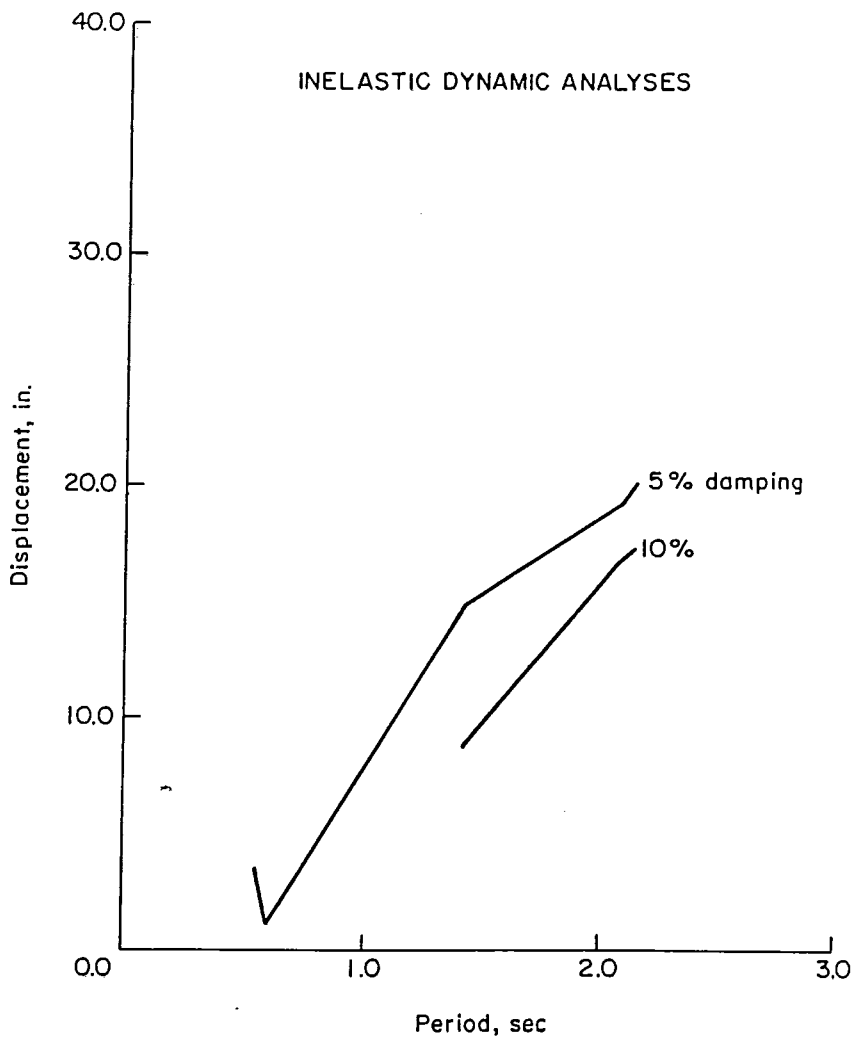


Fig. 5--Calculated inelastic displacements of multi-degree-of-freedom systems subjected to ground motion of Fig. 3

Earthquake Ground Motions

by S.D. Werner

Synopsis: This chapter summarizes fundamental concepts pertaining to the development of seismic input criteria for inelastic design of reinforced concrete structures. The chapter is organized into five main sections that describe concepts related to earthquake phenomena; ground shaking characterization, geologic factors, and measurement; and the three principal approaches for defining seismic design forces or input motions--seismic regionalization maps, design spectra, and motion time histories. Compilation of the information contained in this chapter was completed in early 1988; therefore, the concepts presented in the chapter are based on earthquake ground motion data, results, and procedures developed up to that time.

Keywords: Earthquake-resistant structures; earthquakes; elastic properties; geology; structural design.

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EARTHQUAKE PHENOMENA

This section provides fundamental information pertaining to the physical processes of earthquakes, and how earthquake sizes, locations, and intensities are characterized.

Physical Processes

Physical processes that characterize earthquakes include the global patterns of earthquake occurrence (plate tectonics), the fault rupture process, and the seismic waves that occur as a result of fault rupture.

Plate Tectonics - The zones of rock rupture during earthquakes are termed faults. On a global scale, there is a major pattern of faults, which are boundaries between large regions of the earth's surface termed plates. According to the theory of plate tectonics, these plates are moving relative to one another, and this movement process has led to the occurrence of most of the world's earthquakes along the plate boundaries (Fig. 1)(1). These boundaries can be classed as spreading zones (mid-ocean ridges where plates are moving apart), shear zones (where plates are sliding past one another - e.g., the San Andreas fault in California), or subduction zones (where plates collide, forming the world's great mountain ranges as well as island arcs such as Japan). Approximately 90% of the world's release of seismic energy for shallow earthquakes occur within subduction zones.

Although most of the world's earthquakes occur along plate boundaries, a limited number of significant earthquakes have occurred within plates and cannot be explained by the theory of plate tectonics. Such earthquakes arise from more localized systems of tectonic forces, perhaps due to variations in temperature, depth, and strength of the surface rocks (1). Examples of significant intraplate earthquakes are those at New Madrid, Missouri (1811-12) and Charleston, South Carolina (1885-86).

Fault Rupture Process - Earthquakes are caused by the gradual shifting of the earth's crust, which results in a slow buildup of strain energy in the rock on either side of a fault. When this energy exceeds the strain energy capacity of the rock material along the fault surface, the fault ruptures, resulting in a release of the stored strain energy. This rupture process starts at a point along the fault plane termed the focus of the earthquake, and then spreads from that point in all directions along the fault plane (Fig. 2a). The fault rupture is often irregular and jerky, rather than uniform, because of the variation in rock properties along the fault surface and the redistribution of elastic forces as the rupture process advances. The extent of the rupture depends upon the variation of stored strain energy within the rock throughout the region. The rupture continues until the strain energy remaining in the rock no longer exceeds the rock's strain energy capacity.

The direction of the fault rupture process is used to characterize various fault types. Three general types of faults - strike-slip, reverse, and normal - are illustrated in Fig. 2b. Pure examples of these fault types rarely occur; rather, the displacement of the fault has components both parallel and normal to the fault trace. In addition, the fault slip typically occurs along an irregular surface rather than a plane, and there may also be branch and secondary faulting (2).

Seismic Waves - When rupture occurs along a fault plane, the strain energy stored in the rock is released in the form of seismic waves and heat. These seismic waves propagate away from the causative fault and through the geologic layers, and this wave propagation leads to earthquake ground shaking.

The two classes of seismic waves are body waves - which propagate through the interior of the earth - and surface waves - which propagate along or near the earth's surface (Fig. 3a). The principal types of body waves are P-waves (which compress and dilate the rock) and S-waves (which shear the rock sideways at right angles to the direction of wave propagation). The most significant surface wave types are Rayleigh waves (which cause elliptic retrograde motion) and Love waves (which cause horizontal motion normal to the direction of wave propagation in layered rock media). Of the two types of body waves, P-waves travel faster than S-waves. Surface waves travel more slowly than body waves and, of the two surface wave types, Love waves travel faster than Rayleigh waves. Thus, as these various types of waves radiate away from the causative fault, they separate from one another in a predictable manner (Fig. 3b).

As seismic waves propagate from the source, they are affected by the nature of the geologic medium through which they pass. Body waves are reflected and refracted as they

encounter nonhomogeneities and layer interfaces in the underlying rock medium (Fig. 3c), and surface waves are similarly affected by topographic irregularities and material property variations along their near-surface propagation path. The ground shaking at the site of a given structure results from the superimposed effect of all of these waves, which have a range of wavelengths and directions of incidence as they propagate through the site's underlying geologic materials.

Earthquake Size

There are several measures of the size of an earthquake. Of these, the earthquake magnitude is most commonly used for engineering purposes. The magnitude is calculated from measurements on seismographs and is therefore expressed using ordinary numbers and decimals. Physically, the magnitude has been correlated with the energy released by an earthquake (Fig. 4)(3). There are several different magnitude scales currently in existence as described in Table 1 (4).

Location

The location of a potential earthquake, in terms of the distance from its source to the site under investigation, is of major importance in characterizing ground motions for seismic design purposes. Several alternative measures of this source-site distance have been used, the most common of which are the hypocentral (or focal) distance (i.e., the distance from the site to the point within the earth's crust where initial fault slippage occurs) and the epicentral distance (i.e., the distance from the site to the vertical projection of the hypocenter onto the earth's surface). For smaller earthquakes with shallow hypocentral depths, these distances will be roughly similar; however for larger earthquakes, the distances may differ considerably.

The source-site distance measures are used in engineering representations of the attenuation of ground shaking with distance due to geologic spreading effects and the dissipative properties of geologic materials through which seismic waves propagate. It is well known that long period (low frequency) signals attenuate with distance at a slower rate than do short period (high frequency) excitations. For this reason, ground response spectra may experience changes in shape as the source-site distance varies (5).

Earthquake Intensity

The observed effects of an earthquake on people and on structures and their contents are represented through the use of qualitative, subjective intensity scales. The assignment of intensities is influenced by such factors as the quality of construction, human reaction to ground shaking, structural design practices in the affected region, and the population density at the time of the earthquake. Nevertheless, such scales are valuable in areas where no strong motion records exist, and particularly where they are the only means of interpreting historical information. Several intensity scales have been developed; in the United States, the Modified Mercalli (MM) scale is the most widely used (Table 2).

Procedures for assigning intensities following an earthquake are well documented (6). The isoseismal maps that are developed from these intensities (e.g., Fig. 5) reflect the attenuation of damage with distance from the earthquake source, and the extent of the area in which the earthquake is felt (7). The shapes and extent of the isoseismals in such maps (i.e., the curves that separate regions of successive intensity ratings) may be representative of the tectonic features of the area and, in addition, may be indicative of predominant seismic wave propagation directions and the manner in which the earthquake originates.

Intensity scales (particularly MM Intensity) have been used in recent years for categorizing the seismic vulnerability of buildings according to their height, structural type and materials, age, and/or design requirements (8,9). In addition, statistical regression analysis procedures have been widely used to correlate intensity with the strength, frequency content, and duration of ground shaking, in order to define seismic input criteria for the design of structures in regions where strong motion data have been nonexistent (10,11,12). However, because intensities are qualitative and are influenced by factors other than ground shaking characteristics, such correlations provide at best only a very approximate basis for estimating ground shaking parameters and should be used for this purpose only where no other information is available.

GROUND SHAKING

This section provides background information on earthquake ground motions, including the characterization of strong ground motions for engineering design purposes, the geological parameters that affect ground shaking, and the current strong motion accelerograph network.

Characterization of Ground Motion

The effects of earthquake ground motions on the behavior and integrity of structures are dependent on the strength, frequency content, and duration of the ground shaking. Procedures and parameters for characterizing these quantities are summarized below.

Strength - In current engineering practice, the strength of the ground shaking is most typically represented by a single-parameter quantitative measure. The most widely used measure of this type is the peak ground acceleration; however, this parameter is representative of maximum seismic forces applied to high frequency structural systems only, and not to more typical structures whose natural frequencies fall in the low-to-intermediate range. In fact, it has been shown that the acceleration peaks of many ground motion records can be clipped markedly, without significantly affecting the response of most structures to the ground shaking represented by these records.

Several single-parameter alternatives to the peak acceleration have been suggested over the years, including the peak velocity, peak displacement, spectrum intensity, root-mean-square (rms) acceleration, and Arias intensity (Fig. 6). The strength of ground shaking has also been represented using parameters that characterize the response of structures to the ground shaking (4). One such parameter is the effective peak acceleration which is qualitatively described by Newmark and Hall (13) as being "associated with the significant part of the ground motion containing repetitive motion portions that possess strong energy content and produce significant linear and nonlinear deformation (of the structure)." In view of this, the effective peak acceleration may differ from and be less than the peak free-field ground acceleration. The effective peak acceleration has been widely used in recent years, and is a basis for defining seismic input motion strength levels in the Applied Technology Council (ATC)/National Earthquake Hazards Reduction Program (NEHRP) design guidelines for buildings (14,15). The use of effective peak acceleration in these design guidelines is summarized in a subsequent section of this chapter.

The various strength measures described above may result in different (and possibly misleading) assessments and rankings of the relative strengths of ground motion records. This is because the measures vary widely in their ability to represent the strength of the shaking over the particular frequency range where the energy of a given record is concentrated or where the primary structural response will occur (4).

Frequency Content - To define the frequency content of the ground shaking, a ground response spectrum or Fourier amplitude spectrum is required. Of these, the ground response spectrum is the most widely used, since it describes ground motion frequency characteristics in a form that is directly applicable to structural design and analysis. The ground response spectrum is obtained by applying the ground motions to the base of a single-degree-of-freedom oscillator with a given damping ratio, and plotting its maximum response as a function of its natural frequency or natural period. This peak response can be represented in several different ways, and can be plotted in either linear or tripartite form (Fig. 7) (16).

Duration - In addition to the strength and frequency content, the duration of the ground shaking will also influence the seismic response of structures. This is particularly true for ductile structures designed to yield when subjected to strong ground motions; the inelastic response of such structures is sensitive to the number of cycles of strong motion that will be applied during the earthquake.

At present, there is no single universally accepted approach for numerically defining the duration of strong shaking of a given ground motion accelerogram. Several such approaches have been suggested, some of which are summarized in Table 3; however the differing duration definitions employed in these approaches may lead to conflicting trends regarding effects of such parameters as magnitude, distance, site conditions, etc. (5,12,17-20).

Significant Geologic Factors

As discussed earlier, earthquakes are caused by fault rupture which leads to seismic waves that propagate through the underlying and near-surface geologic media. As these seismic waves propagate in the vicinity of the ground surface, they induce motions of the near-surface soil materials that, in turn, cause shaking of structures. Thus, the nature of the ground motions at the site of a given structure is dependent on the characteristics of the earthquake source mechanism, the source-site transmission path, and the local geologic conditions at the site (Fig. 8) (21).

The features of the source mechanism that influence ground shaking are the stress drop, the total fault displacement, the length of the fault break, the nature of the rupture process (i.e., the single or multiple fault breaks that occur), the fault shape, and the proximity of the fault plane to the ground surface. The resulting seismic waves that propagate from the source to the site are, in turn, influenced by numerous parameters related to the geometry and material properties of

the transmission path. As the waves reach the vicinity of the site, they must be transmitted through the near-site geologic media to reach the structure itself. The resulting ground motions at the site of the structure are therefore also dependent on the local geologic conditions at the site, including the properties and geometry of the subsurface soil layers, the surface topography, and the depth and properties of the underlying bedrock.

Strong Motion Measurements

The first strong motion accelerographs were deployed in the early 1930's, and ground motion measurements were obtained soon thereafter during the 1933 Long Beach, California, earthquake. Since then, the deployment of strong motion instruments has expanded tremendously in the United States and throughout the world. The technology associated with strong motion instrumentation and data processing has grown at a corresponding rate, and the resulting strong motion data base is an invaluable source of information for understanding effects of earthquakes on structures and for further improvement of seismic design procedures.

Strong motion data are available from several sources. Digital and analog copies of the more significant strong motion records in the United States are available through the National Geophysical and Solar Terrestrial Data Center in Boulder, Colorado. In addition, these data can be transmitted to individual computer terminals via telephone hookup through the University of Southern California (22). Other agencies that disseminate processed strong motion data include California Division of Mines and Geology (data obtained from their accelerograph network) and the World Data Center A for Solid Earth Geophysics, which can provide earthquake data from around the world.

SEISMIC REGIONALIZATION MAPS

General Discussion

A cornerstone of the seismic input provisions in design codes has been seismic regionalization maps that define intensity factors for various regions of the United States, based on their general seismologic and geologic characteristics. The intensity factor defined for each region is used to account for the seismicity of the region when computing equivalent lateral forces for seismic design

purposes. The provisions of the current Uniform Building Code (UBC) along these lines are shown in Fig. 9 (23).

Although many seismic regionalization maps have been developed through the years, the maps included in the design provisions recommended by the Applied Technology Council (ATC-3) (14) and later adopted in the National Earthquake Hazards Reduction Program (NEHRP) provisions (15) have a probabilistic basis. The basic features of these particular maps are briefly summarized below. For a more extensive discussion of the development and implementation of the maps, the reader is referred to the commentary in ATC-3 (14) and the discussions by Algermissen (24) and by Berg (25).

Parameters

Two seismic regionalization maps provided in ATC-3 (and the NEHRP seismic design provisions) are shown in Fig. 10; one corresponds to "effective peak acceleration (EPA)," and the other to "effective peak velocity-related acceleration (EPV)." Neither of these parameters has a precise physical definition. In ATC-3, the EPA is considered to represent the structural response over a period range from 0.1 sec to 0.5 sec, whereas the EPV represents the structural response at about a 1 sec period (Fig. 11). The EPA, expressed in units of g , is used to scale the intensity of the ATC-3 spectrum shape to obtain a design spectrum. The EPV, expressed as a velocity-related acceleration in g 's, is used (1) to adjust the spectrum shape to account for extended distance effects, and (2) to represent the strength of shaking in the computation of lateral design forces. The ATC-3 spectrum shapes are discussed later in this chapter.

Map Development

The ATC-3 map for EPA utilized the work of Algermissen and Perkins (26) to develop EPA contours that represent a 10-percent probability of exceedance over a 50-year time period for firm site conditions throughout the United States (Fig. 10a). It is based primarily on the approximately 200 years of historical seismicity records within the United States, incorporating geologic evidence to supplement this seismicity data. The EPV map was developed by using EPV vs distance attenuation data by McGuire (27) as well as modified Mercalli intensity data, in order to modify the EPA maps to incorporate extended distance effects (Fig. 10b).

DESIGN SPECTRA

General Description

A design spectrum is a means of specifying the level of force to be applied to a structure as a function of its natural frequency. Its shape is typically developed as a smoothed composite of an appropriate ensemble of ground response spectra, and its amplitudes must be associated with (1) the anticipated characteristics of the ground shaking at the site; and (2) the specified level of structural resistance (e.g., allowable stresses, ductility ratio, etc). Of the various alternative forms of seismic input criteria described in this chapter, design spectra are most commonly used in current practice for design of structures where dynamic analysis is required.

The use of design spectra in the seismic analysis of a structure requires computation of the natural period, mode shape, and modal participation factor for each significant normal mode of vibration of the structure, as well as specification of the damping ratio for each mode. Then, standard response spectrum analysis procedures are used to compute the structure's maximum response when it is vibrating in each of its normal modes (28). These modal responses are combined using approximate procedures (29), in order to compute the structural response quantities necessary to assess the ability of the structure to withstand the anticipated ground shaking.

The above dynamic analysis procedure is strictly valid only for use with linear elastic models of the structure. However, for structures that exhibit elasto-plastic resistance characteristics and are designed for small excursions into the inelastic range, the design spectrum can be adjusted to correspond to the design ductility ratio (13).

There are two main approaches currently used for developing design spectra -- deterministic and probabilistic. These two approaches, together with ground motion attenuation relationships which are a cornerstone of each approach, are described in the remainder of this section.

Ground Motion Attenuation Relationships

Ground motion attenuation relationships are derived empirically through the use of statistical regression analyses that relate ground motion parameters to magnitude, distance, and, in some instances, general classes of site conditions.

The various relationships that have been developed generally lead to similar estimates of ground motion parameters for those ranges of earthquake magnitude and source-site distance where ample strong motion data exist. However, for magnitudes or distances where strong motion data are scarce, the various ground motion attenuation curves often differ substantially, because of differences in the strong motion data base or the regression analysis procedures used by the developers of the various curves. This is illustrated in Figure 12, in which Donovan (30) compared several peak acceleration attenuation curves for a Magnitude 6.5 earthquake (31-36). These curves compare well at distances beyond about 25 km, where strong motion data are ample. However, at shorter distances where only limited data still exist, the various curves differ markedly. Average peak acceleration attenuation curves developed from the attenuation relationships given in Ref. 31-36 are shown in Fig. 13 (30).

Most of the ground motion attenuation curves have been derived for peak acceleration, since this parameter is the most widely used measure of the strength of the ground shaking (despite its shortcomings as discussed earlier in this chapter). Less common are empirical relationships for peak velocity or displacement or for the duration of strong shaking. An excellent summary of ground motion attenuation relationships developed from 1974 to 1984 is provided by Campbell (37).

It has previously been noted that most of the strong motion records in the United States have been measured in the west, and primarily in California. Therefore, most of the ground motion attenuation curves are strictly valid only for these regions. However, some attenuation relationships are available for other regions of the United States; for example, relationships developed for the central United States by Nuttli and Herrmann (38) are based on limited recorded data, empirical correlations between peak ground acceleration and Modified Mercalli Intensity, and theoretical seismic wave attenuation relationships developed for the central United States. These relationships exhibit a much more gradual rate of attenuation with increasing distance from the source than do relationships based on strong motion data from California (Fig. 14) (39,40).

Deterministic Procedures for Developing Design Spectra

The first step in the use of deterministic methods to develop design spectra involves the identification of specific earthquake events associated with particular faults or other geologic features, together with the sizes and source-site distances associated with these events. Standard ground motion vs. distance attenuation curves as described above are then used to establish the general levels of shaking at the s

(usually in terms of peak ground acceleration). These ground shaking levels are used to develop design spectra either: (a) by scaling of standardized spectrum shapes; or (b) by providing a basis for developing site-dependent spectra. These two forms of the design spectra are described in the following paragraphs.

Standardized Spectrum Shapes - Standardized spectrum shapes have been developed from statistical analyses of spectra from an ensemble of accelerograms that represent either: (a) a wide variety of geologic and site conditions (site-independent); or (b) broad classes of local site conditions. These spectrum shapes are typically normalized to a unit value of peak acceleration, a , and more recently developed shapes also use specified velocity parameters, v/a , and displacement parameters, ad/v^2 (where v and d are the peak velocity and peak displacement respectively) (13).

Site-independent spectrum shapes were first developed by Housner(41) and more recently by Newmark and his associates (13, 42-44). The most recent of these spectrum shapes are given in Figure 15 (13). Examples of spectrum shapes that represent broad classes of local soil conditions are those developed by Seed and Idriss (40), which form the basis for the spectrum shapes provided in the ATC-3, and NEHRP, and 1988 UBC seismic design provisions (Fig. 16) (14,15,23).

Site-Dependent Spectra - As described by Guzman and Jennings (45), site dependent spectra are developed from an ensemble of strong motion records whose conditions of magnitude, distance, local site geology, tectonic province, and fault mechanism are comparable to those of the site and the postulated design earthquake. Accelerograms are added to the ensemble until, according to Guzman and Jennings, "the conditions under which they were recorded are no longer representative of the design earthquake, or until records included in the ensemble are so clearly more representative that there is little confidence that additional records are meaningful to the selection of the design spectra." Once this ensemble is selected, empirical ground motion attenuation curves are used to adjust the amplitudes of the selected accelerograms to correspond more closely to the postulated design earthquake. Spectra from the accelerograms scaled in this manner are then plotted on a single figure and used as a basis for defining an "appropriately conservative" ground response spectrum representative of the design earthquake and the geologic conditions at the site.

Numerical procedures involving analytical models of the propagation of seismic waves through the adjacent or underlying geologic media can serve to guide the development of site-dependent design spectra at sites with unique topographic features or underlying geologic layers that may not be well represented by the current strong motion data base (46-48).

However, existing analytical methods can provide only a first approximation of the complex nature of the seismic wave generation and propagation process during an earthquake. This should preclude the use of extensive numerical procedures and analytical models to tailor a design spectrum to fit particular geologic conditions; furthermore, as suggested by Housner and Jennings (4), modifications to design spectra to incorporate numerical results from analytical models should be simple and relatively minor in nature.

Probabilistic Methods for Developing Design spectra

Overview - The seismic design or upgrading of a structure requires an assessment of the potential level of shaking at the site of a structure due to future earthquakes. However, the location, magnitude, and other characteristics of future earthquakes are unknown and generally random. Probabilistic methods, termed seismic hazard analyses (SHAs), incorporate laws of probability to account for these uncertainties during the development of seismic design spectra. In this way, probabilistic methods differ from previously described deterministic methods which do not directly account for the uncertainties associated with the occurrence and size of future earthquakes.

SHAs define the seismic hazard for a site in terms of probabilities that various levels of ground motion will be exceeded during the design life of the structure. These results are developed from information that describes the seismicity, geometry, and location of each potential earthquake source, and from empirical ground motion vs. distance attenuation curves for a range of earthquake magnitudes. Analytical procedures are then used to synthesize this regional information and to derive the probabilities of exceedance associated with various levels of ground shaking at the site.

Basis for Development of SHAs. The development of SHAs is typically based on the following assumptions (49-51):

- Seismic sources are modeled as either known active faults or seismically active point sources or area zones. In these various source models, the earthquakes are assumed to originate as straight-line ruptures extending on known or active faults.
- For a given earthquake, the amplitude of the ground motion at the site is dependent on the earthquake's magnitude and the distance between the site and the ruptured segment of the fault zone. Simple random attenuation laws have been developed to represent the distribution of possible ground motion strength levels for a given earthquake magnitude and source-site distance (50).

- The probability distribution of earthquake magnitudes is based on the magnitude-recurrence law developed by Richter (6).
- Earthquakes are assumed to occur with equal probability at any location within a region and at any time. This corresponds to the modeling of the occurrence of earthquakes within a region as a homogeneous Poisson process with a constant mean occurrence rate.

Form of Results - The above process for incorporating SHAs in the development of design spectra is illustrated in Figs. 17 and 18. These figures demonstrate the two forms of design spectra resulting from SHAs. In the first approach (Fig. 17), the SHA is used to develop probabilities associated with various peak ground acceleration (PGA) levels. The PGA that corresponds to an acceptable probability of exceedance is then used to scale a standardized spectrum shape in order to develop the design spectrum for the site.

The above approach, although widely used, is deficient in that the probability of exceedance is defined only for the PGA, which is not a good measure of the effects of ground shaking on the response of most structures. The corresponding probability levels at the frequencies of interest for structural response are not known and may vary substantially over the range of these frequencies.

In view of this, recent developments in SHA methodologies have focused on the definition of constant hazard spectra - that is, spectra whose amplitudes correspond to the same probability of exceedance over the entire frequency range of interest. The development of constant hazard spectra (Fig. 18) requires the performance of a separate SHA at each discrete frequency, and the development of spectral amplitude vs. probability curves for each frequency. These curves are then each entered at the same probability level, and the resulting spectral amplitude points are then plotted vs. frequency to obtain the constant hazard spectrum. Figure 19 provides an example of constant hazard spectra developed for a rock site (52).

MOTION TIME HISTORIES

Application in the Design Process

A third approach for specifying seismic input criteria for structural design purposes is through the direct use of an ensemble of accelerograms (i.e. acceleration time histories). These accelerograms should be consistent with the design spectra developed for the site, and should represent the anticipated shaking at the site due to all of the significant potential earthquake sources in the vicinity of the site.

Accelerograms specified in this way can be used as input motions for a computerized time-history analysis of the dynamic response of a structure. Time history analysis procedures use numerical integration of the coupled equations of motion of a mathematical model of the structure, in order to compute the model's response at successive time steps throughout the duration of the input motions. Such analysis methods are more time consuming and expensive than are the response spectrum analysis procedures described earlier. However, time history methods are necessary when the response of the structure is anticipated to be nonlinear, which would invalidate the use of normal mode methods of analysis.

In view of this, motion time histories are the most relevant of the various available forms of seismic input motion for the inelastic dynamic analysis of reinforced concrete structures according to procedures described in subsequent chapters of this publication. The development of motion time histories for such analyses, as well as for general seismic design applications, has been based primarily on measured and processed accelerograms contained in the current strong motion data base. In addition, various types of synthetic accelerograms have been used for certain applications.

Accelerograms from Strong Motion Data Base - A key source of motion-time histories for seismic design purposes is the strong motion data base, which contains an extensive array of recorded and processed accelerograms. The various stations at which these accelerograms were recorded represent a wide variety of geologic, tectonic, and subsurface soil conditions - all of which can influence the characteristics of the recorded motions. Differences in these conditions at the instrument locations for the various accelerograms shown in Fig. 20 (all plotted to the same amplitude and time scale) are undoubtedly the source of the marked differences in the features of these recorded motions. Therefore, the engineer must be aware of the significance of these conditions, and should judiciously select an ensemble of accelerograms that collectively best represent the particular conditions at his structure's site. In addition, such accelerograms may be adjusted, where appropriate, such that; (1) the composite spectra from the accelerograms are reasonably consistent with the design spectra developed for the structure; and (2) the composite characteristics of the accelerograms are reasonably consistent with those indicated by applicable ground motion attenuation relationships.

With this as background, the following paragraphs summarize some of the basic features of the current strong motion data base. The earthquake magnitudes corresponding to the current data base range from about 2.8 to about 7.6. Most of the data are from earthquakes whose magnitudes are about 6.5, such as San Fernando (1971) and Imperial Valley (1979). Very few

strong motion records have been obtained during earthquakes with magnitudes greater than 7.0.

The distances from the earthquake epicenters to the instrument locations for the strong motion records most typically range from about 20 km to about 200 km. Only within the last 10 years have strong motion data begun to be recorded at arrays of instrument locations very near the causative fault (e.g., from the Imperial Valley (1979), Coyote Lake (1979), and Mammoth Lakes (1980) events); however, records from near-source locations are still relatively sparse.

Most of the strong motion accelerograms from the United States have been recorded in California, with important but fewer accelerograms recorded in the northwest (i.e. Washington, Montana). Strong motion accelerograms from the eastern and midwestern United States are scarce, although this limited data base has been enhanced in recent years by accelerograms recorded during moderate earthquakes in North Carolina (1978), New Hampshire (1982), New Brunswick, Canada (1982), and Indiana (1987).

Synthetic Accelerograms - The development of synthetic accelerograms was first motivated by the need to at least partially fill important gaps in the current strong motion data base. Toward this end, various methods based on random vibration theory or on analytical wave propagation models were used to develop synthetic accelerograms that embody the basic characteristics of strong motion records, as indicated by available data and by engineering judgment regarding future earthquakes (53,54).

A second class of synthetic accelerograms has also been developed, for cases where regulatory agencies have specified the use of accelerograms whose response spectrum at a particular damping ratio matches a given smoothed design spectrum (55,56). However, because this is not a valid physical basis for developing synthetic accelerograms, the characteristics of the time signature of such accelerograms are often atypical of actual recordings of strong earthquake motion. As a result, this class of synthetic accelerograms may not be appropriate for use in analyzing nonlinear structural response, which will be strongly dependent on the signature of the input motions. Therefore, care and judgment should be exercised when identifying suitable spectrum-compatible accelerograms for use in nonlinear structural analyses, and when interpreting the results of such analyses.

CONCLUDING COMMENTS

This chapter is intended to provide a basic framework for defining seismic input motions for use in the dynamic analysis

of inelastic reinforced concrete structures. Details of the various methods have of necessity been omitted; the reader is referred to the appropriate references cited in this chapter for further description of the concepts and procedures that have been presented.

It is particularly important to note that, of the various forms of the seismic input criteria described, ground motion time histories are the only form that is of direct use for inelastic dynamic response analysis. Within this context, design spectra should be regarded as a possible basis for assessing the appropriateness of a given ensemble of ground motion time histories or for scaling them to represent an appropriate overall strength level. Similarly, seismic regionalization maps have been described only for purposes of completeness, since they are embodied in the principal seismic design codes used or proposed in the United States. However, such broad regionalization maps are of little direct use in defining ground motion time histories for inelastic response analysis. The development of motion time histories should instead be based on more detailed site-specific assessments.

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