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

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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 WIND AND SEISMIC DESIGN PRACTICE

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 structured 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 thorough 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.

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* 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%.

<u>Inelastic Response of Example Buildings</u>

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.