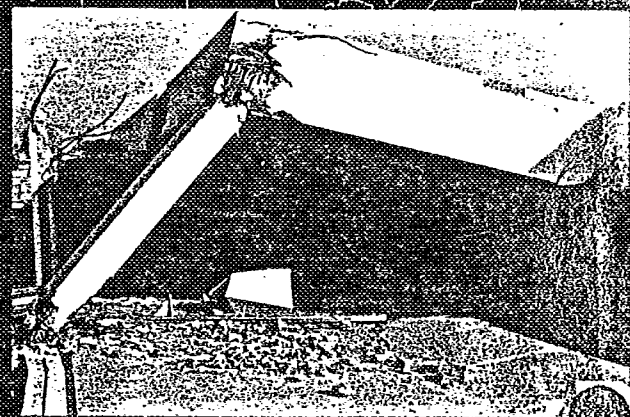


# Reinforced Concrete Structures In Seismic Zones



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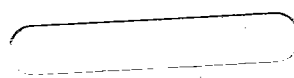
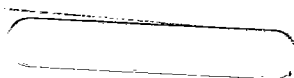
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# Reinforced Concrete Structures In Seismic Zones

*Cover: World Seismicity Map, compiled by Dr. Arthur C. Tarr and published by the U.S. Geological Survey.*



Publication SP-53  
American Concrete Institute, Detroit



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DISCUSSION of individual papers in this symposium may be submitted in accordance with general requirements of the ACI Publication Policy to ACI headquarters at the address given below. Closing date for submission of discussion is January 1, 1978. All discussion approved by the Technical Activities Committee will be published in the August 1978 ACI JOURNAL together with appropriate closing remarks by the authors.

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## PREFACE

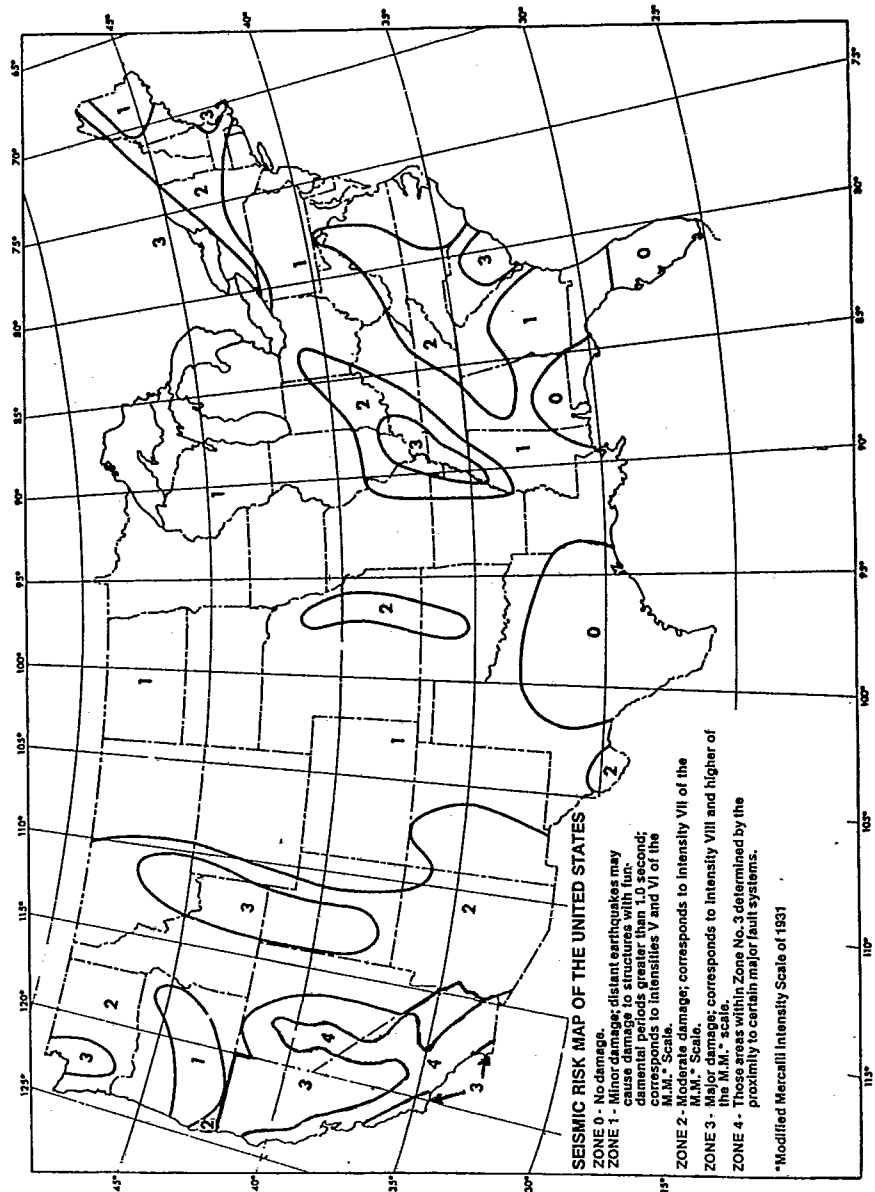
This volume is an outgrowth of a Symposium on Reinforced Concrete Structures in Seismic Zones, held at the 1974 ACI Annual Convention in San Francisco and sponsored by the ACI Technical Activities Committee, ACI Committees 318, 348, 352, 426, 441, 442 and 443, and the Structural Engineers Association of Northern California. The steering committee for the Symposium was chaired by James O. Jirsa, who also represented Committee 352 - Joints. Other members of the steering committee and the ACI Committees they represented were: John E. Breen, ACI-TAC; John A. Sbarounis, ACI 318 - Building Code; Emilio Rosenblueth, ACI 348 - Structural Safety; Paul F. Fratessa, ACI 426 - Shear and Diagonal Tension; Loring Wyllie, Jr., ACI 441 - Reinforced Concrete Columns; Mark Fintel, ACI 442 - Response of Buildings to Lateral Forces; and W. Gene Corley, ACI 443 - Bridge Design. Alex C. Scordelis represented the Structural Engineers Association of Northern California.

Seventeen papers were presented at the Symposium which was organized around three sessions, Reinforced Concrete Structural Systems in Seismic Zones, Shear Problems in Seismic Zones and Design Problems. The symposium was introduced with short presentations by H. S. Kellam, the President of the Structural Engineers Association of Northern California and by John L. McLaughlin, the Chairman of ACI-TAC. Following the Symposium, the speakers and others, whose papers could not be presented because of time limitations, were invited to submit articles for this volume. That invitation generated sixteen of the papers in this volume. To that number were added two papers, one by Gulkan and Sozen and the other by Fintel, published previously in the ACI Journal and having considerable significance for understanding the behavior of concrete structures in seismic zones. Finally, as editors we added an introduction intended to place the earthquake problem in a historical perspective and the contents of this volume in a contemporary perspective.

Thanks are due to the Symposium steering committee for initiating this effort and to all the authors, the reviewers of the manuscripts and the ACI staff for their efforts in the preparation of this volume.

Neil M. Hawkins, editor

Denis Mitchell, associate editor



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## Introduction

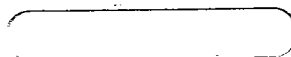
### Historical Perspective

by Neil M. Hawkins and Denis Mitchell

The growth of building code provisions for seismic design and the structural engineering profession's interest in seismic phenomena are closely linked to experience with the damaging effects of major earthquakes. In North America, collective experience has been small. Only recently has the majority of the structural engineering profession begun to recognize that earthquake loading effects differ markedly from other live loading effects.

There is a major need for researchers to share as rapidly as possible with the designer their emergent understanding of the earthquake problem. Earthquake damage increases with increasing urbanization. North America has only recently become a continent of urban dwellers. In the known major earthquake zones, it has only been over the past 100 years or less that urban growth has made the earthquake problem a major source of potential disaster. One hundred years is a short time compared to the known return periods for major earthquakes.

Earthquakes of magnitude seven or greater have been reported near the junction of the St. Lawrence and Saguenay rivers in Canada at about 70 year intervals since 1663. The 1663 Earthquake was felt generally over eastern North America and caused a religious revival in the New England States (1). Three great earthquakes of approximately equal strength occurred near New Madrid, Missouri in 1811 - 12 (2). The earthquakes were felt from Canada in the north to New Orleans in the south and Washington D. C. in the east. Five foot fissures opened in the earth, waves on the Mississippi River swamped boats and yet only one life was lost because the epicentral area was lightly inhabited. The largest known earthquake in the Pacific Northwest (3) occurred in 1872 and was probably centered somewhere near the junction of the Cascade range and the Canadian border. Damage was light. The total population of Washington State was less than 30,000. However, the earthquake caused a landslide near Wenatchee that dammed the Columbia river for 16 hours. The river ran dry and residents walked across its bed.



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The good old-days of little damage due to major quakes in North America have since departed and the subsequent sequence of events can be viewed as building towards an inevitable crescendo which apparently has yet to arrive. The new era started when the first major quake in an urban area occurred in Charleston, South Carolina, in 1886 (2). Twenty-seven people were killed outright and 83 more died from related effects. From this and subsequent East Coast shocks it has been established that for quakes of equal magnitude, zones of equal intensity extend over a much greater area on the east than on the west of the continent. That behavior is attributed to geological differences affecting attenuation of the seismic ground waves.

The second major U. S. shock in an urban area occurred in San Francisco in 1906 (4). About 700 people out of a population of 400,000 died in the earthquake and the subsequent three day conflagration of the damaged area. In spite of the intensity of the quake, the San Francisco experience had no immediate effect on building practice. The city was rebuilt to a design code that treated wind and earthquake alike as a horizontal pressure of 30 psf. Further in subsequent publicity, the earthquake effects were played down and the consequences of the fire emphasized. Fire rather than earthquake was more a psychologically acceptable rational explanation for the disaster. The 1925 Santa Barbara earthquake was not a large one (2). However, it was centered close to the city, causing considerable damage and killing 14 people. That earthquake ushered in the current era of formal earthquake engineering studies. It prompted the U. S. Coast and Geodetic Survey to initiate a study that developed the first strong motion seismograph. Further, because of that quake, seismic design provisions were introduced into the newly created Uniform Building Code (5). Those provisions were in the form of an optional appendix with earthquake forces calculated as a function of mass.

For U. S. cities the destructive effects of the 1933 Long Beach earthquake (6) rank third to those of the 1906 San Francisco and 1964 Alaskan shocks. About 100 people were killed and property damage exceeded 40 to 50 million dollars. More importantly this quake settled forever the then existent debate about the need for special building provisions for earthquake resistant design. Because many public

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school buildings were damaged, the California legislature, in session at the time of the quake, passed the Field Act. That act required all new public school construction in California to be earthquake resistant and subject to careful supervision during construction. Masonry buildings without frames had to be designed for a lateral force equal to 10 percent of the dead load plus a portion of the live load. Other structural forms had to be designed for lateral loads ranging from two to five percent of the vertical design load. The percentage decreased as the allowable foundation load increased. An amendment to the Field Act in 1937 made the coefficients also a function of the number of stories in the building. For all other buildings, except certain dwellings and farm structures, the legislature passed the "Riley Act" which required design for lateral forces ranging from two to three percent. Both Acts required lateral bracing in all structures and reinforcement in masonry structures. The City of Los Angeles passed an even more conservative ordinance requiring a lateral design force equal to eight percent of the dead load plus half the occupancy load.

In 1935 the Uniform Building Code, (UBC), requirements were changed to provisions modeled after the Field and Riley Acts. Those code provisions remained essentially unchanged through 1952. As late as 1946 the earthquake provisions were still in an Appendix headed "Lateral Bracing." Design was required for a horizontal force,  $F$ , computed from the formula

$$F = CW \quad \text{--(1)}$$

where  $C$  was a coefficient ranging from 0.02 for the whole structure to 0.25 for parts.  $C$  also varied with foundation conditions.  $W$  equaled the total dead load plus one half the design vertical live load. The 1946 code made  $C$  also a function of the Seismic Zone with values doubled for Zone 2 and quadrupled for Zone 3.

The 1935 Helena, Montana, earthquake surprised many residents because it occurred in an area not previously considered seismically active (7). There were three earthquakes separated by a period of 19 days. Six people were killed and property damaged exceeded three million dollars. The unreinforced brick masonry construction of the area was badly damaged and the cumulative effects of the successive quakes readily apparent.

In the 1940 Imperial Valley earthquake (8) there was marked surface faulting. Nine people were killed and property damage exceeded five million dollars. An accelerograph located in the city of El Centro provided the first good recording of the characteristics of a strong earthquake close to its epicentral region. That record is a classic widely used for dynamic response studies. This earthquake clearly demonstrated the benefits of the Field and Riley Acts. Fifteen "Field Act" schools in the area survived without damage as did several privately owned buildings designed to the Riley Act. In contrast, non "Field Act" schools suffered considerable damage. As a result of the Imperial Valley earthquake, the coefficient  $C$  in Eq. (1) in the Los Angeles Building Code was made a function of the flexibility of the structure.

The late 1940's was a period of major seismic activity in the Pa-

## Hawkins and Mitchell

cific Northwest. In 1946 an earthquake occurred in the Straits of Georgia about 100 miles north of Vancouver, Canada. It did considerable damage to man-made structures in the region. The 1949 Olympia, Washington, earthquake (9) was similar to the 1935 Helena earthquake in that it occurred in a region generally not considered seismically active by the residents. There was considerable damage to older unreinforced masonry buildings and to school buildings. Eight people were killed and property damage exceeded 15 million dollars. This quake, the subsequent 1965 Seattle quake and other less major quakes in the area have been characterized by deep focal depths. Consequently although there has been no surface faulting, strong motion effects and damage have occurred over wide areas. In this respect the Pacific Northwest quakes have had characteristics closer to Japanese than Californian quakes (10). This quake moved the Puget Sound region from its Zone 1 status in the 1946 UBC Code to Zone 3 status for the 1952 Code.

The 1952 Kern County, Southern California, earthquakes (11) were the first affecting a significant number of structures engineered to the requirements of the Field and Riley Acts, the UBC Code and the City of Los Angeles ordinances. Twelve people were killed in the major quake, surface faulting extended over 14 miles and property damage exceeded 48 million dollars. Damage effects from the main quake and after shocks were unmistakably cumulative. The quake again demonstrated the high susceptibility of unreinforced masonry and brick to severe damage. Schools designed in accordance with the Field Act performed excellently while those constructed prior to the act were generally badly damaged. This quake clearly demonstrated the need for good quality control and construction practices for reinforced concrete structures. It was also the first major quake affecting precast concrete construction. Where carefully detailed and constructed that material performed well. However, several failures occurred where joint details were poor.

In 1949 the Earthquake Engineering Research Institute (EERI) was formed, as an outgrowth of an Advisory Committee on Engineering Seismology appointed by the U. S. Coast and Geodetic Survey. The principal objective of the EERI was to promote research towards the development of improved earthquake resistance in buildings. Toward that end, it organized a Conference on Earthquake and Blast Effects on Structures at UCLA in June 1952; it was expected that wartime experience with blast effects on structures might be applicable to earthquake design problems. That conference's success resulted in EERI organizing the "First" World Conference on Earthquake Engineering, at Berkeley in June 1956. That Conference led in turn to the organization of the International Association for Earthquake Engineering which has sponsored World Conferences at four year intervals since 1956. The Proceedings of those conferences (12) have helped engineers and scientists from around the world pool their knowledge and advance the art of earthquake resistant design and construction. Those Conferences have also heightened interest in earthquake research within countries and regions. Canada, Europe, Japan, New Zealand and the U.S.A. now schedule conferences at mid-term intervals between the world conferences. The first Canadian conference was held in 1971 and the first U. S. Conference in

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1975. The Proceedings (13) of those Conferences contain much information relevant to the design of concrete structures in seismic zones. The Earthquake Engineering Research Institute has done much since 1956 through its conferences and newsletters to support research and disseminate information on earthquakes.

The 1959 Hebgen Lake, Montana, earthquake (14) and the 1964 Alaska earthquake (15) both highlighted damage likely due to earth movements. In the Montana quake 28 people were killed by landslides. However, damage to masonry buildings was slight apparently because vertical rather than horizontal motions dominated. By contrast, the 1964 Alaskan earthquake had major structural implications with landsliding and liquefaction contributing markedly to that damage. Faulting extended over 400 to 500 miles. Although only 15 people were killed by landslide or falling debris, 110 perished in the seismic sea wave (tsunami) caused by the quake. Property damage was officially estimated at 311 million dollars. This quake emphasized the importance of the duration of major shaking. Seismic data indicated an intensity exceeding two percent of gravity for more than three minutes. Eyewitnesses frequently reported that collapse occurred in the later stages of shaking. Thus, although the maximum horizontal acceleration was less than 16 percent of gravity major damage occurred to many relatively new buildings designed to be earthquake resistant. For reinforced concrete structures major damage included shear failures of deep spandrel beams connecting shear walls, failures of shear walls, complete collapse of a prestressed concrete lift slab structure, numerous failures of reinforced concrete joints and precast joints, severe slippage and pounding along construction joints and failures of reinforced masonry walls due to poor grouting of the reinforcement.

California's rapid increase in population and urbanization in the 1940's made the need for comprehensive earthquake design provisions increasingly obvious. Toward that end a Joint Committee of the San Francisco Section, ASCE, and the Structural Engineers Association of Northern California published in 1952 a classic report (16) that was the first proposing an explicit relationship between the seismic coefficient and the fundamental vibration period of the building, and the first to provide for distribution of lateral forces to floors in proportion to their height. The Structural Engineers Association of California, (SEAOC) then formed a statewide committee to develop a seismic code based mainly on the Joint Committee's concepts. That SEAOC committee's task was finished in 1959 (17) and the lateral forces specified by that code have remained the main basis for design up through 1974. In that SEAOC Code the base shear for the structure was computed by the formula

$$V = KCW \quad \text{--(2)}$$

The C factor was specified as

$$C = \frac{0.05}{\sqrt{T}} \leq 0.10 \quad \text{--(3)}$$

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K was a horizontal force factor ranging from 0.67 to 1.33. K depended on the type and arrangement of resisting elements and was intended to reflect the historical experience of certain building systems performing better than others. W equalled the total dead load plus 25 percent of the floor live load for storage and warehouse occupancies. The code also introduced specific requirements and limitations for portions of buildings and for the use of different construction elements.

In 1961 UBC officials incorporated the SEAOC provisions into their code. They modified Eq. (2) to the form

$$V = ZKCW \quad \text{--(4)}$$

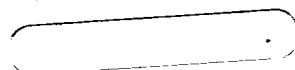
where Z was a seismic zoning factor equal to 0.25, 0.5 and 1.0 for seismic zones 1, 2 and 3 respectively.

SEAOC and UBC officials recognized that if a structure behaved elastically, the maximum lateral forces induced in or generated in it in a major earthquake would exceed those implied by Eqs. (2) and (4). By using Eqs. (2) and (4) they presumed that the structure would respond inelastically and that considerable energy would have to be dissipated by yielding in preferred localities within the structure. Thus, building components had to be detailed so that they had sufficient ductility under reversed cyclic loading to provide that energy dissipation. SEAOC envisaged that structures designed to their provisions should be able to:

1. Resist minor earthquakes without damage;
2. Resist moderate earthquakes without structural damage, but with some non-structural damage;
- and 3. Resist major earthquakes as intense as the strongest experienced in California, without collapse, but with some structural and non-structural damage.

1961 also saw publication of the text "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions" (18). That classic examined critically the interrelationship between earthquake ground motions and the ductility demands for such a structure designed by the equivalent static force concept. That text demonstrated the practical use of the design spectrum concept developed earlier by Cal Tech researchers (19), examined the various factors affecting the limiting ductilities of concrete members and presented a complete example for the design of a 24 story concrete structure. That text and the 1959 SEAOC Code were the inspirational sources for a large part of the current flood of analytical and experimental research on the earthquake-resistant characteristics of reinforced concrete structures.

From 1959 through 1974, changes in the SEAOC provisions have been aimed at ensuring building components with the degree of ductility required by the lateral force design procedures. In particular, in 1966 SEAOC introduced recommendations for ductile concrete frames over 160 feet in height and for the use of concrete shear walls or braced frames. The recommendations for ductile moment-resistant space frames



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drew strongly on results obtained in the experimental investigation of beam-column joints undertaken by the Portland Cement Association (20).

The 1967 Caracas, Venezuela, earthquake killed 266 people and severely damaged many multistoried reinforced concrete structures. Earthquake resistant design provisions comparable to those of the 1943 Los Angeles Code had been incorporated into Venezuelan standards since 1955 and many of the concrete structures had been designed to standards comparable to ACI 318-56 (21). This earthquake highlighted the effects of non-structural components, particularly in-fill walls, on response; the need to allow for loading reversals and the fact that actual earthquake forces can be considerably larger than the design forces; the need for adequate column ties; the need for joints as strong as the members themselves; the likelihood of punching failures at concrete slab to column connections and the need to allow for diaphragm stresses in the design of connections between slabs and walls or frames. Observations of severe damage to tied concrete columns in the Caracas earthquake resulted in the 1968 SEAOC and the 1970 UBC revisions including recommendations for minimum column ties greater than those specified in ACI 318-63. In order to classify a concrete frame as ductile-moment-resisting, confinement of the concrete columns was made mandatory along with additional considerations for shear in connections and detailing of connections.

The late 1960's saw the development of two major earthquake simulator facilities in the U.S.A. The University of Illinois instillation has a 10,000 lb capacity, 12 ft. square table with one horizontal degree-of-freedom. The University of California at Berkeley's instillation is a central feature of their Earthquake Engineering Research Center. Their simulator has a 20 ft. square table, a vertical and one horizontal degree-of-freedom, and a 100,000 lb capacity. Since 1967 both Universities have conducted a variety of shake-table tests on concrete structures and published many reports dealing with earthquake related problems.

The damage experienced by modern structures in the Skopje (22), Caracas, and particularly the Alaskan earthquake forced prominent American engineers to carefully reassess earthquake resistant design concepts. As a result in 1969 the National Academy of Engineering published a report entitled Earthquake Engineering Research (23). That report addressed the practical problems posed by earthquakes and discussed the research needed to solve those problems. It briefly described the nature of the earthquake problem, the present state of knowledge in the field, where knowledge was lacking and where further research was needed. In response to that report the U. S. Federal Government initiated in 1970 through the National Science Foundation a Research Applied to National Needs Program on Earthquake Engineering. The overall objectives of that program, NSF-RANN, are the development of economically feasible design and construction methods for building earthquake resistant structures of all types, the development of procedures for integrating information on seismic risk with ongoing land use planning processes, the development of improved understanding of social and economic consequences of individual and community decisions

on earthquake related issues and the presentation of program results in forms usable by the affected interested communities to control their vulnerability to earthquakes. Funding for that program has risen from 0.7 million dollars in 1970 to six to eight million dollars per year since 1974.

In 1970 and 1971 two texts were published that brought the structural engineering profession up to date on the state-of-the art of earthquake resistant design. The 1970 text (24) was a collection of articles by prominent experts and included features on earthquake causes, dynamic analyses, response spectrum, dynamic testing and the formulation of mathematical models, reviews of earthquake damage, and the design of earthquake resistant reinforced and prestressed concrete structures. The 1971 text (25) presented a unified treatment of earthquake engineering and structural response and the decisions and information necessary for optimization of earthquake resistant design.

The 1971 ACI Code 318-71 was published with an Appendix A "Special Provisions for Seismic Design." Those provisions have served as a model for subsequent clauses dealing with earthquake resistant proportioning of concrete members in other codes. The intent in the Appendix was to minimize seismic forces by requiring a ductile energy-absorbing structural system whose components had strengths developed through the formation of plastic hinges rather than less ductile flexural, shear or compression failure modes. The provisions of Appendix A were based largely on the Portland Cement Associations beam-column results (20) and earthquake damage effects observed in Skopje (22), Anchorage (15), and Caracas (21). That Appendix developed guidelines for the design and detailing of joints, confining concrete columns and the proportioning of ductile moment resistant concrete frames.

The 1971 San Fernando, California earthquake (26) caused major damage to several different types of reinforced concrete structures. The recently completed Psychiatric Unit at the Olive View Medical Center was designed as a ductile moment resistant frame according to the 1960 SEAOC Lateral Force Requirements. Its response verified subsequent changes to that Code covering the confinement and shear reinforcement necessary for columns. The first floor columns with insufficient, improperly spaced tie reinforcement according to ACI 318-71 Appendix A requirements suffered shear and compression failures. Spirally reinforced columns in the Main Building of the Center exhibited large ductilities but their connections to the beams behaved poorly because of poor confinement for the joint regions and insufficient anchorage for the spiral reinforcement. Collapses of parts of the Veteran's Administration Hospital indicated clearly the differences between poorer, older buildings and the better response of newer buildings designed to more up to date codes. Numerous other reinforced concrete failures occurred including spandrel beam failures, column shear failures, slab shear failures, precast connection failures and reinforced masonry wall collapses. In all the performance of concrete buildings designed to ACI 318-63 and the 1960 and subsequent SEAOC lateral force requirements could only be described as "fair".

In addition to the lessons on building design the San Fernando

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experience also provided valuable information on concrete material effects and likely damage to concrete bridges. Many lightweight aggregate concrete members performed poorly as compared to similarly designed dense aggregate concrete members. Because of that result there is an on-going careful reassessment of the strength and ductility of lightweight aggregate concrete members. Epoxy injection procedures were proven to be an effective means of repairing moderately damaged concrete members. In laboratory tests repaired members have shown strength and stiffness characteristics comparable to those of undamaged members. There were numerous examples of collapses of recently completed reinforced and prestressed concrete bridges. The causes included loss of support due to excessive horizontal movements, pier collapse due to insufficient confining ties for the vertical reinforcement, girder-pier connection failures, splice failures, expansion joint failures, abutment and wing wall failures. Those failures have initiated an on-going study of earthquake resistant design requirements for bridges.

Partly in response to the San Fernando earthquake and to Public Law 91-606 concerning Disaster Preparedness Studies, the National Bureau of Standards, with National Science Foundation funding, organized a workshop on Building Practices for Disaster Mitigation in 1972. The report on that workshop (27) contains many fine review articles that evaluate current building practices, define opportunities for improving practice from documented research findings and recommend the research necessary to fill gaps in existing knowledge. Particular emphasis is placed on procedures for implementing improved practices at both professional and policy levels. As a consequence of that workshop, NSF-RANN has sponsored several major studies on Disaster Mitigation Planning. The major projects likely to affect reinforced concrete practice have been the Applied Technology Council's study of the response spectrum approach (28) and their efforts to develop a comprehensive earthquake design code (29).

In December 1972, a devastating earthquake hit Managua, Nicaragua, killing about 8,000 people out of a city of 450,000, injuring 20,000 and causing property damage in excess of a billion dollars (30). That quake represented another test of concrete structures designed to modern building codes. Most engineered buildings survived the quake, though some suffered major damage. Again it was demonstrated, as in San Fernando, that the state-of-the art for reinforced concrete structures was only fair. Buildings with structural shear wall systems generally performed better than buildings with moment-resistant frame systems. The large deformations of the frame structures resulted in unacceptable amounts of both structural and non-structural damage. Many upper floors in concrete buildings collapsed as a result of abrupt changes in floor plans at upper levels. Shear wall and upper floor collapses clearly indicated a need for better understanding of the response in such cases. The considerable amount of non-structural damage caused by failures of attachments for ceilings, light fittings, interior walls, etc. also demonstrated the need for more information on requirements for anchorage to concrete in seismic zones.

**Documents**

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In recent years ACI Technical Committees have recognized the importance of the earthquake problem and its impact on concrete design practice. The 1971 Committee 442 report (31) reviewed and evaluated various structural systems with particular reference to their function in resisting lateral loads. A subsequent symposium volume (32) added significantly to that understanding. A recent report by Committee 352 (33) on the proportioning and detailing of beam-column joints includes particular recommendations for joints undergoing load-reversals.

In 1974 SEAOC issued a revised "Lateral Force Requirements" based on experience and the correlation of actual seismic and design data for the response of 60 buildings in the Los Angeles area to the San Fernando earthquake. Eq. (4) was revised to the formula

$$V = ZIKCSW \quad \text{--(5)}$$

where I was an importance factor equal to 1.5 for essential facilities. C was a coefficient determined from the formula

$$C = \frac{1}{15\sqrt{T}} \not\geq 0.12 \quad \text{--(6)}$$

T was the period of the structure established using the structural properties and deformation characteristics of the resisting elements and S was a coefficient for site-structure resonance. S was to be not less than unity and the product CS did not need to exceed 0.14. The Code also introduced a requirement that drift should not exceed 0.005 times the story height unless it could be demonstrated that greater drifts were tolerable. Displacement calculated from the application of the required lateral forces were to be multiplied by 1.0/K to obtain the drift.

1975 saw publication of a text (40) that provided an up-to-date comprehensive source of information on the likely behavior of concrete structures in seismic zones.

#### THIS PUBLICATION AND THE STRUCTURAL DESIGN PROBLEM

The design of reinforced concrete structures for seismic zones is an emerging field in which account must be constantly taken of new lessons to be learned from recent earthquakes and from the utilization of recent documented research results. Problems associated with the design of reinforced concrete structures to resist earthquakes can be classified as shown in Fig. 1. Structures can be divided into two broad classes, new and existing. In the former case conceptual problems dominate while in the latter retrofitting considerations control.

New Structures. As implied by the 1974 SEAOC and 1975 Building Code of Canada provisions (35), probably the first step in designing a new structure is classification of the importance of the structure. Although all structures designed for seismic zones need careful attention there are many structures demanding special consideration. Exam-



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ples are those structures posing large loss of life hazards to the community: Nuclear power plants, high rise structures, dams, convention halls, schools, etc; and those structures necessary for post disaster services such as hospitals, civil defense centers, utilities (particularly water), etc.

Once the classification of the structure has been recognized, the design process is likely to diverge into two parts: analysis to determine the loadings and analysis to proportion the members. The analysis to determine the loadings requires examination of the local seismicity. Ideally, the designer should become knowledgeable on the seismic history of the area, the geology of the area, the damage to structures in the area in previous earthquakes and the soil properties of the area in general and the site in particular. Such a procedure permits microzonation of the area and determination of design criteria on a local basis rather than the broad basis mandated by present building codes. The importance of ground motions effects and local site conditions have been highlighted by a recent comparative study of the damage to the Olive View Medical Center in the San Fernando earthquake and the Charaima and Banco de America buildings in the Managua earthquake (36).

For completeness of this introduction, the zonations recommended for Canada by the Associate Committee of the National Building Code (35), for the U.S.A. by the International Conference of Building Officials in the 1976 UBC Code (5), and for Mexico by the Ministry of Public Works (41), are shown in Figs. 2 through 5 respectively.

Conceptualizing the structure and its likely earthquake behavior is the most vital step in the design process. The choice of the framing system, ductile-moment resistant shear walls, a combination of frames and shear walls, framed tube, tube-in-tube etc. (31) is very important. The designer must also consider the effect of the chosen structural system on the likelihood of damage to non-structural elements. The recent Managua and San Fernando experiences raise many questions concerning the stiffness necessary for lateral load resisting elements. The papers in this volume by Sharpe on the "Earthquake Problem" and by Engelkirk on "Seismic Design of Reinforced Concrete Buildings from a Designers Viewpoint" provide further background on conceptual problems and their relationship to current building code provisions.

The designer can choose either a highly sophisticated approach or a simpler code approach in order to determine the member forces for proportioning the structure. A sophisticated approach may include use of one or more of the following features: a complete time history analysis for particular one, two or three dimensional ground motions; inelastic modeling of structural elements in two or three dimensions; and the consideration of several different seismic inputs or a random excitation analysis. A response spectrum or dynamic analysis is desirable in most cases. The paper by Clough provides further data on dynamic analysis, as well as the correlation between such an analysis, using different mathematical models for section behavior, and the response in an earthquake simulator test of a two-story reinforced con-



crete frame. The mathematical modeling of section behavior is also discussed in the papers by Yamada and Kawamura, and by Gulkan and Sozen. Both papers examine how to arrive at a reasonable mathematical model for predicting the response of concrete structures. In the Yamada and Kawamura paper it is proposed that the aseismic capacity of a reinforced concrete structure can be taken as the steady state resonance capacity for an idealized one mass model of that structure subjected to a sinusoidal forced vibration. The paper by Gulkan and Sozen proposes that the design base shear can be estimated by interpreting the maximum inelastic response in terms of a linearly elastic analysis of a fictitious linear structure whose stiffness and damping characteristics are determined as a function of the assumed or known maximum displacement.

While the concepts discussed by Clough, Yamada and Kawamura, and Gulkan and Sozen can be extrapolated to shear walls, the discussion in those papers is relevant primarily to frame structures. The papers by Fintel and Paulay address basic design concepts for shear wall structures. Fintel's paper outlines clearly the damage control benefits that can be obtained with properly designed shear walls while Paulay's paper addresses in particular the detailing problems associated with shear walls if those walls are to behave in a ductile manner.

The bulk of the papers in this publication deal with the response of various reinforced concrete components to reversed cyclic loading. The papers discussed previously have been concerned primarily with input concepts for the design equation, i.e. lateral force, stiffness and ductility requirements. The remaining papers deal primarily with output concepts, i.e. the sizing of members and the detailing of the reinforcement. To apply the results of such investigations, an understanding is needed of the importance of hysteretic behavior and the factors to be considered in evaluating that behavior. Such considerations are addressed in References 37, 38 and 39.

There are eight papers dealing primarily with component behavior. The paper by Barda, Hanson and Corley reports results of tests on low rise shear walls with boundary elements and suggests design procedures for such walls. The paper by Laible, White and Gergely reports results of cyclic interface shear transfer loadings of large precracked concrete blocks. The results in the other six papers address problems associated primarily with frames. The paper by Ernst and Smith compares the response of reinforced concrete frames to displacements simulating known earthquake records, with the response to load-seeking and displacement-seeking lateral force effects. The paper by Popov and Bertero reports the results of tests on interior beam-column sub-assemblages and from an evaluation of those results makes recommendations for improving aseismic code provisions. The paper by Uzumeri reports the results of tests on corner and isolated beam-column joints examining the effects of the amounts, size and mechanical properties of the reinforcement in the joint. The paper by Townsend and Hanson also reports the results of tests on isolated beam-column joints for which the axial loading on the column and the load history were variables. A third paper on beam-column joints by Henager examines the

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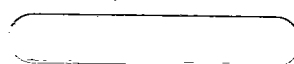
benefits of using steel fibrous reinforcement rather than hoops in the joints of isolated beam-column connections. The last component behavior paper by Hassan and Hawkins reports results of tests to determine the bond characteristics of bars subjected to high intensity cyclic loadings.

While the results of properly planned component tests can often be used directly to improve aseismic design provisions, the range of variables is often so wide or possible interaction effects so complex that the implications of component tests can be properly evaluated only through the development of mathematical models to predict the responses measured in tests. The second paper by Hassan and Hawkins proposes such a mathematical model for interpretation of the significance of their bond tests results. They use that model to deduce that the bond characteristics for the beam bars dictated the response of the beam-column specimens tested by Hanson and Conner (20). The paper by Selna and Lawder uses special purpose computer programs to study the importance of biaxial coupling and overturning effects on frame behavior.

As indicated in Fig. 1 a final consideration would be the characteristics of the materials used in the structure. No papers in this publication deal directly with materials although that by Henager is relevant. Information on materials is summarized in References 34 and 38 and new data on lightweight concrete reported in Reference 40.

Existing Structures. For such structures a distinction must be made between those that have not experienced an earthquake and those that have. Large changes in design philosophy and building code requirements have taken place in the last forty years and it is obvious that many structures in seismic zones have inadequate aseismic capacity. In addition many structures in such zones have experienced several earthquakes with damage accumulating from one quake to another. There is ample historical experience indicating that damage caused by a lesser quake if left unrepaired, can precipitate collapse in a subsequent quake. It is therefore important for designers to develop a consistent philosophy for assessing the safety of existing buildings and for retrofitting them. Similarly a designer needs to develop a consistent philosophy for assessing the degree of damage in a structure surviving an earthquake and for repairing that damage.

A thorough inspection of the building and its structural drawings should be the first step in any evaluation of its aseismic capacity. Knowledge of the year the building was constructed will help the engineer evaluate the philosophy used for its design and its possible structural deficiencies. Non-destructive testing using a rotating mass-vibration generator to excite low-amplitude elastic vibrations is another possible procedure for assessing the likely aseismic characteristics of existing buildings. While such tests provide valuable data on elastic properties and damping they cannot provide information on a structure's likely inelastic response. The paper by Gallegos and Rios outlines a consistent procedure for assessing damage in existing structures and for repairing and retrofitting those structures.

**Documents**

Document Sharing Hub

They illustrate the viability of their approach by showing how they have used it to successfully repair several structures.

#### ACKNOWLEDGMENTS

The editors gratefully acknowledge the co-operation of the U.S. Geological Survey in supplying the World Seismicity Map for the cover, the National Bureau of Standards in supplying photographs of earthquake damage from the San Fernando earthquake for the cover, the Associate Committee of the National Building Code of Canada in supplying Fig. 2, the International Conference of Building Officials in supplying Figs. 3, 4 and 5, and Dr. Luis Esteva of the Institute de Ingenieria, Ciudad Universitaria, Mexico for supplying Fig. 6. The World Seismicity Map was compiled by Arthur C. Tarr.

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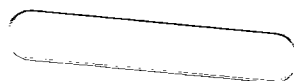
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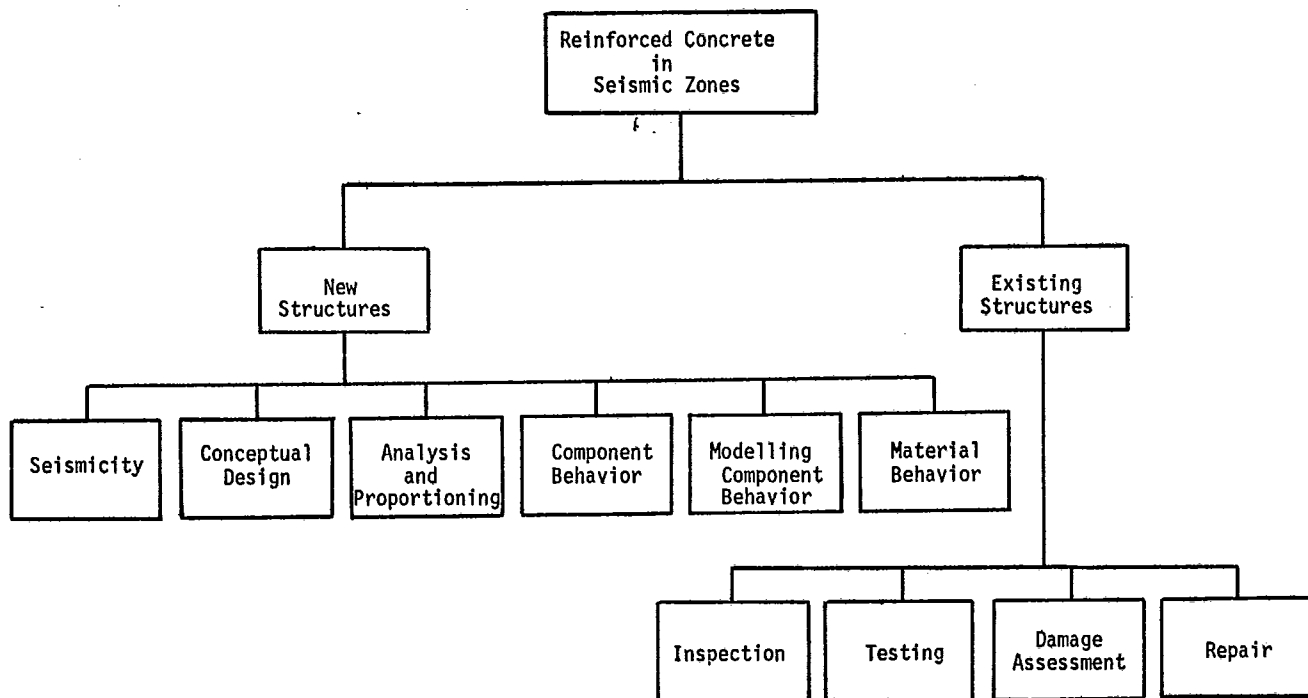


Fig. 1--Problems associated with concrete structures in seismic zones

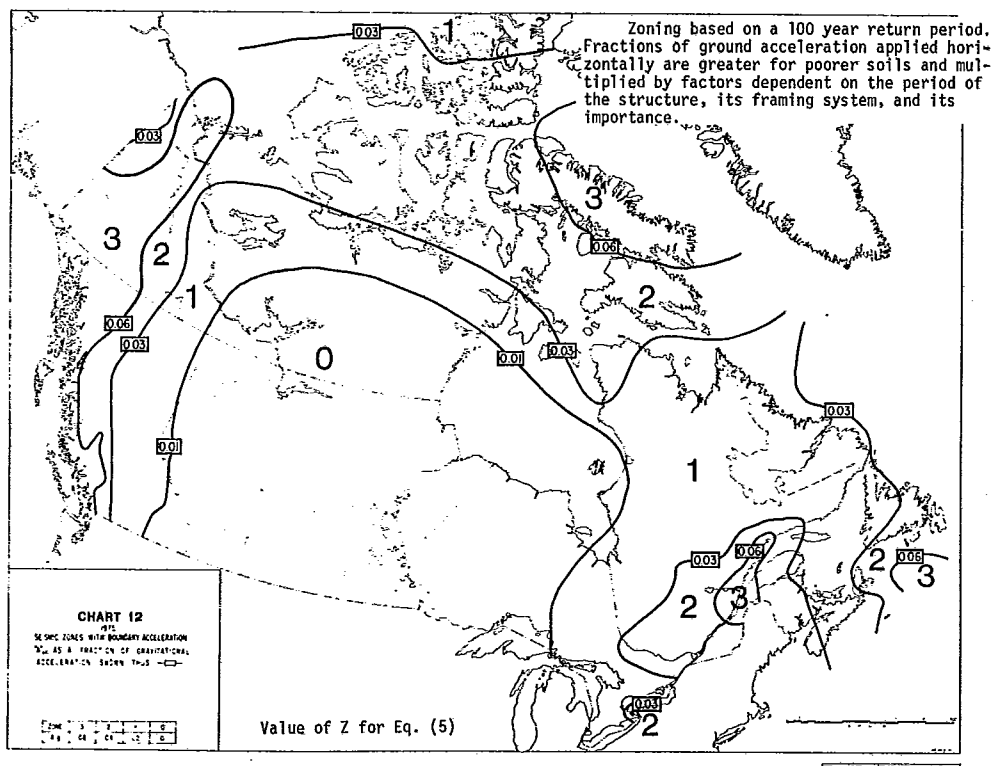


Fig. 2--Seismic zone map for Canada. Chart 12 of Supplement No. 1, National Building Code. Reproduced by permission of Associate Committee on the National Building Code

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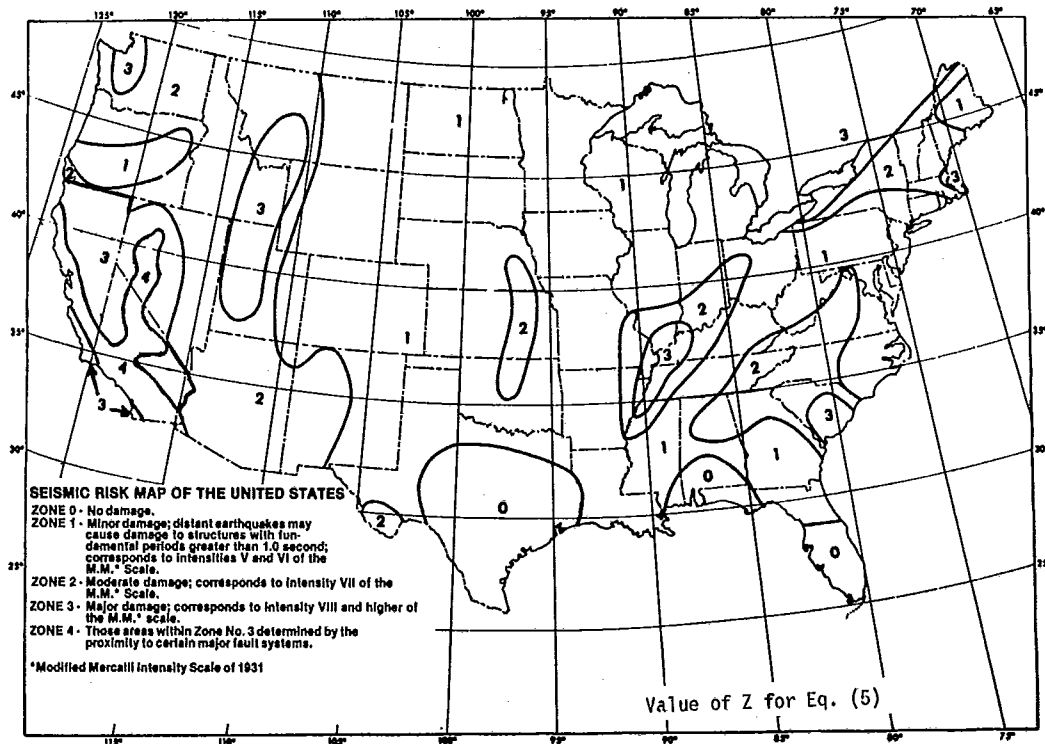


Fig. 3--Seismic zone map of the United States. Reproduced from the 1976 edition of the Uniform Building Code with permission from the International Conference of Building Officials

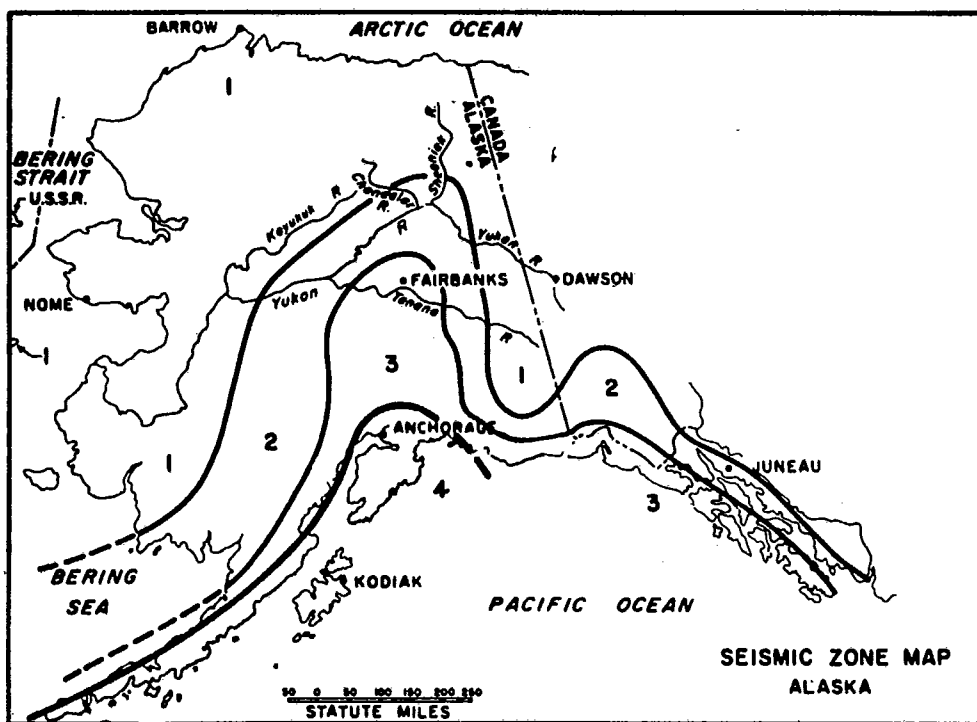


Fig. 4--Seismic zone map of Alaska. Reproduced from the 1976 edition of the Uniform Building Code with permission from the International Conference of Building Officials

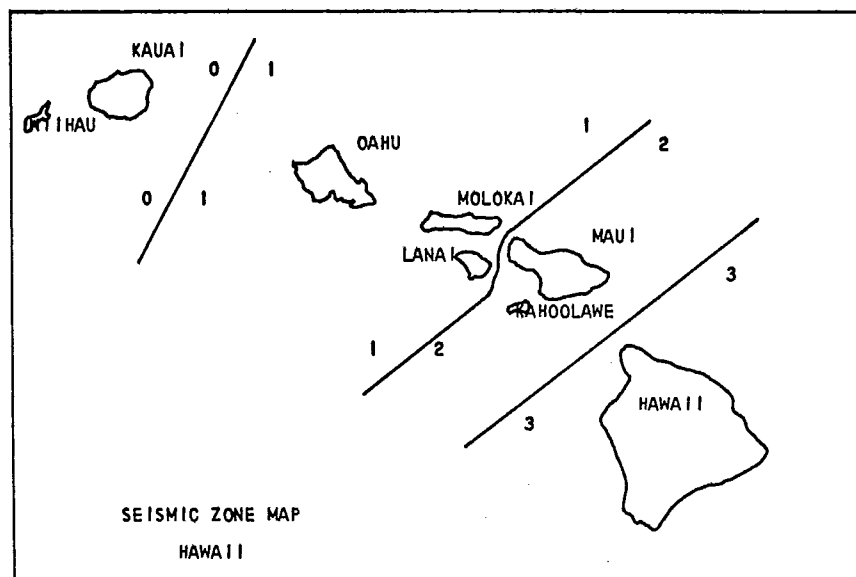


Fig. 5--Seismic zone map for Hawaii. Reproduced from the 1976 edition of the Uniform Building Code with permission from the International Conference of Building Officials



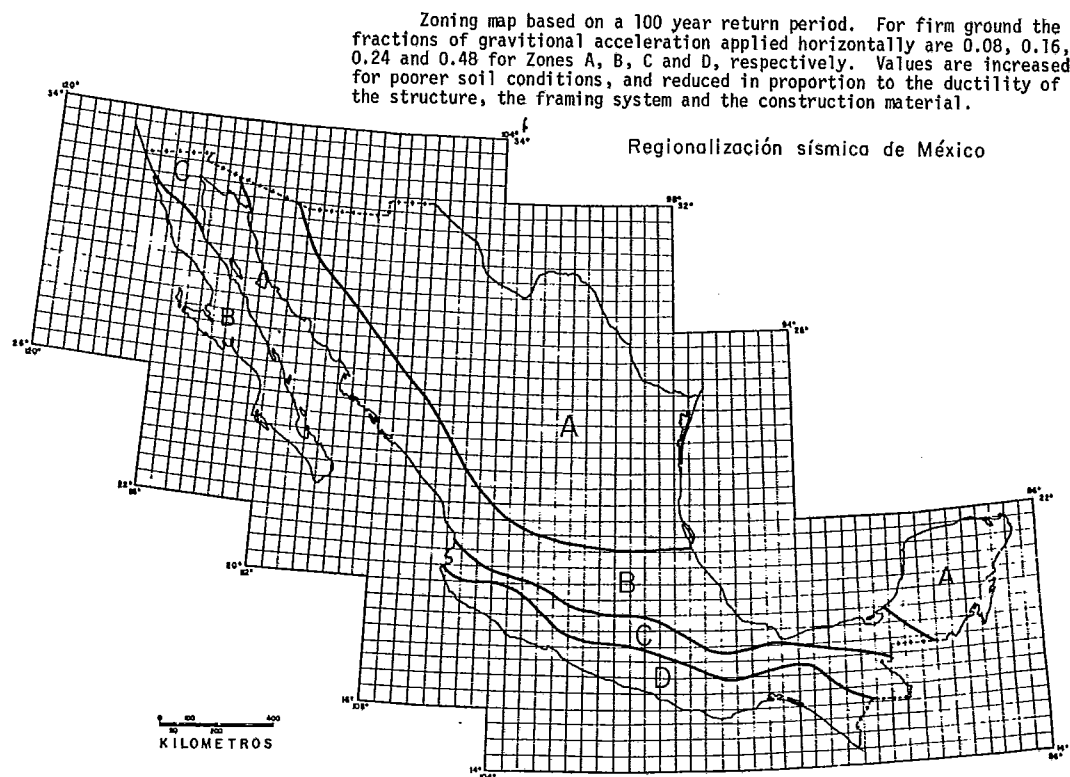


Fig. 6--Seismic zone map for Mexico. Developed for the Ministry of Public Works by the Instituto de Ingenieria, Ciudad Universitaria

**SP 53-1****The Earthquake Problem**

By Roland L. Sharpe

**Synopsis:** An overview of the earthquake problem is presented. How earthquakes occur, the resulting ground motions, effects of site conditions, response of structures, performance of concrete buildings and recent design improvements are discussed. Damaging earthquakes can occur in many areas of the world and therefore must be considered in the design of buildings of all types. Thousands of concrete buildings have been subjected to damaging earthquake ground motions and, in general, their performance has been reasonably good although some have collapsed. Careful quality assurance in design and construction and the use of improved design provisions, tempered with seasoned earthquake engineering judgment, can provide good performing concrete buildings.

**Keywords:** building codes; damage; deformation; ductility; earthquake resistant structures; earthquakes; foundation; geological faults; loads (forces); reinforced concrete; seismic waves; seismology; soil dynamics; structural analysis; structural design.

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## INTRODUCTION

The occurrence of earthquakes and their consequent impact on people and the facilities they live and work in have been with us since shortly after the earth was formed. Earthquakes are of special interest because they are the only natural phenomena that can affect nearly simultaneously all components of a structural system and the people and equipment supported therein. Earthquakes occur at irregular time intervals and transmit transient dynamic ground motions which in turn can induce the largest natural force that can be applied to man-made facilities. The response of structures and their contents to such dynamic motion is very complex.

Destructive earthquakes have been occurring in many areas of the world and the United States for millions of years with about 80 per cent of the world's earthquakes occurring around the rim of the Pacific Basin. As is apparent from Fig. 1, earthquakes have occurred in almost every state of the United States, although most occur in the western states. References to damage from earthquakes appear frequently since the beginning of historical records.

In this paper a review is made of some of the factors involved in the earthquake problem -- how earthquakes are caused, the significant characteristics of the induced ground motions, the effects of site location and foundation conditions on these ground motions, the response of structures to seismic motions, the performance of concrete buildings when subjected to earthquakes, and the measures that have been taken in the past few years to improve concrete structure performance.

## EARTHQUAKES AND INDUCED GROUND MOTIONS

Most earthquakes are caused by sudden ruptures or slippages of adjacent blocks of the earth's crust. The ruptures or slippages along which earthquakes have or can occur are referred to as active faults. Many active faults in California and elsewhere in the U.S. have been plotted by geologists. However, not all of them have been located -- especially where they are overlain with deep soil -- for example, much of the area east of the Mississippi River.

Earthquakes are unique in that the ground motions they generate can travel great distances and affect everything supported above or below the ground through which the waves pass. The ground waves emanate



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outward in all directions -- somewhat like the waves generated when a large object is dropped into a pond of water. Most of the seismic energy is transmitted through the earth's crust and then at a site is propagated up through the soil overlying the crustal bedrock.

Earthquakes induce transient dynamic and reversible cyclic ground motions that occur concurrently in all directions -- horizontally and vertically. Most earthquake motion records are made by seismographs which record, as shown in Fig. 2, acceleration versus time. As far as buildings are concerned, the most significant characteristics of seismic motions are their amplitude, distribution of motion frequency, phase relationships between the pulses and duration of strong motion. For most engineering purposes, an acceleration time history is difficult and too costly to use, and hence the response spectrum method of analysis was developed. A response spectrum is a plot of the maximum response value of some parameter, such as acceleration, experienced by a family of single-degree-of-freedom, spring-mass systems when the systems are subjected to a time-history of input ground acceleration. Since real structures themselves behave as spring-mass systems having natural periods of oscillation dependent upon their geometry and construction, response spectra are useful in the analysis of structures and in the evaluation of observed damage to structures exposed to the earthquake ground motion. Shown in Fig. 3 are response spectra curves for several damping values for an input acceleration normalized to 1.0 g. These curves are also often plotted on tri-partite log paper to show acceleration, velocity and displacement versus period or frequency.

### EFFECTS OF SITE LOCATION AND FOUNDATION CONDITIONS

Seismic wave motions attenuate as they emanate outward from the epicenter of the earthquake. The high frequency components attenuate more rapidly than the lower frequencies. Sites near active faults can thus be expected to have high amplitude motions with considerable high frequency content. Conversely, sites at some distance from an active fault can be expected to have less intense motion made up primarily of lower frequency components.

As noted previously, seismic motion is transmitted through the base rock and then up through the overlying soil. Certain soil characteristics, such as the seismic shear wave velocity, appear to modify the motions as they pass upward to the surface. Softer soils tend to amplify the lower frequency components of motion while firmer soils amplify the higher frequency motions. Studies (1) of the 1957 San Francisco earthquake indicate that local soil conditions amplified as well as attenuated ground motions.

### RESPONSE OF STRUCTURES TO SEISMIC MOTIONS

Structure response to seismic motion is complex. The response is interdependently affected by the characteristics of the input motion (motion frequencies and their distribution, amplitude, phase



relationships, and duration of strong motion), the stiffness characteristics of the foundation materials, and the dynamic response characteristics of the structure.

The input motion pulse frequencies and their distribution can have a major effect on structure response. For example, if a series of motion pulses with frequencies near those of a structure precede several larger pulses with frequencies also close to those of the structure, the structure response will be greater than if the larger pulses were preceded by pulses with greatly different frequencies.

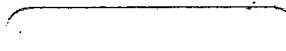
Generally as amplitude of the input motion increases, the response of the structure increases. Research on the significance and effects of phase relationships on structure response has only recently begun (2). Phase relationship effects could be important in explaining the extreme response of some structural components or equipment to seismic motions.

The duration of strong motion significantly affects a structure's response because the time of maximum response is longer. One or two excursions to yield or beyond does not cause near the damage of ten or more such excursions. The larger number of cycles of maximum deformation can cause considerable cracking and spalling of brittle-type materials such as concrete.

The effects of the stiffness characteristics of foundation materials have been and are continuing to be extensively studied. Since seismic motions are transmitted to structures through the underlying foundation materials, the structure's response to ground motion is affected by local soil or rock conditions. Numerous observations concerning the influence of local soil conditions on structure damage have been documented -- starting with the 1906 San Francisco earthquake, where many buildings on bay fill suffered more extensive damage than those on hard rock. At Kanto, Japan, in 1923, rigid buildings founded on rock were more severely damaged than those on deep alluvium.

Studies by Scholl (3), Seed (4), and others indicate that structure response may be affected by the relative frequencies of the structure and the supporting foundation materials, and by the damping of the soil-structure system. The Imperial Hotel in Tokyo, designed by Frank Lloyd Wright and completed in 1921, was designed to "float" during an earthquake. The building was much stiffer than the underlying soils. The hotel withstood the 1923 Kanto earthquake with essentially no damage while thousands of other structures were badly damaged or collapsed. Studies have also indicated that in many cases it is possible local soil conditions have a greater effect on the response of low-rise than for high-rise structures (5).

The dynamic characteristics of a structure affecting its response to earthquake motions include its mode shapes and periods of vibration. The building mode shapes and periods are functions of the building mass and stiffness and their distribution, including the variations in stiffness of adjacent floors or sections. The arrangement,



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connections and stiffness of non-structural elements such as window walls and partitions can also materially affect building response, especially during the initial cycles of response (6,7).

Concrete buildings or structures designed to resist significant seismic forces usually have lateral force resisting systems comprised of moment resisting frames, shear walls, or combined moment resisting frame-shear wall construction. Figure 4 illustrates how these three different systems respond. As shown in Fig. 4(a), moment resisting frames respond as shear beams, shear walls, as shown in Fig. 4(b), respond as cantilevers, and combined frame-shear walls respond, as shown in Fig. 4(c), as a cross between shear beam and cantilever.

Experience has shown that earthquake loadings can substantially exceed those specified in present building codes (8). Strong motion records of earthquakes support these observations (9) while analytical studies of damaged structures also confirm this fact. Because the actual forces may greatly exceed the design forces, structures may be deformed beyond the elastic limit in order to absorb all of the energy imparted to them. In such cases ductility, the ability of a structure to undergo increasing deformations beyond yield stresses while still sustaining gravity and other loads, is therefore necessary in order to prevent catastrophic collapse. However, over-reliance on ductility to provide energy absorption is also inappropriate. Excessive structure deformations can cause considerable damage to structural members and non-structural elements such as exterior walls, windows, partitions, ceilings, heating and ventilating systems, and other mechanical-electrical components (10). Deformations must be controlled within limits or else the building components must be designed to tolerate large deformations without significant damage (11).

### PERFORMANCE OF CONCRETE BUILDINGS SUBJECTED TO SEISMIC MOTIONS

The performance of concrete buildings subjected to earthquake motions has been both good and bad. Where building components were well tied together, and the lateral force resisting system and non-structural elements designed to accommodate or minimize deformations, the results were generally good. Where the design did not consider that excessive stresses could be induced by building flexibility, building irregularities, or excessive deformations, the performance has ranged from minor damage to complete failure.

There are several important considerations which can affect the performance of concrete structures. Concrete members designed so that the reinforcing governs their capacity will exhibit ductile behavior. However, when the concrete is overstressed instead of the reinforcing, the failure is likely to be sudden and catastrophic. Such failures can result from:

- compressive failure of concrete,
- shearing failure of concrete,
- loss of reinforcing anchorage.

The performance of concrete buildings during the 1967 Caracas, 1971 San Fernando, and the 1972 Managua earthquakes illustrates the range of results and types of damage.

Caracas, Venezuela, July 24, 1967 (12). The Caracas earthquake subjected about 7,000 reinforced concrete buildings over four stories in height to a severe test. Of these, about 180 suffered serious damage. However, except for some 25 to 30 buildings showing significant structural distress, the damage was mostly non-structural. Figure 5 shows a concrete compression failure due to excessive moment at a joint, Figure 6 shows a compression failure of a concrete column, Figure 7 shows a shear failure in a beam and Figure 8 shows shear failure in a column.

The 8-story Atlantic Building, shown in Figure 9, survived the earthquake with only minor damage -- shear cracks in columns on the third, fourth and fifth floors. The 15-story Mi Veguita Building, shown in Figure 10, also exhibited only minor structural damage. As shown in Figure 11, hinges developed in many lower floor beams while some beams also exhibited diagonal shear cracks. Many buildings in Caracas contained clay tile or block partitions that stiffened the building frames and adversely changed their response from that anticipated during design.

San Fernando, California, February 9, 1971 (13,14). The San Fernando earthquake also demonstrated that the dynamic characteristics of structures should be considered in their design, especially if the building is irregular in plan or has significant variations in stiffness between adjacent floors. The severe damage to Olive View Hospital, shown in Figure 12, vividly demonstrated the effects of variations in stiffness between floors. The upper four stories had shear walls to resist lateral forces while the lower stories were moment frames that deflected almost two feet.

The Indian Hill Medical Center, shown in Fig. 13, was a shear wall-frame type building. It performed quite well although the shear walls exhibited some diagonal cracking and other localized distress. The building was back in operation shortly after the earthquake.

A failure due to loss of reinforcing anchorage is shown in Fig. 14. In this case a large concrete pier supporting an overpass structure collapsed because the reinforcing pulled out of the foundation.

Managua, Nicaragua, December 23, 1972 (15,16). The Managua earthquake caused extensive and serious damage to many structures in the area. However, two concrete buildings are of particular interest because of their good performance. The Banco de America Building is an 18-story shear wall-frame structure. Its symmetrical arrangement of core walls is apparent from Figures 15 and 16. The beams connecting the core walls at each floor suffered repairable shear damage. The remainder of the building including non-structural elements was not damaged. The building remained in use while repairs were made.



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The 5-story Enaluf Building, shown in Fig. 17, endured the earthquake motions with only minor horizontal cracking in some of the exterior first story columns and some shear cracking near a duct opening in a ground floor shear wall.

Numerous other examples of good and poor performance are documented in various reports on the Managua earthquake. Historical evidence proves that properly designed and engineered concrete structures will perform well during earthquakes. Shear wall stiffening or connection details that allow for differential movement of the structural frame and the non-structural elements are especially conducive to satisfactory performance.

### RECENT IMPROVEMENTS IN DESIGN PROCEDURES

Careful evaluation of the earthquake damage that has occurred to reinforced concrete structures has led to many recent improvements in design procedures. Ductile concrete requirements and increased reinforcing and better connections for shear walls have been two major steps. In 1973 a requirement was incorporated into both the Uniform Building Code (17) and the Structural Engineers Association of California (SEAOC) "Recommended Lateral Force Design Requirements" (18) specifying that all concrete frames required by design to be part of the lateral force resisting system and all concrete frames located in the perimeter line of vertical support shall be space-frame-ductile-moment-resisting (SFDMMR). Such frames are also required for all structures over 160' in height and all structures designed for horizontal force factor "K" of 0.67 or 0.80.

Frame members may be enclosed or adjoined by more rigid elements where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting capability of the space frame. All braced frame members must be designed for 1.5 times the force calculated by using the Code shears and the member connections must develop the capacity of the member without using the one-third increase allowed for seismic forces.

Code provisions for ductile concrete require design and detailing procedures which will inhibit failures from compression, shear, or loss of anchorage. The necessary ductility for a concrete SFDMMR is provided by complying with SEAOC Section 2630 (18), UBC Sections 2625 or 2626 (17) and ACI 318-71, Appendix A.

Compression failures in beams or columns are controlled by requiring confinement or special transverse reinforcing of the longitudinal reinforcement in the manner shown in Fig. 18. Such confinement substantially increases the strain capacity of the concrete and also increases its compressive strength. Maximum confinement is required in columns or beams at or near beam-column connections. In addition, columns and beams must be proportioned so that inelastic deformations or hinging will occur in the beam and not the column, thus minimizing ductility demands on the column.



Special attention to anchorage of the reinforcing bars is required because both ductility and confinement are dependent upon proper anchorage. Flexural reinforcement must be anchored into or beyond the column. If anchored within the column, anchorage must, as shown in Fig. 19, be within the zone of confined concrete. Stirrups, hoops and stirrup ties must continue to be effective even if the concrete shell of the member spalls off. Therefore, 135 degree bends are required for ties with sufficient extension to provide anchorage within the core of the member.

#### ONGOING EFFORTS TO IMPROVE SEISMIC DESIGN

The Structural Engineers Association of California (SEAOC) Seismology Committee has been a leader since the 1950's in developing seismic design provisions. In October 1971, SEAOC authorized the formation of the Applied Technology Council (ATC). The Council has the following objectives:

- (1) The fostering and coordination of research efforts in earthquake and other aspects of structural engineering.
- (2) The review and evaluation of the voluminous research results being published on earthquake and structural engineering.
- (3) The translation of these results into a form usable and understandable by the practising structural engineer.

ATC has not supplanted the SEAOC Seismology Committee; rather, the two complement each other.

In December 1973, ATC received a contract from the National Bureau of Standards (NBS) with funding by the National Science Foundation (NSF) and NBS\* to develop comprehensive seismic design provisions which could be adaptable to all jurisdictions in the U.S. The areas of design ground motion, risk analysis, soil-structure interaction, structure response and design, and architectural-structural interaction, structure response and design, and architectural-structural interaction requirements, plus mechanical and electrical systems are included.

The first phase of the work developed the philosophy, scope, and format of the provisions, and the organization, budget and schedule for writing the provisions.

Two advisory groups were established, a Seismic Design Review Group (SDRG) and a Building Code Consultants Group (BCCG). The SDRG members represent the top expertise in both academic and professional

\* The project is part of the Cooperative Federal Program in Building Practices for Disaster Mitigation initiated in the spring of 1972 under leadership of the National Bureau of Standards.



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areas. In addition to structural and soils engineers, architects, mechanical and electrical engineers, and engineering geologists are represented. The membership of the BCCG is composed of seven individuals, two from the west, three from the midwest, and two from eastern U.S.A.

The completed design provisions will be written to ensure that:

- (1) The design professions, officials and public are aware of what the design provisions will or will not do.
- (2) Buildings are designed to survive (with acceptable damage) the maximum ground motion that has a relatively small probability of being exceeded during the life of the structure. This does not necessarily mean that function will be maintained.
- (3) Structural and non-structural damage are limited so that life hazard is minimized. Life-safety provisions are to apply to structural and non-structural elements. For critical facilities, structural and non-structural systems should have damage limits related to maintenance of function.
- (4) Design is emphasized rather than analysis.
- (5) Design provisions are written so as to not inhibit ingenuity of solution but with definite criteria given for evaluating the resulting design.

### CONCLUSIONS

Concrete structures have resisted earthquake motions quite well as a whole, but there have been a number of poor performances. The profession has gained considerable knowledge of how to improve the performance of concrete structures. Designing in accordance with the requirements of the latest seismic codes should result in generally good performance.

Quality assurance in design and construction is mandatory if concrete structures are to perform adequately during earthquakes. A design can only be effective if the drawings and specifications clearly present the design requirements and the designer must be sure his design can be constructed. Care must be exercised to ensure that the actual structure is built in accordance with the drawings and specifications. Earthquake engineering is still an art and not a science. Seasoned engineering judgment is a critical ingredient.

### ACKNOWLEDGEMENTS

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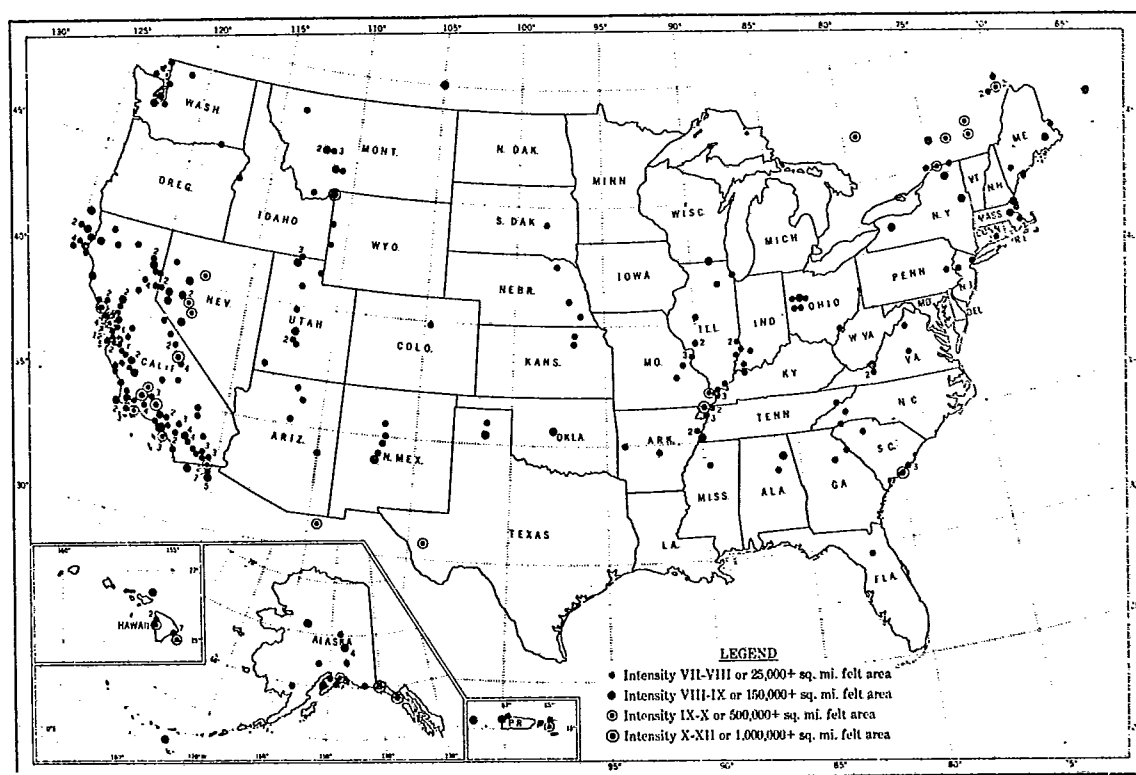


Fig. 1--Damaging earthquakes of the United States through 1967 (prepared by the National Earthquake Information Center)

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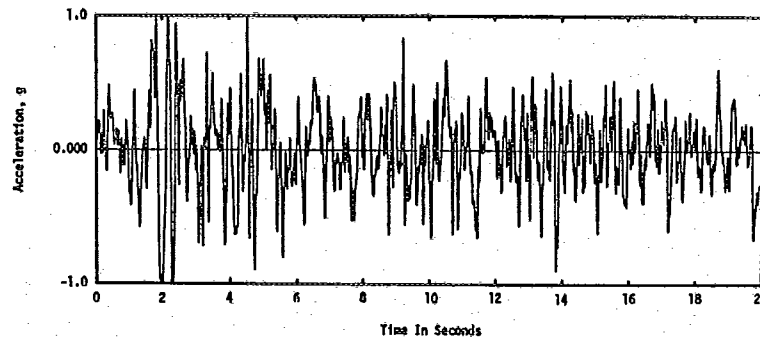


Fig. 2--Acceleration time-history

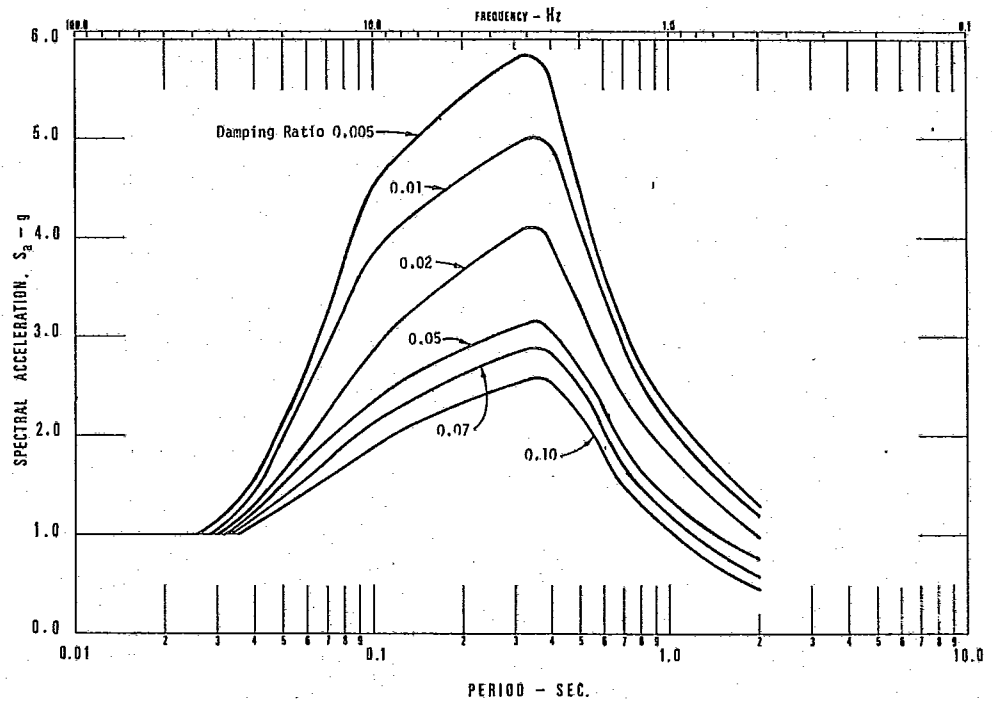


Fig. 3--Response spectra

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Sharpe

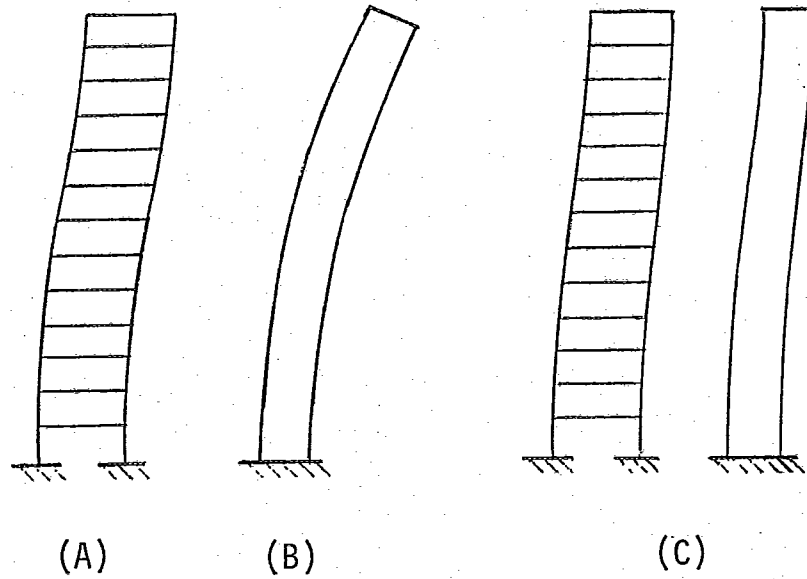


Fig. 4--Building response

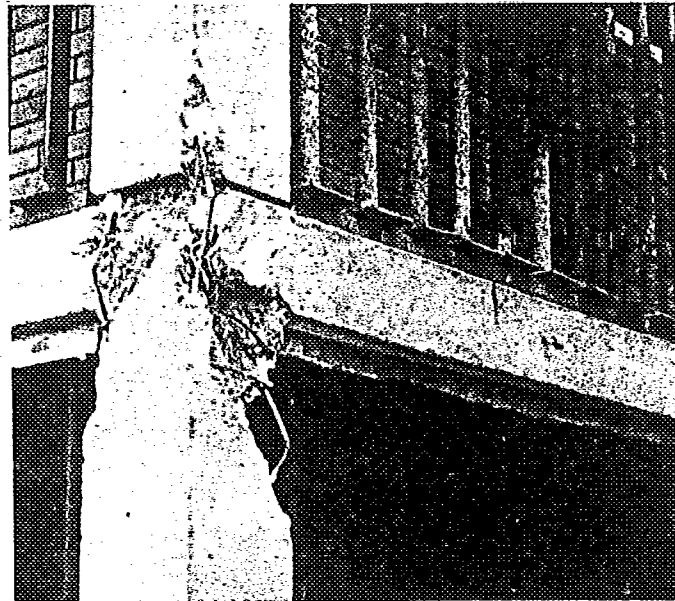


Fig. 5--Moment compression failure



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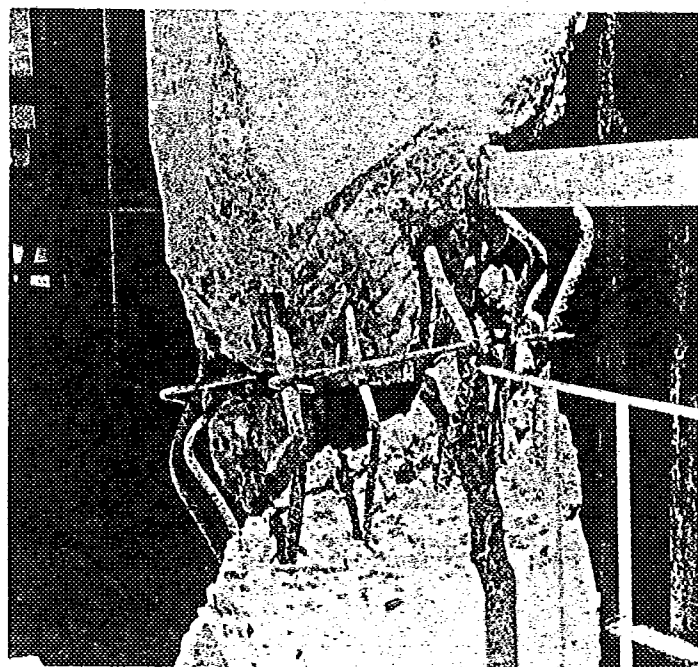


Fig. 6--Compression failure in column

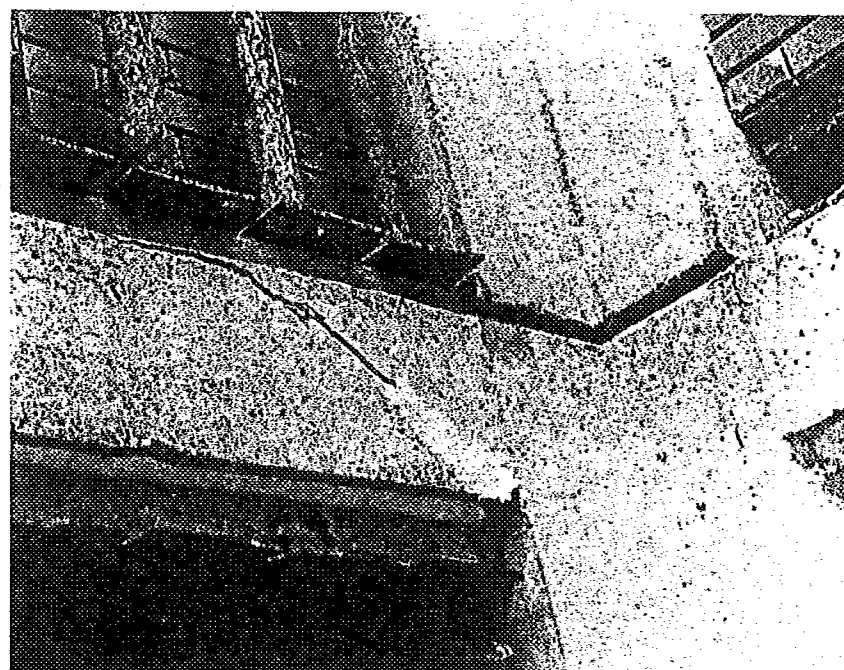


Fig. 7--Shear failure in beam



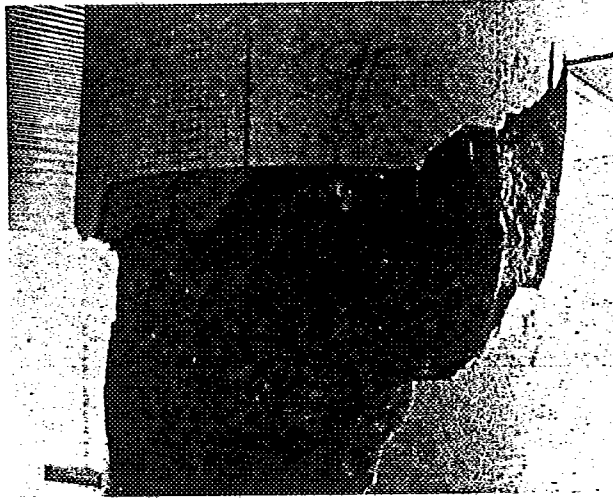


Fig. 8--Shear failure in column

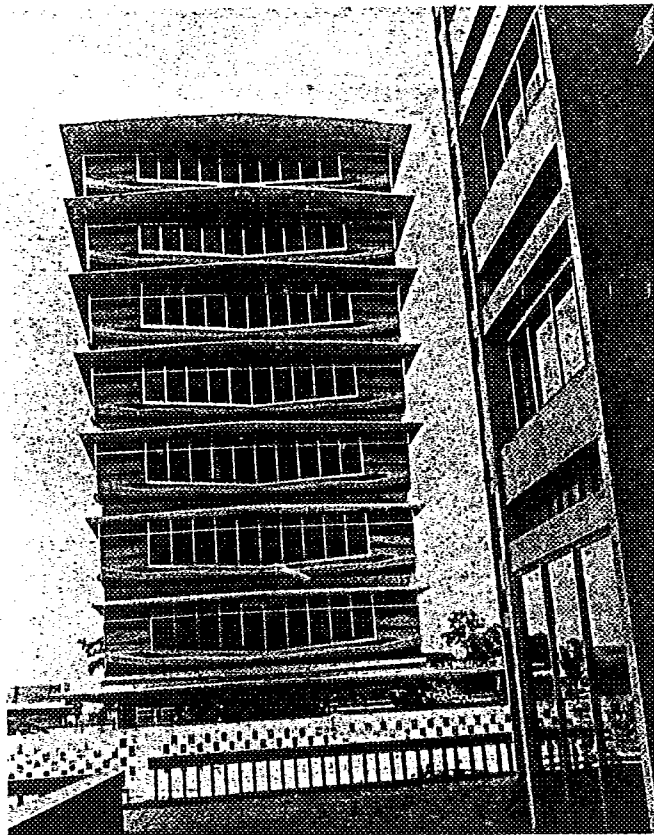
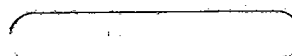
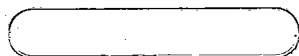


Fig. 9--Atlantic building, Caracas, Venezuela



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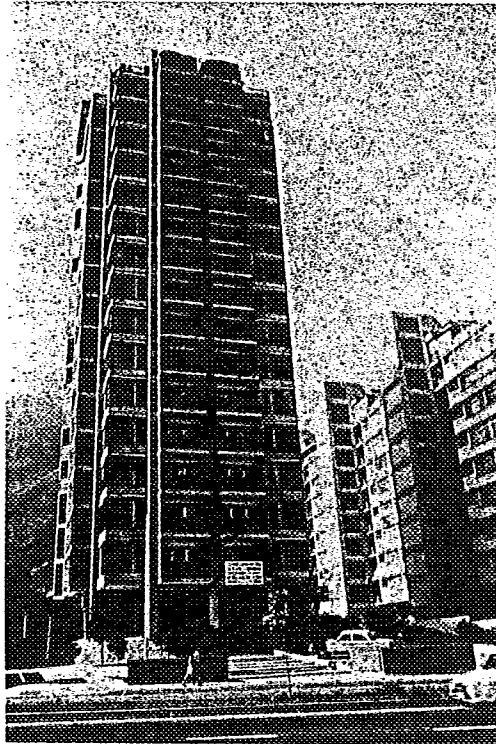


Fig. 10--M1 Veguita Building, Caracas, Venezuela

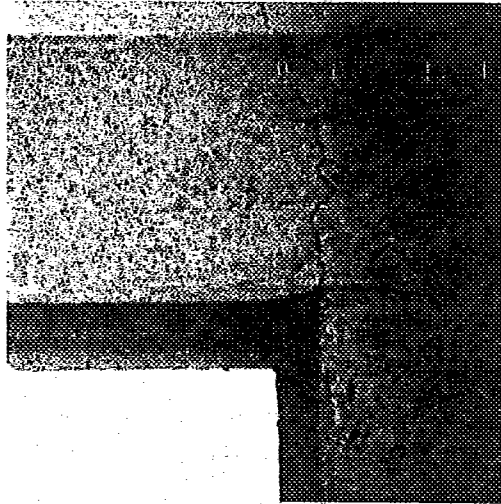


Fig. 11--Hinging in beam

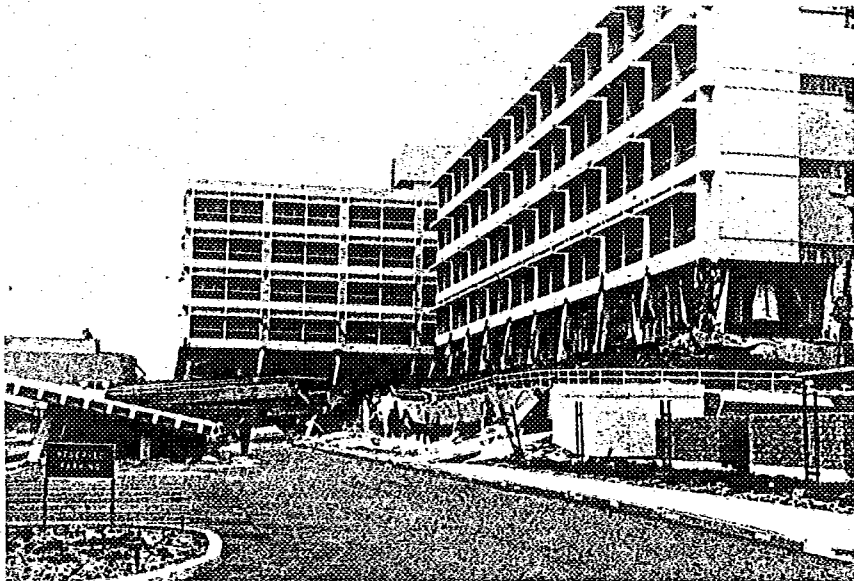


Fig. 12--Olive View Hospital, San Fernando, Calif.

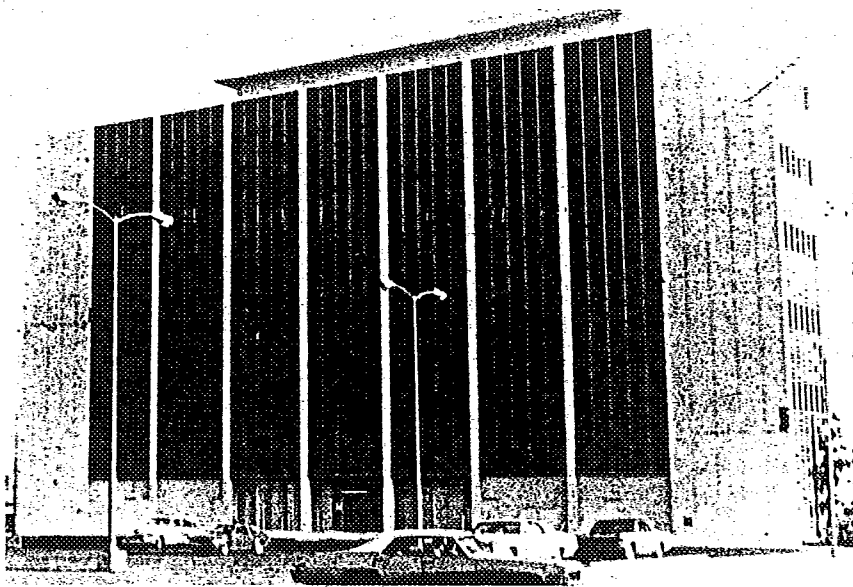


Fig. 13--Indian Hill Medical Center, San Fernando, Calif.



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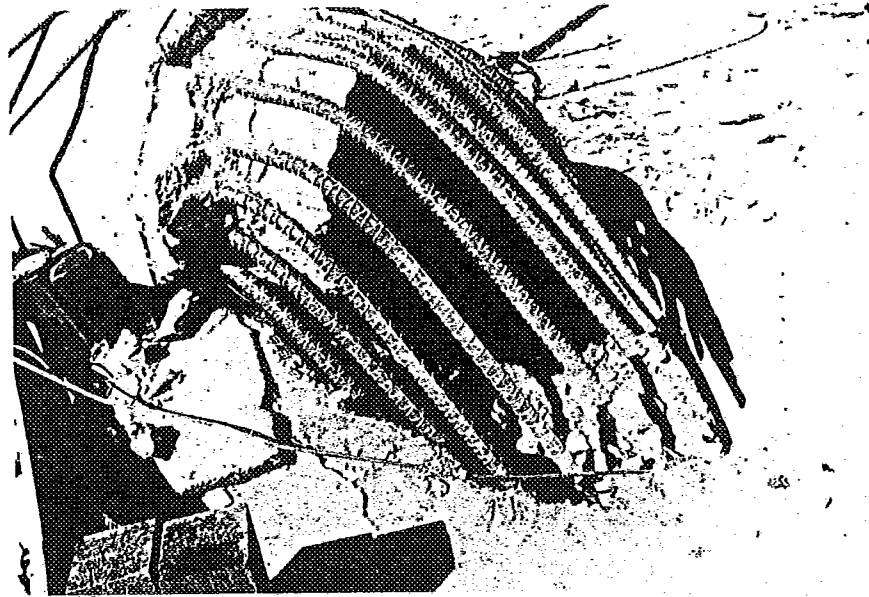
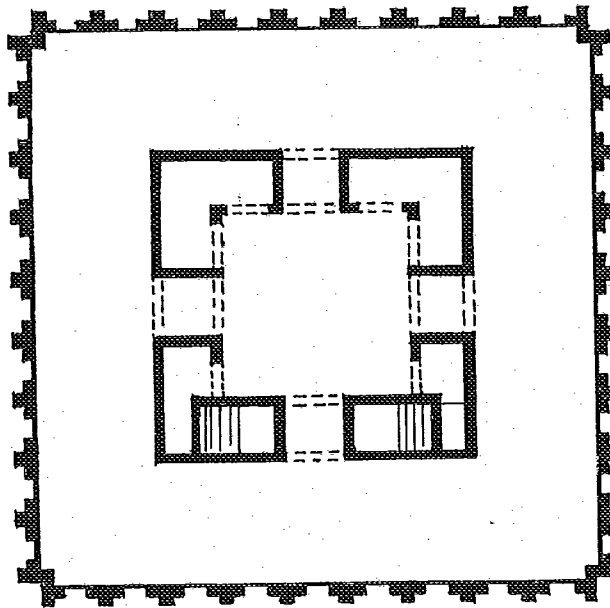


Fig. 14--Loss of reinforcing anchorage

Fig. 15--Banco de America Typical floor plan--  
Banco de America, Managua, Nicaragua

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Sharpe

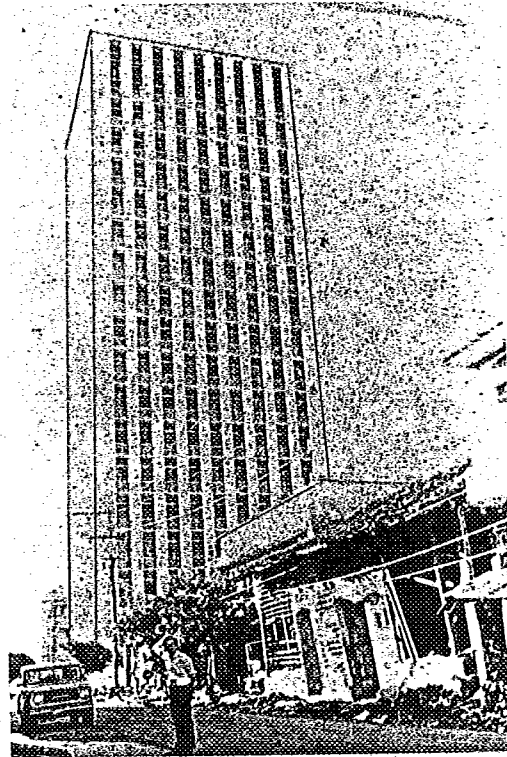


Fig. 16--Banco de America, Managua, Nicaragua

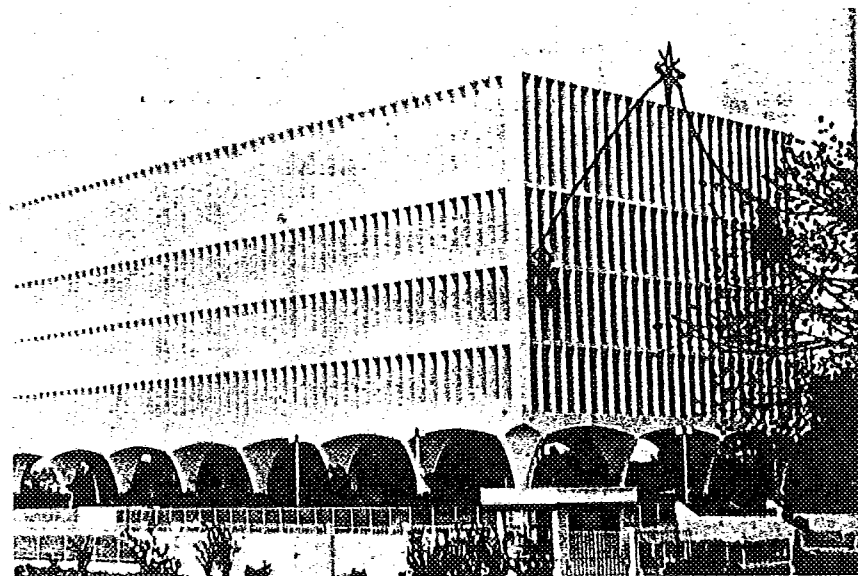


Fig. 17--Enaluf Building, Managua, Nicaragua

