Major issues and future directions in earthquake-resistant design

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**ABSTRACT:** After general remarks about the nature of the earthquake (EQ) problem, the conditions that determine the occurrence of an EQ disaster, and the resulting seismic risks in our urban and rural areas, the role and importance of EQ Engineering, particularly EQ-Resistant Design (EQRD) and EQ-Resistant Construction (EQRC) in the overall problem of controlling these risks are discussed. The need for EQ preparedness programs is emphasized. Analysis of events of the last four years since the 1988 WCEE shows that seismic risks have increased rather than decreased. Evaluation of the lessons learned from recent EQs and research results provides reasons for this increase, points out that the lessons learned are quickly forgotten or not taken seriously, and indicates the need for EQ preparedness to control these risks. This will require control of the built environment, which is a complex problem requiring integration of knowledge and collaboration of experts from many different disciplines. In a brief evaluation of the state of the knowledge and particularly the state of the practice in EQRD of new structures and seismic upgrading of existing facilities, major issues and pressing problems requiring short-term solutions are identified. Emphasis is placed on identifying the main issues in the formulation and application of seismic code procedures in EQRD, particularly regarding the establishment of reliable design EQs. Solutions are suggested based on an energy approach. A conceptual methodology for the numerical design part of EQRD is proposed. This methodology is based on well-established fundamental principles of structural dynamics, mechanical behavior of entire facility systems and comprehensive design, and is in compliance with the world-wide accepted philosophy of EQRD. The main advantages of this conceptual methodology are discussed. The lecture concludes with pleas, first to all of the experts in the different disciplines that are involved in solving the EQ problem to collaborate closely toward a timely solution to this problem; and secondly to all of the participants in this 10th WCEE to screen the research and development (R&D) results that are presented at this conference and disseminate in their communities those that can be applied immediately, and also to collaborate in the preparation and implementation of EQ preparedness programs.

1 INTRODUCTION

1.1 Introductory remarks

Thank you very much, Dr. Blásquez, for your kind introduction. Professor Grandori, president of the International Association for Earthquake Engineering (IAEE), members of the Steering Committee for the Tenth World Conference, officers and national delegates for the IAEE, distinguished guests, fellow participants, ladies and gentlemen:

It is a privilege to attend this Tenth World Conference on Earthquake Engineering (10th WCEE), and a great honor for me to participate through this keynote lecture in this significant event. This gathering of more than 1,500 leading researchers and practitioners from so many countries, with its impressive technical program, gives clear evidence of the remarkable progress in this so important field since the first WCEE in 1956. It is indeed a great pleasure every four years to see and renew relationships that I have had over the years with so many colleagues around the world and with my ex-students and research associates working in the field of EQ engineering. It is heartening to see so many young engineers and scientists getting together with those of us who have been working for many years in this field, because all of these young researchers and practitioners will be conducting the needed research and development (R&D) and implementing the results of such R&D in the field to reduce the seismic risks in our urban and rural areas. The importance of implementation cannot be overemphasized: there cannot be reduction of seismic risks without implementation of research results and developments in the field. This in turn requires transfer of knowledge to practitioners and to government officials who manage the natural disaster programs. It would be of great interest to know how many of the 1,500 participants are practitioners and government officials, because they are the ones who will make possible the so-needed improvement in seismic risk reduction by implementing the results of R&D. To achieve this implementation it will be necessary to formulate and implement massive and comprehensive education programs. This lecture, which has the main objectives and scope outlined below, will emphasize the need for such education and implementation programs.

1.2 Objectives and scope of the lecture

When I was approached to prepare and deliver this keynote lecture, my first intention as a teacher and researcher was to prepare a technical lecture on "Major issues and future directions in Earthquake-Resistant Design (EQRD) of new structures and seismic upgrading of existing hazardous facilities." Then a series of events,
including recent EQs and my participation in a series of committees on EQ hazard reduction, led me to change my mind and start looking more deeply at the main objective of the IAEE, which is: "To promote international cooperation among scientists and engineers in the field of EQ engineering through the interchange of knowledge, ideas and the results of research and practical experience." I questioned whether this is all that the IAEE expects, and I questioned the purpose of promoting interchange. The answer is in the definition of EQ Engineering: "EQ Engineering is the branch of engineering that encompasses the practical efforts to reduce, and ideally to avoid, EQ hazards."

Clearly, EQ Engineering is of great importance to human life and welfare. I decided to look at the role and importance of EQ Engineering, particularly EQRD and EQRC, not only as a researcher, but also from a humanistic viewpoint. I decided to present a lecture with the following main objectives and scope:

1. To make some general remarks about the nature of the EQ problem and the resulting seismic risks in our urban and rural areas.
2. To review the role and importance of EQ Engineering, particularly EQRC of existing and new facilities, within the overall problem of reducing seismic risks in our urban and rural areas to acceptable levels, which should be the ultimate goal of EQ Engineering.
3. To discuss briefly where we are in our knowledge and particularly our practice in EQRD of new structures and the seismic upgrading of existing constructions. (This discussion will be based on lessons learned from recent EQs and associated research. Most of these lessons are discussed in the "Primera Conferencia Internacional Torroja," 1989 by the lecturer, whose publication has been distributed to the participants of this 10WCEE). This will involve: (a) an attempt to identify the pressing problems requiring reliable solutions in the short term; and (b) suggestions for future directions for our R&D and practice for finding and implementing such required solutions.
4. To conclude with: (a) first, a brief discussion of the need for a massive and comprehensive education program to transmit our present knowledge, permitting implementation of EQ preparedness programs and results of R&D in EQRD and EQRC, which will lead quickly to reduction to acceptable levels of the seismic risks in urban and rural areas; and (b) a plea to experts in the different disciplines involved in solving the EQ problem to collaborate closely toward a timely solution.

It should be noted that all that I will say has been said in previous WCEEs. The only difference is in the things that I believe should be emphasized so that seismic risk can be reduced quickly to a socially and economically acceptable level. These emphases will be on: first, a thorough assessment of the results of EQ Engineering R&D that has been conducted and can be immediately applied to improve the state of the practice in this field through formulation of better seismic codes (format and regulations) for EQRD and EQRC of new structures and upgrading of existing hazardous facilities than those that are presently enforced. This will require collaboration of experts from the many disciplines involved in EQ Engineering. Second, comprehensive EQ preparedness programs must be formulated; and third, there is a need for a massive and comprehensive education program to facilitate systematic and strict implementation of preparedness programs and new codes.

2. THE NATURE OF THE EQ PROBLEM; OCCURRENCE OF AN EQ DISASTER; AND CONTROL OF SEISMIC RISKS

1. The nature of the EQ problem

The nature of this problem has been very well defined by Press (1984): "Earthquakes are a very special type of natural hazard in the sense that they are very rare, low-probability events, whose consequences, when they do occur, are very large in terms of destruction and suffering." Hazards with these characteristics create the difficult public policy problem of how to sustain public interest and involvement and attract adequate government resources for mitigation programs. Only countries with recent catastrophes become concerned and organize national programs. Unfortunately, a few years after the catastrophe, efforts to implement these programs usually decrease and fall into neglect. It is a high duty for a civilized society to anticipate and control, rather than react only after, a disaster.

EQs are natural disasters whose feature is that most of the human and economic losses are not due to the EQ mechanisms but to failures of human-made facilities: buildings and lifelines, such as dams, bridges, transportation systems, etc., which supposedly were designed and constructed for the comfort of human beings. Although this is depressing, it is also fortunate and encouraging, because it tells us that in the long run the EQ problem is in principle solvable.

With sufficient resources for R&D, formulation of EQ preparedness programs, and education needed for the implementation of the results of this R&D, EQs are hazards to which it is in our power to respond effectively. We can reduce their threat over time by as much as we want. We can learn where not to build and of how to build so that facilities will not fail. On the average, more than 10,000 EQs are recorded each year, of which about 60 are significant (producing significant damages or having a magnitude M≥5.5). Between 1890 and 1978, an average of more than 10,000 people died in EQs per year (Fig. 1) and nearly 500,000 people were left homeless. Economic losses due to physical damage amount to about $10 billion per year.

The psychological impact on millions of people who experience major EQs is an enormous, complex fear that remains a nightmare to them for many years. Thus, it is

Figure 1. Loss of life caused by major EQs (after Hiroo Kanamori, 1978)
of importance that we engineers and scientists attempt to
find the reasons for EQ disasters and to eliminate or
reduce the potentially catastrophic consequences of
major EQs.

2.2 Occurrence of an EQ disaster

2.2.1 EQ disaster potential. Four conditions determine
the occurrence of an EQ disaster. The first is the
magnitude of the EQ, since a small-magnitude EQ
will not induce ground-shaking severe enough to produce
extensive damage. Secondly, the EQ source must be
sufficiently close to the urban area, because at greater
distances the ground-shaking attenuates below the
level required to induce serious damage [it should be
noted that EQ disasters can occur at distances from the
source considerably greater than the 160 to 240 km at
most which has usually been assumed: 400 km (in the
1957 and 1985 Mexico EQs) and over 500 km (in the
1972 Caucete (Argentina) EQ). These experiences show
that under special conditions damage can occur at
epicentral distances even larger than 400 km. Thirdly,
the size and distribution of the population and the
economic development (high-technology industries); and
fourthly, the degree of EQ preparedness, determine the
possibility of disaster. Obviously, the potential disaster
becomes greater the larger and nearer the EQ is to an
urban center, the larger the population, the greater the
economic development, and the poorer the preparation.

Clearly, EQ hazard depends not only on the seismicity
of the region, but also on its population density,
economic development, and degree of preparedness. In
this respect, EQ hazards are becoming more important
each year. Even though seismicity remains constant, the
uncontrolled and rapid increase in population, urbanization and economic development of our urban
areas are not being counterbalanced by an increase
in preparedness. For example, in terms of population and
economic development, disaster potential in California
is now at least ten times what it was at the time of the
1906 San Francisco EQ [Committee on EQ Engineering
Research, 1982].

In 1988, in Tokyo, during the 9th WCCEE, Professor
Hudson [1988], in the conclusion to his keynote address, questioned the overall effectiveness of
the IAEÉ's work on the following questions: (1) To
what extent have our efforts helped to reduce the threat
of EQs? (2) How far along the way are we to
preventing EQs from becoming disasters?

Professor Hudson's conclusion was, "We must hasten
to admit that we are very far from our goal."

There is no doubt that there has been an impressive
increase in knowledge in EQ Engineering, particularly
in the seismic upgrading of existing hazardous facilities.
However, if today at the 10th WCCEE, we ask, "How
effective has this increase in knowledge been in
reducing the seismic risks in our urban and rural
areas?" we will have to give practically the same
answer. We are very far from our main goal, which is
the reduction of seismic risks in our urban and rural
areas. Why is this so? An answer to this question can
be obtained through the following analysis.

2.2.2 Analysis of what has happened in the four years
since the 1988 9th WCCEE due to significant EQs. As
previously defined, significant EQs are those events
which either produce significant damage or have
magnitudes M ≥ 6.5. The main results obtained in this
analysis are given in Table 1. The loss of life during this
period (1988-1991) is illustrated in the plot of
Figure 2.

To summarize, this analysis has shown that the average number of deaths per year has increased, in
even one of the cases, the smallest magnitude, Mw 6.5
(surface wave), of all of the recorded EQs do not
exceed 6.8 and 7.7, respectively. The total number of
people living in houses destroyed by EQs during these last four years exceeded 1.7 million, i.e., an average of 425,000 per
year. A brief description of the main EQs that occurred
during this four-year period follows.

- The Armenian EQ of December 7, 1988 [Wallie et al.
(1989) and Bormer and Ambraseys (1988)]: The main
shock of this EQ, with a body wave magnitude of
Mw = 6.3 and corresponding to a surface wave magnitude
Mw = 6.8, occurred at 11:41 am local time near the
relatively new town of Spitak (population about 20,000),
which is located midway between the cities of Leninakan
(population about 290,000) and Kirovakan (population
about 170,000), located about 60 km east of Leninakan.
This main shock, with a focus (hypocenter) estimated at
15 km and with strong motion lasting nearly 30 seconds,
casted catastrophic damage, and was followed four
minutes later by an aftershock with Mw = 9.2, which

Table 1. Effects of significant EQs during 1988-1992

<table>
<thead>
<tr>
<th></th>
<th>No. OF SIGNIFICANT EQs</th>
<th>MAX. Mw</th>
<th>No. DEAD</th>
<th>No. HOMELESS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1988</td>
<td>60</td>
<td>6.8</td>
<td>7.5</td>
<td>27,000 to 32,000</td>
</tr>
<tr>
<td>1989</td>
<td>60</td>
<td>6.5</td>
<td>7.4</td>
<td>600</td>
</tr>
<tr>
<td>1990</td>
<td>73</td>
<td>6.7</td>
<td>7.7</td>
<td>42,000 to 52,000</td>
</tr>
<tr>
<td>1991</td>
<td>61</td>
<td>6.5</td>
<td>7.4</td>
<td>3,300</td>
</tr>
<tr>
<td>1992</td>
<td>26</td>
<td>7.4</td>
<td>522 to 840</td>
<td>50,000</td>
</tr>
</tbody>
</table>

*UP TO JULY 1992

Figure 2. Loss of life caused by EQs: 1988-1991
Figure 3. 1982 Armenia EQ. Photos illustrating collapse of modern precast RC frame buildings [EERI Slide Library]

caused significant additional damage. Although the exact number of people killed is not known, official reports placed it at about 28,000 (ranging from 25,000 to 30,000) and unofficial reports gave a death toll ranging from 45,000 to 60,000. It has been estimated that about 130,000 people were injured, of whom 18,000 required hospital care. The mortality among the injured was also high, especially among those who remained buried alive for long periods of time. The number of people left homeless was estimated at between 500,000 and 700,000. The Armenian EQ was one of the most lethal of the decade, but an even more important lesson from this $M_w=6.8$ EQ is that most of its losses resulted from the collapse of modern buildings (Figure 3).

- **The Manjil-Rudbar, Iran, EQ of June 21, 1990** [Astaneh (1990)]: The magnitudes of this EQ, which occurred in the northern part of Iran at 03:00 (local time), were $M_w=6.8$ and $M_s=7.6$. It has been estimated that more than 35,000 (up to 50,000) persons were killed and that about 400,000 persons became homeless. Hundreds of towns and villages were destroyed [Figure 4(a)], and more than 130,000 homes and commercial buildings were damaged [Figure 4(b)]. A very severe aftershock of Richter $M=6$, which occurred 12 hours after the main shock on June 21, contributed to the collapse of many structures that had been damaged during the main shock. The Manjil-Rudbar EQ also caused severe damage to transportation facilities and lifeline structures. Almost all of the damage appears to have been the result of poor seismic design, construction and maintenance.

- **The Luzon, Philippines EQ of July 16, 1990**: This EQ, with a magnitude of 7.7, affected a large area. More than 2,000 were killed and 3,500 were injured. About 22,000 buildings were destroyed, and the number of persons who were evacuated was estimated at 1,600,000. In Baguio City, more than 50 multi-story RC buildings collapsed or were severely damaged, including several hotels. The hotel shown in Figure 5(a) was designed using seismic code regulations that were practically identical to those of the UBC in the U.S.A. In the city of Dagupan, numerous buildings suffered significant damage owing to soil liquefaction [Figure 5(c)]. A large number of major roads were blocked by numerous landslides.

- **The Loma Prieta EQ of October 17, 1989**: A clear example of the increased seismic risk in California is given by analysis of what occurred during the Loma Prieta EQ. In spite of the fact that the number of injured (around 3,000) and particularly the number who lost their lives (about 65) were surprisingly low, it is clear from analyzing the seismological and engineering aspects of the Loma Prieta EQ that the effects of this EQ have been devastating, producing what has been
evaluated as one of the largest natural disasters in U.S. history. Estimates of economic losses due to physical damage alone (i.e., the costs of replacing physical facilities (structures) that were damaged) reach $8 billion. Further EQ consequences that can and will affect people’s lives are “functional and indirect damage.” Functional damage includes psychological stress and disruption of everyday routine caused by functional disorder of urban facilities; indirect damage denotes the loss of business opportunities. When these are added to the physical damage, total losses will exceed $10 billion.

The economic losses of the Loma Prieta disaster were of the same order as those of the great 1906 San Francisco EQ. This is somewhat surprising, because while the magnitude (M) of the 1906 San Francisco EQ was estimated at $M_{SF}=8.3$, the surface wave magnitude of the Loma Prieta EQ ($M_{LP}$) was estimated originally at 6.9 and then upgraded to 7.1. It is clear that the energy, $E_{LP}$, released by this EQ was significantly lower than the energy, $E_{SF}$, released by the San Francisco EQ. In fact,

$$\frac{E_{SF}}{E_{LP}} = 63 \quad (1)$$

Furthermore, because the epicenter was more than 90 km from Oakland and San Francisco (Figure 6), thisEQ cannot properly be considered a Bay Area EQ. Then why was the 1989 Loma Prieta disaster as great as the 1906 San Francisco disaster? Does the greater economic loss mean that our knowledge of how to design and construct EQ-resistant structures has not advanced since 1906? No, that is not the case; even though EQ Engineering is a relatively new branch of engineering research, advances in this field have already played a significant role in reducing seismic hazard through the improvement of the built environment, making possible the design and construction of EQ-resistant civil engineering structures (highway structures, dams, pipelines, critical facilities, high-rise buildings, etc.), and improving the seismic safety of non-engineered construction.

Rather, the answer can be found in the following circumstances. First, numerous studies have shown that the greatest current threat to life and safety arising from moderate to severe EQs occurring near urban areas is posed by existing hazardous structures. Many hazardous structures and facilities exist in the U.S., because many buildings were constructed when EQ Engineering was in its infancy. Furthermore, although EQ resistance requirements in building codes have become more stringent and improved significantly, even current codes are not infallible. Second, this problem of existing hazardous civil engineering structures has become markedly exacerbated by continuous uncontrolled

Figure 4. 1990 Manjil-Rudbar Iran EQ. Photos illustrating damage of buildings (courtesy of A.H. Astaneh)
Figure 5. 1990 Luzon EQ: photos illustrating collapsed and damaged buildings

(a) Collapsed modern hotel

(b) Modern hotel: collapse of 1st story

(c) Tilted corner building

(d) Tilted slender building

Figure 6. Map showing locations of the 1906 San Francisco and 1989 Loma Prieta EQs

(a) Immediately following the 1906 EQ

(b) San Francisco today

Figure 7. Views of San Francisco
Figure 8. Failure of one span of the upper deck of the San Francisco Bay Bridge [Astaneh et al., 1989]

(a) Part that survived the EQ  
(b) Part that collapsed

Figure 9. Cypress Street double-deck viaduct

Figure 10. Freeway structures that collapsed during the 1971 San Fernando EQ
population growth, increased urbanization, and the development of high-technology industries in our urban areas. The growth of San Francisco and the concomitant increase in EQ risk are illustrated in the photos of Figure 7. In 1906 there were very few medium-rise buildings. There were no bridges and elevated highways connecting San Francisco with the other cities of the Bay Area. This confirms the statement made previously that the disaster potential in California is now significantly higher than it was in 1906.

The Loma Prieta EQ might be called "the transportation and geotechnical engineers' EQ," because of: (1) the large number of damaged highways, roads and bridges, and the damage to several facilities at the Oakland airport and harbor; (2) the spectacular failures of one span of the Bay Bridge (Figure 8) and the Cypress Street double-decker viaduct (Figure 9), and the economic impact of these failures; (3) the many failures related to geotechnical effects; and (4) the wealth of strong motion records which permit study of these effects. Although the spectacular failures cannot be considered unexpected, because similar failures have occurred before (Figures 10-13), they do emphasize that either we very quickly forget the lessons learned in previous EQs or we do not take the warnings seriously. Although the collapse of freeway structures (particularly overpasses) taught the most important lesson learned from the effects of the 1971 San Fernando (or Sylmar) EQ (Figure 10), there is no doubt that the dramatic collapse of the Cypress Street double-deck viaduct (Figure 9) is the first observed collapse of this particular type of freeway structure. Because it was designed and constructed between 1951 and 1957, when
very little knowledge existed about EQ-resistant construction of this type of structure, its dramatic collapse clearly points out the need for conducting vulnerability assessments of this type of hazardous existing structure, and for immediate reliable upgrading of similar existing structures. It appears that the lessons learned from the 1971 EQ and the reminder given by the failure of the piers in one bent of a major freeway overpass during the 1988 Whittier Narrows EQ (Figure 11) were not taken seriously.

The collapse of the upper deck of the Bay Bridge (Figure 8) points out that it is necessary to conduct reliable analyses of the possible relative movement between two adjacent and different structures, in the case of long multi-span bridges subjected to moderate or severe EQGMs, as well as the necessity of learning how to upgrade such structures. Again, this is not a new lesson. Analysis of what happened in the 1964 Nigata EQ (Figure 12) and in the 1985 Chile EQ (Figure 13) again underscores the fact that we forget very quickly the lessons learned in previous EQs and do not take seriously their warnings of the urgent need to perform assessments of the vulnerability of similar important transportation facilities in our urban areas.

The dramatic collapse of several buildings in San Francisco’s Marina District during the Loma Prieta EQ does not teach a new lesson. The collapses were, in most cases, consequences of a combination of the following factors: liquefaction of the soil, inadequate foundations, “soft” first stories, and poor structure conditions (woods rotten or infected or both due to poor maintenance). The importance of the effects of liquefaction has been taught by many previous EQs. The occurrence of liquefaction in general is not a surprise. In the Nigata (Japan) and Alaska EQs of 1964 (see Figure 14), many buildings subsided, inclined, overturned, and in some cases were translated very large distances by landslides as a consequence of liquefaction of saturated sand. Comparison of the photos shown in Figure 15, taken in 1964 in Nigata, to the photo taken at the Oakland Airport in 1989 shows that the sand boils observed at the Oakland Airport are nothing new. Furthermore, there is evidence that in San Francisco during the Great San Francisco EQ of 1906, liquefaction occurred in the same areas in which it was observed in 1989. A significant and surprising feature of the 1989 EQ is that liquefaction occurred at large epicentral distances (100 km), and after very few seconds of strong motion (less than seven seconds in the Bay Area). It appears that if the strong motions had lasted a few seconds longer, the amount of liquefaction, and hence the amount of damage, would have increased dramatically. There are major areas throughout the San Francisco Bay Area with liquefiable sites unsuitable for EQRC on standard foundation types which remain at considerable risk in large EQs.

More than 105,000 homes and 320 apartment buildings were damaged by the 1989 Loma Prieta EQ. It is estimated that the number of people displaced from their homes exceeded 14,000. Sheltering those displaced by the EQ was a major problem, especially in Watsonville and other small communities in the epicentral region. This large number of houses damaged and people displaced clearly points out not only the need to improve non-engineered construction of dwellings through improved design practices and codes, but also the urgent need to develop effective and economical techniques for the seismic upgrading of existing hazardous dwellings.

2.2.3 The issue of EQ preparedness. It has been stated previously that “the poorer the preparedness, the greater the disaster.” A country with poor preparedness will suffer more than a country with good preparedness. Preparedness is necessary. A comprehensive program of reduction of any natural hazard should include attempts to prevent the event creating the hazard, i.e., prevention of the event. If this is not possible, the program should include the possibility of prediction of the event using a probabilistic approach (analysis), and prevention of the
disaster by preparedness through a comprehensive hazard reduction plan.

Because of this content, such a comprehensive program could be called the 4P program (Prevention, Probability, Prediction and Preparedness). Note that prevention usually implies prediction. In the case of EQs, prevention or control of the event may never be achieved, at least not in the near future, and prediction is uncertain in the near future and may never be achieved for certain faults. The only effective way to prevent an EQ disaster is to reduce the consequences of EQ effects through a comprehensive preparedness program; effectively implemented. The ideal solution of the EQ problem will be through prediction of the event and hazard reduction through preparedness. Prediction research should be continued, but should not interfere with efforts to solve the present and pressing problems through an adequate preparedness program.

To summarize: EQs are inevitable, but the fault rupture generating the EQ does not itself kill people or induce great economic losses. What causes most of the injuries and losses is the interaction of the EQGMs with the built environment. What is needed is to control seismic risk in our urban areas by controlling the built environment, which should be the main point of the EQ preparedness program.

2.3 Control of seismic risks

In order to learn how to control seismic risk, it is necessary to define it. According to the glossary of the EQ Engineering Research Institute’s Committee on Seismic Risk [1984], seismic risk is “the probability that social or economic consequences of EQs will equal or exceed specified values at a site, at various sites, or in an area, during a specified exposure time.” According to Dowrick [1987], seismic risk is an outcome of seismic hazards, as described in the following relationship, which is also illustrated in the flow chart of Figure 16.

\[
\text{Seismic Risk} = \frac{\text{Seismic Hazard}}{\text{Vulnerability} \times \text{Value}}
\]

A seismic hazard is any EQ-related physical phenomenon (e.g., ground-shaking, ground failure) that may produce adverse effects on human activities. As indicated in Figure 16, seismic hazards at any site or region are consequences of the interaction of the sources of potential EQ hazards (created by the local seismic activity) with the degree of vulnerability of the built environment. Built environment denotes the different facilities (engineered or non-engineered) such as buildings, transportation and communication systems, dams or lifelines in general, and equipment, that are located on a site or in an area. Vulnerability is the amount of damage induced by a given degree of hazard, expressed as a fraction of the value of the damaged item or facility. Therefore, to assess the degree of vulnerability of the built environment, it is necessary to assess the response (performance) of whole systems (i.e., soil, foundation, superstructure and nonstructural components and contents) of the facilities in the built environment.

From the above considerations, it is clear that to control seismic risk at any given site it is necessary to estimate seismic risk, which requires the following:

- First, estimation of the seismic activity at the site, which requires identification of all sources of EQGMs that could affect the built environment, i.e., that could induce damage. Once the sources of various potential seismic hazards created by the seismic activity have been identified, it is necessary to determine whether the EQs will be single or multi-events, and to estimate their moment magnitudes, recurrence periods, and the attenuation of the intensities of their EQGMs with distance.
- Second, prediction of whether the EQ faulting originating the EQGMs could induce any of the following potential seismic hazards at the site or the surrounding region: surface fault ruptures, tsunamis, seiches, landslides, floods and ground failure.
- Third, prediction of the time history of the six components of the EQGM at the site and at the foundation of each facility.
- Fourth, prediction of whether the predicted EQGMs can induce ground failure, i.e., liquefaction, settlement, subsidence, differential compaction, loss of bearing and shearing strength, lateral spreading, landsliding and/or lurching.
- Fifth, assessment for any given facility of the mechanical behavior (performance) of the whole facility system under the predicted six components of the EQGM at its foundation, estimating the degree of damage and losses, considering the possibility of fire.
flood, and other consequent or indirect sources of seismic hazards.

- Sixth, evaluation of the economic consequences of the losses and the socio-economic impact on the community. The costs and benefits of seismic upgrading of existing hazardous facilities should be estimated.

Analysis of Table 2, which summarizes the experts needed to perform the required assessments indicates that: (1) the estimation of the seismic activity should be conducted by geoscientists (geologists and seismologists); (2) the prediction of possible sources of potential EQ hazards should be conducted by geoscientists and geotechnical engineers; (3) the prediction of EQGMs at the site and at the foundation of the facility should be conducted by geoscientists and geotechnical engineers in consultation with structural engineers; (4) the prediction of the ground failures that can be induced by EQGMs at the site has to be done by geotechnical engineers; (5) the assessment of the mechanical behavior of the whole system of any given facility when subjected to the predicted EQGMs requires the collaboration of geotechnical, foundation, structural, construction and mechanical engineers, in consultation with architects and contractors; and (6) the evaluation of the economic consequences of the losses and the socio-economic impact on the community requires the collaboration of engineers, architects, contractors, socio-economists, government officials and politicians. From this analysis it is clear that reduction and control of seismic risk in any given urban area is a complex problem, requiring the integration of knowledge and the collaboration of experts from many disciplines.

Furthermore, the problem of seismic risk reduction will not be solved just by the acquisition of the required knowledge through research. Research must be accompanied by the necessary technological developments and the implementation of the knowledge and the developments in practice. What is needed is a translation of current engineering and architectural know-how into simplified options which can answer the socio-political and economic concerns. This will require not only a multi-disciplinary approach, but also a comprehensive educational program, not only for owners and future users but also for all of the different audiences that in one way or another are involved in the implementation of the seismic risk reduction measures. This education program should emphasize the importance of EQ disaster preparedness, including preparing for fires, control of panic, etc.

Until now, most of the emphasis has been on (1) trying to predict EQs based on probabilistic approaches, and (2) gaining knowledge of the mechanical behavior (performance) of different facilities. While these are necessary, prediction alone will not solve the problems. What is necessary is to improve the preparedness of the public against EQ disaster. There is an urgent need to coordinate the acquisition, processing, evaluating and synthesizing of the research results already available, and the knowledge gained through lessons learned in past EQs. This integrated knowledge must then be converted into action. There is a need for multi-disciplinary groups of researchers, practicing professionals, users, government officials, etc., who will develop and ensure the implementation of reliable and suitable policies and strategies which will help to reduce and control seismic risks to acceptable levels, which is what is needed.

2.4 Concluding remarks regarding the control of seismic risks, the need for EQ preparedness, and future directions toward a solution

During the 1984 8th WCECE Press [1984] stated that, "Good government management is the key factor in preparedness, and therefore government performance is a major controllable factor influencing the impact of a disaster." There is no doubt that if government does not assume its proper role of hazard management after being provided with the required assessments by competent professionals, seismic risk reduction to an acceptable level will not occur. With all of the possibilities for reducing EQ hazards by controlling the built environment (through reliable assessment of risks, seismic codes and construction standards, land use, and criteria for identification of existing hazardous facilities and upgrading them), one wonders why we have not found proper solutions to the following issues:

- Why have we not progressed very much in the reduction of seismic risks in our urban and rural areas?
- Why are there not adequate preparedness programs in most countries?

While the lack of preparedness against EQ disasters can be justified in countries that are very poor, it is difficult to understand the short-sightedness of some industrialized countries. There are many parts of the world that are particularly prone to EQs but have not had the advantages of risk assessment for their regions and the development and/or implementation of EQ preparedness programs. The importance of the need for developing and implementing comprehensive EQ preparedness programs is clearly demonstrated by what has happened in the past in the following cities:

- San Juan, Argentina. In 1944, this city was destroyed by an EQ. A very simple seismic code was formulated, with very specific recommendations regarding the use of masonry and concrete in the construction of buildings. Through the strict enforcement of this code, the city was rebuilt and survived, without any significant damage and without loss of life, an EQ in 1977 with characteristics similar to that of the 1944 EQ.
- El Asnam, Algeria. This city was practically destroyed during an EQ in 1954. A new, modern seismic code was developed, but was not properly enforced or implemented in the rebuilding of the city, particularly in the large number of buildings built in the 1970s. During the 1980 EQ, most of these buildings collapsed.
- Erzincan, Turkey. This city was destroyed by an EQ of magnitude M=8.0 in 1939, and was warned again by a moderate EQ of M=5.6 in 1983 which damaged several buildings. In 1992, the city was severely damaged by an EQ of M=6.6. More than 2,100 buildings collapsed or were heavily damaged. This last EQ also affected about 70 villages and towns, in most of which 40% of the houses collapsed or were damaged beyond repair. The total number of people left homeless was estimated at about 120,000. Most of the buildings that failed in the city were modern buildings of four or more stories (Figure 17) designed and, particularly, constructed without compliance with seismic code regulations. The history of the effects of EQs in Erzincan demonstrates again that either we very quickly forget the lessons learned, or we do not take their warnings seriously.

It is clear from the above examples and from the
lessons learned in many other recent EQs that the potentially destructive EQGMs will not wait for our knowledge to be transferred to the practitioners and government officials in charge of EQ hazards management by osmosis from the R&D publications on their shelves. There is an urgent need to educate not only practitioners involved in EQ Engineering (particularly EQRD and EQRD), but also government officials, politicians, and the public in general.

To summarize: the main issue confronting all of us interested in EQ Engineering is the need to control the seismic risks in our urban and rural areas. The solution is controlling the vulnerability of the built environment, because this allows us to control the potential sources of EQ hazards, which are consequences of the interaction of seismic activity (which we cannot control) with the vulnerability of the built environment.

3. ROLE AND IMPORTANCE OF EQ ENGINEERING IN THE OVERALL PROBLEM OF CONTROLLING SEISMIC RISK IN URBAN AND RURAL AREAS

3.1 Role and Importance of EQ Engineering

3.1.1 Definition and objectives of EQ Engineering. As stated previously, EQ engineering is the branch of engineering that encompasses the practical efforts to avoid EQ hazards. It is a relatively new field of engineering: in the U.S. it is not more than 65 years old. Even so, EQ engineering has already played a significant role in mitigating seismic hazards around the world. Significant advances in analysis of the seismic response of mathematical models of structures and even in the understanding of real structures have been made. Nevertheless, there has not been a corresponding improvement in the reliability with which new structures and facilities are designed, constructed and maintained, nor in how existing structures are seismically upgraded to resist effectively the seismic hazards to which they may be exposed in their service lives. Therefore, it is not surprising that, as mentioned earlier, most human injury and economic losses from moderate to severe EQGMs are caused by the failure of engineered facilities. One main reason for this is that the state of the practice in EQ Engineering, and particularly in EQRD, as reflected by present seismic codes, is based on zonation maps which do not reliably depict the real seismic hazards of the sites on which facilities are built. The slow rate of improvement in EQ hazard reduction has been a consequence of the tardy introduction of advances in zonation and microzonation into EQ engineering and particularly into EQRD codes [Bertero, 1991 (Seismic Zonation)].

As pointed out above, the evaluation of the EQ hazards to which a facility may be subjected is a complex task, requiring not only probabilistic consideration of the EQ occurrence and the physical effects of the source, propagation path, and local site geology, but also the interaction of the ground-shaking with the whole soil-foundation-superstructure-and-nonstructural components and contents system of the facility to be designed. Unfortunately, most of the research on these different areas has been concerned with isolated areas of each specialized discipline. There have not been serious attempts to integrate the knowledge and the requirements of the various disciplines involved in the general...
problem of EQ hazard reduction. The engineers are perhaps not asking the geoscientists the right questions, and the geoscientists are perhaps more interested in trying to predict EQs than in the problems involved in the EQRD of engineered facilities. These attitude have to be changed.

3.1.2 Role and importance of EQRD of structures.
From previous discussion, it is clear that EQRD of structures is at present the key element in the problem of EQ hazard reduction. As has been mentioned, one of the most effective ways to mitigate the destructive effects of EQs is to improve and develop more reliable methods than are now available for designing, constructing and maintaining and monitoring new structures and seismically upgrading existing hazardous facilities. In order to find out what information is necessary for the realization of such improvements, it is convenient to analyze the main issues. These can be expressed as questions about what went wrong in the past (why are there so many hazardous facilities?), what is happening at present (where do we stand right now?) and where should we go (what are the directions for short-term and long-term solutions?).

Because the design and construction of most EQ-resistant facilities (particularly buildings) in practice generally follow seismic code provisions, it is convenient to examine these provisions briefly and to scrutinize what has been done and what should be done to improve the present state of the practice. Before examining present seismic codes, it is convenient to review briefly the problems involved in EQRD and EQRC, and the general philosophy of EQRD.

3.2 Problems involved in EQRD and EQRC.
The general aspects and problems involved in EQRD and EQRC for buildings (on which the following discussion will concentrate) have been discussed by Bertero [1982], and from analysis of these general aspects it is clear that seismic codes should regulate:
(1) Selection of building sites, siting restrictions, land use, and building site suitability analysis;
(2) establishment of design EQs, EQRD criteria, and design methodology;
(3) estimation and design guidelines regarding proper selection of building configuration, foundation, structural layout, structural system, structural material and nonstructural components;
(4) estimation of demands on a structure and its contents at the different levels of design EQs;
(5) estimation of the supplied capacities to a structure; and
(6) analysis of the performance of a designed structure under different established levels of design EQs. This should be done visualizing the mechanical characteristics that the real constructed facility will have.

(7) Construction (supervision), use and maintenance of the constructed facility.
As will be discussed in more detail later, review of the results conducted on the importance and effects of the general aspects of EQRD of structures indicates that the principal issues that remain to be resolved for the improvement of such design are related to the following three basic elements: EQ input, demands on the structure, and supplied capacities to the structure. After a brief review of how seismic codes in the U.S. have been developed and have attempted to resolve these three issues, this lecture will focus on the first of these, the EQ input element, which involves the following interrelated issues: Design EQs, design criteria, and selection of design methodology. The importance of proper establishment of the design EQs is reflected by the need to know against what we have to design the structure.

Design criteria should reflect in a transparent way the general philosophy of EQRD, which has been well established and is accepted world-wide. However, as will be discussed below, current code design methodologies in the U.S. fall short of realizing the goals and objectives of this philosophy [Bertero and Bresler (1977), Bertero (1986)].
The problems encountered in EQRD are complex, and therefore in general their solutions are also complex. In order to keep code design procedure as simple as possible, as it should be, it is necessary to specify very severe and restrictive regulations regarding the siting of facilities and the selection of their configurations, structural layouts, structural systems, structural materials and nonstructural components. This is the best way to avoid creating complex problems (soil-structure interaction, foundation movements, P-D effects, torsional effects, etc.). When these restrictive regulations are not followed, the code needs to specify that simple code procedure should be complemented with dynamic linear and nonlinear analyses and design procedures, which should be subjected to peer review.

3.2.1 General philosophy of EQRD. What can be considered the general philosophy of EQRD of buildings sheltering other than essential and hazardous facilities was introduced in the U.S. for the first time in the Commentary of the 1967 edition of the Structural Engineers’ Association of California (SEAOC) Blue Book [1967], and it has changed very little since then. Essentially, this EQRD philosophy states that the design should accomplish the following objectives:
1. Prevent nonstructural damage in minor EQ ground shakings, which may occur frequently during the service life of the structure.
2. Prevent structural damage and minimize nonstructural damage during moderate EQ ground shakings, which may occasionally occur.
3. Avoid collapse or serious damage during severe EQ ground shakings, which may rarely occur.

3.2.2 Ideal philosophy of EQRD. Recognizing first that the acceleration and deformations that can be developed during the responses of building systems to severe and even to moderate EQGMs are very high, and secondly that there are many uncertainties in the estimation of demands and supplies, the ideal philosophy should attempt to realize all of the objectives of the above general philosophy by providing all the needed stiffness, strength and energy dissipation capacity that can be accomplished with the minimum possible extra cost in initial construction and/or the slightest possible sacrifice in the architectural features that would be required for the design of the building for just gravity loads.
The above general philosophy is in complete accord with the concept of comprehensive design. However, current code design methodologies, at least in the U.S.,
fall short of realizing the goals and objectives of this philosophy.

Although in the commentary of the 1988 SEAOC Blue Book recommendations it is stated that structures designed in conformity with these recommendations should, in general, be able to accomplish the objectives of the above general philosophy, they are primarily intended to safeguard against major failure and loss of life, not to limit damage, maintain functions, or provide for easy repair. In few words, current code design methodology is based on a one-level design EQ. Moreover, the SEAOC commentary states that, "the protection of life is reasonably provided but not with complete assurance." To summarize, the primary goal of the U.S. seismic provisions is to protect life. The secondary goal is to reduce (not eliminate) property damage. The questions that need to be answered are:

First, does the application of current seismic code provisions accomplish the above goals?; and second, are these goals sufficient? Before attempting to answer these questions, it is convenient to review the philosophy of building codes, particularly seismic codes, and to review the history and development of these codes.

3.2.3 U.S. code philosophy. Building codes are primarily technical legal requirements, adopted by government agencies, specifying minimum standards for the design, manufacture, installation and use of building materials and components. Although the primary function of a building code is to provide minimum standards to assure public safety, it usually has other objectives as well. For example, the intention of the 1991 UBC is clearly stated in its Section 1-02:

The purpose of this code is to provide minimum standards to safeguard life or limb, health, properties, and public welfare by regulating and controlling the design, construction, quality of materials, use and occupancy, location and maintenance of all building and structures within this jurisdiction and certain equipment specifically regulated therein.

In view of the above code purpose, it is not surprising that SEAOC has established a seismic code philosophy that is in accordance with the above purpose of building codes. Thus, the basic philosophy of the SEAOC seismic code, as well as most of the other seismic codes, has been to protect the public in and about buildings from loss of life and serious injury during major EQs. However, some owner-sponsored codes have gone further than this. For example, in 1975 the Titles 17 and 21 of the California Administrative Code related to the design and construction of hospitals and public school buildings includes, as its added purpose, the protection of property. At present, Title 24 of the California Administrative Code regarding hospitals has the additional purpose that hospitals remain operational after an EQ.

3.2.4 History of seismic design codes. The 1980 edition of the SEAOC Blue Book describes the history of EQ codes in California. Table 3 summarizes the history of seismic design codes and their provisions in the U.S. [Committee on EQ Engineering Research, 1982].

The first EQ design requirements appeared in the 1927 edition of the Uniform Building Code. Although these provisions were not put into effect in any city, they required all buildings over twenty feet in height to be designed for a lateral force applied at each floor level and at the roof level generally parallel to the two main axes of the structure. The force required was a percentage of the total dead and live loads, with the exception of buildings with a live load not over 50 pounds per square foot, for which only a percentage of the dead load was required to be used. Structures on soils with a bearing value of two or more tons per square foot were to be designed for 7.5% of their vertical loads, and those with lesser soil bearing value and those on piles were to be designed for 10% of their vertical loads.

When seismic requirements first appeared in building codes and were put into effect, practically nothing was known about EQ Engineering. The 1933 L.A. Building Code, for example, merely stated that a building should

<table>
<thead>
<tr>
<th>Date</th>
<th>Code or Provisions</th>
</tr>
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<tbody>
<tr>
<td>Post-1906</td>
<td>San Francisco rebuilt to 30 psf wind</td>
</tr>
<tr>
<td>1927</td>
<td>First seismic design appendix in Uniform Building Code: V= CW (C=0.075 to 0.10)</td>
</tr>
<tr>
<td>1933</td>
<td>Los Angeles City Code: V= CW, (C=0.08) First enforced seismic code.</td>
</tr>
<tr>
<td>1943</td>
<td>Los Angeles City Code: V= CW, [C=60/(N+4.5)] N greater than 13 stories.</td>
</tr>
<tr>
<td>1952</td>
<td>ASCE-SEAONC: (C=K_T), (K_T=0.015-0.025)</td>
</tr>
<tr>
<td>1959</td>
<td>SEAOC: V=KC_W, [C=0.05/(T)^(1/3)]</td>
</tr>
<tr>
<td>1974</td>
<td>SEAOC: V=ZIKCSSW</td>
</tr>
<tr>
<td>1976</td>
<td>UBC: V=ZIKCSSW</td>
</tr>
<tr>
<td>1977</td>
<td>ATC-3 Tentative Recommendations: V=Z_W, C=1.2 A_s S/RT &lt;= 2.5 A_r/R</td>
</tr>
<tr>
<td>1988</td>
<td>SEAOC and UBC: V=ZIC W/Rw, C=1.25 S/TA^2 ≤ 2.75, C/R_w ≥ 0.075</td>
</tr>
</tbody>
</table>

NOTE: W=weight of building; V=base shear; T=period of vibration; N=number of stories; Z,K,ZI and S=numerical coefficients ( C was originally a seismic design coefficient but in codes later that 1943 a numerical coefficient dependent on T; Z=factor dependent on the zone in a seismic risk map; I=occupancy importance factor; and S=site-structural response or soil-profile coefficient); C_s=seismic coefficient; A_s=effective peak-velocity acceleration; R= response modification factor; and R_w=numerical coefficient (called system quality factor).
be designed to withstand a steady horizontal thrust equal to 8% of its weight, in effect treating EQ forces as wind pressures. In recent years, understanding of EQ Engineering problems and EQRD has undergone remarkable development. In the U.S. this was made possible largely by research after World War II on military protective structures, and after 1960 by EQ Engineering research programs conducted in several countries.

Building codes to which ordinary buildings are designed have also developed impressively, so that they are now much better suited to guide realistic design against EQ forces. Clearly, present U.S. methods of EQRD are an outstanding improvement over methods available 20 or even 10 years ago, particularly in regard to sizing and detailing of superstructures of ordinary buildings. To elaborate on this, it is convenient to classify seismic code provisions into the following two main groups.

1. **EQ-resistant criteria.** This group covers the basis for design and specifications of minimum lateral forces and related effects (estimation of seismic demands).

2. **Material code specifications.** This group regulates sizing and detailing of the structure.

In the last two decades there have been tremendous improvements in the code specifications for the sizing and detailing of structural members and their connections and supports. Figure 18 illustrates the changes in the spacing of ties in the EQRD of RC columns. Although the importance of providing the structure with large ductility was already recognized in the 1959 SEAOE-recommended requirements, special provisions regarding the EQRD of RC structures first appeared in the 1971 edition of the ACI code. Because the amounts of detailing and transverse reinforcement for achieving high ductility demands depart somewhat from the requirements of the ordinary practice in RC design and construction, the cost is higher for EQRC. This higher cost has caused some concern and the complaint that there may have been too much emphasis on creating ductility for ductility's sake (Dowrick 1987). This has also raised the following valid question: "How do we design less ductile structures which are sufficiently reliable against EQs?"

Regarding these complaints and questions, the lecturer believes that ductility requirements should not be relaxed in EQRD, at least until the results of new and reliable research and developments become available to justify such relaxation. These stringent requirements for sizing, and particularly for detailing, have been the blessing of the current code requirements for EQRD. The reasons for this are that there are many uncertainties involved in the estimation of the demands and supplies for EQRD procedure. As discussed below, the present code specifications for estimating seismic lateral forces and their effects are far from reliable. It is for this reason that the lecturer believes that stringent sizing and detailing requirements, rather than complex numerical analyses conducted to comply with code formulae for estimating demands, have permitted many buildings to survive recent moderate-to-severe EQGMs.

3.2.5 **U.S. code EQ-resistant criteria: estimation of demands.** The several sources of uncertainty in the estimation of demands can be grouped into two categories: 1) specified seismic forces, and 2) methods used to estimate responses to these seismic forces.

**Figure 18. Illustration of the changes in RC lateral reinforcement requirements**
1. Estimation of seismic forces. For regular buildings, the lateral seismic forces can be derived as follows.
(a) Base Shear:

\[ V = C_s W = \frac{C_{sp} W}{R} \]  

where \( V \) is base shear, \( C_s \) is defined as the design seismic coefficient, \( W \) is the weight of the reactive mass (i.e., the mass that can induce inertial forces), \( C_{sp} \) is the seismic coefficient equivalent to a linear elastic response spectral acceleration, \( S_w \) (\( C_{sp} = C_s R = S_w / g \)), and \( R \) is the reduction factor.
(b) Distribution of base shear over the height of the structure:

\[ V = F_1 + \sum_{i=4}^{n} F_i \]  

where \( F_i \) is concentrated force at the top and represents the effects of higher modes (whiplash effect) and:

\[ F_i = \frac{(V-F)w_i h_i}{\sum_{i=4}^{n} w_i h_i} \]  

is the force at level \( i \) (usually the floor level), \( w_i \) is the portion of \( W \) located at or assigned to level \( i \), and \( h_i \) is the height above the base to level \( i \).

2. Estimation of structural response to seismic forces. Structural response can be estimated using linear elastic analyses, either directly using the above statically equivalent lateral forces (Eqs. 4 and 5), or multiplying them by load factors, depending on whether the design will use allowable (service) stress or strength method.

The uncertainties involved in the estimation of base shear and its distribution over the height of the structure, as well as the reliability of the procedures and values specified by present U.S. seismic codes, have been discussed in detail by Bertero, V.V. [1982 and 1986].

A review of the history of how the values for the base shear resistance (Table 3) have been computed clearly shows that the equation recommended for its evaluation has become more and more sophisticated and requires more and more empirical numerical coefficients. However, what is really surprising is that the code requirement for base shear resistance remains practically the same as the first seismic code in 1927, and has even been reduced, as is shown by comparing Tables 3 and 4. This is surprising, because the building technology of the 1930's was quite different from the present one, and although it resulted in buildings with smaller ductility, these buildings usually had higher overstrength.

As indicated in Tables 3 and 4, SEAOC introduced significant changes into their code recommendations in 1988 by adopting some of the 1977 ATC-3 recommendations. The new SEAOC recommendations have been adopted in the 1988 UBC. Although these recent codes and recommendations recognize the severity of seismic hazard for different seismic zones in the U.S. and incorporate modern seismic design philosophies and approaches, they continue to place too much emphasis on designing for a yielding strength capacity which is the same as or even less than that which resulted from applying the provisions of the first U.S. seismic code regulations in 1927.

The lecturer has recently analyzed present trends in EQRD and EQRC of buildings in the U.S. and has made the following observations.

1. Recent code recommendations recognize the probable occurrence of very severe EQGMs at a given site in a region of high seismicity (high intensity and long duration of EQGMs). In spite of this and the significant changes in construction technology, there has been very little change in the overall seismic coefficient for which buildings must be designed.
2. The code continues to place too much emphasis on strength design based on fictitious seismic forces and linear elastic analyses of their effects.
3. Because of economic pressures, designers try hard to comply with just the code minimum requirements for strength.
4. The development and use of computer programs based on optimal design of members of a structure will lead to final designs with very little overstrength with respect to the code-required minimum strength. This problem is exacerbated by

| Table 4. Comparison between: |

(1) Expressions for effective seismic coefficient, \( C_s = V/W \), specified by the 1985 UBC, ATC-3 and the 1988 SEAOC recommendations.

<table>
<thead>
<tr>
<th></th>
<th>UBC</th>
<th>ATC</th>
<th>SEAOC</th>
</tr>
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<tbody>
<tr>
<td>Z1KCS</td>
<td>( \frac{1.2 , A_v , S}{R^{2/3}} )</td>
<td>( \frac{\tau}{ZIC} )</td>
<td>( \frac{\tau}{R^{2/3}} )</td>
</tr>
<tr>
<td>ZIK</td>
<td>( \frac{1.25 , A_v , S}{R^{2/3}} )</td>
<td>( \frac{\tau}{155ZIC} )</td>
<td>( \frac{\tau}{R^{2/3}} )</td>
</tr>
</tbody>
</table>

(2) The values of \( C_s \) for ductile moment-resisting space frames, DMRSF, in regions of high seismic risk.

<table>
<thead>
<tr>
<th></th>
<th>UBC</th>
<th>ATC</th>
<th>SEAOC</th>
</tr>
</thead>
<tbody>
<tr>
<td>( (K=0.7) )</td>
<td>( (R=8) )</td>
<td>( (R_p=12) )</td>
<td>( (R=12) )</td>
</tr>
<tr>
<td>( 0.61 , S )</td>
<td>( 0.060 , S )</td>
<td>( 0.042 , IS )</td>
<td>( 0.055 , S )</td>
</tr>
<tr>
<td>( 12 , \sqrt{\tau} )</td>
<td>( \tau^{2/3} )</td>
<td>( \tau^{2/3} )</td>
<td>( \tau^{2/3} )</td>
</tr>
<tr>
<td>For ( I = 1 ) and ( T = 1 , sec. )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( V )</td>
<td>( 0.045 , S )</td>
<td>( 0.060 , S )</td>
<td>( 0.042 , S )</td>
</tr>
<tr>
<td>( Vu )</td>
<td>( 0.063 , S )</td>
<td>( 0.060 , S )</td>
<td>( 0.059 , S )</td>
</tr>
</tbody>
</table>
the use of taller and slenderer buildings.

(5) The use of very light and weak nonstructural elements (walls, partitions, claddings, etc.) which, furthermore, are built such that their performance will not interfere with the deformation of the structure, results in buildings whose strength and stiffness are just those of the bare structural system.

All of the above developments and trends result in the construction of buildings with very little overstrength beyond the minimum code-required strength. There is an urgent need for calibration of the real strength and stiffness of buildings that have been designed and constructed according to present codes. There can be no improvement in the EQRD of new buildings, in seismic performance evaluation of existing buildings, or in vulnerability assessment and upgrading of hazardous buildings, if there is no improvement in predicting stiffness, strength, and energy absorption and dissipation capacities of real building systems (soil-foundation-superstructure and nonstructural components).

In recent years there has been an increasing amount of research on design concepts based on probabilistic approaches. This activity has resulted in a re-examination of past data, a close analysis of design concepts, and a formulation of design provisions to make the latter more logical for practitioners. Much remains to be done to apply such research to assessments of seismic risk, particularly in areas of low seismic activity, and to adapt such research to practical design and construction.

3.2.6 Comparison of current seismic code provisions.

Analysis of comparisons of U.S. codes with the present seismic codes of Europe, Chile, Japan, Mexico D.F. and New Zealand makes it clear that there are some significant discrepancies among the seismic provisions of the current codes [Bertero et al. (1991)]. It is believed that this is a consequence of the fact that seismic codes, of necessity, are generalized oversimplifications of the very complex real EQRD problem. Modern building codes, which try to reflect great advances in knowledge and understanding in a very simple way, are not transparent about the expected level of performance of the whole building system (soil-foundation-superstructure and nonstructural components). Expected level of performance has become an implicit, rather than an explicit, part of the codes through a series of empirical factors and detailing requirements which obscure the true nature of the EQRD problem: building performance.

Current seismic codes lack transparency in their provisions regarding: the reliable establishment of critical EQQMs for the desired performance of the whole building system at the different limit states through which it can go during its service life; and also the real expected response of the resulting designed and constructed building system to real critical EQQMs (not just to code-specified motions). Although there have not been enough moderate and severe EQQMs in urban areas to permit analyses and judgements of the performance of building systems designed according to current seismic codes, the observed behavior of some modern buildings in recent EQs, particularly in Mexico City during the 1985 Michoacan EQ and in the Bay Area during the 1989 Loma Prieta EQ, and also in the 1990 Luzon, Philippines and the 1992 Erzincan, Turkey EQs, indicates the need for improvement of the current U.S. EQRD code approach.

The slow rate of improvement in our seismic codes is not surprising because as pointed out by Housner (1984), the code is a legal document that specifies minimum level of design that must be attained by facilities and because it has a large socio-economic impact, substantial changes in code requirements are made slowly and cautiously. In addition, because building codes affect so many agencies, groups, individuals, etc., there is a great inertia against change, and therefore developments in building codes tend to lag behind developments in research, as will be discussed in more detail below. Unfortunately, needed changes in the code are usually deferred until the occurrence of a destructive EQ.

4 MAIN ISSUES REGARDING PRESENT EQRD SEISMIC CODE PROCEDURES

4.1 Introductory remarks.

From the analysis and discussion presented in section 3.2, it can be concluded that current codes in the U.S. and most other countries have as primary goals the protection of human life, and that the second goal is to reduce (not eliminate) property damage. However, as these codes are based on just one-level design EQ, the main issues that remain to be answered regarding present seismic code regulations and their implementation are:

(1) Does the implementation of current seismic code provisions accomplish the above primary and secondary goals of these codes?
(2) Are these goals sufficient?

As previously discussed, structures designed in conformance with present seismic code regulations cannot guarantee the accomplishment of the above main goals, and particularly the objectives of EQRD philosophy. In order to be able to accomplish such goals and objectives, seismic codes should define clearly: the damages to the entire facility system that can result from the sources of potential seismic hazards originating by all of the possible EQs affecting the region of the structure site; and then what constitutes acceptable damage, i.e., what constitutes unacceptable risks. As already discussed, facility damage may result from different seismic effects, which may be classified into two main groups: direct effects and indirect effects. The direct effects are: (1) ground failures due to fault ruptures or to the effects of seismic waves; (2) vibrations transmitted from the ground to the structure; and (3) seismic sea waves (tsunamis) and tsunami-like disturbances in lakes (seiches). The indirect, or consequential, effects result from other EQ phenomena, such as fires and floods caused by dam failures and landslides. Thus, the first and perhaps the main step in a comprehensive design approach should be to conduct a reliable assessment of the above seismic hazards and to analyze the suitability of the selected building site. This requires reliable seismic microzonation of urban areas. Regarding this matter, the commentary of the SEAOC Blue Book [1988] states:

It is to be understood that the damage due to the earth slides such as those that occurred in Anchorage, Alaska or due to earth consolidation such as occurred in Nagata, Japan, would not be
prevented by conformance with the SEAOC Code. The SEAOC Code has been prepared to provide minimum required resistance to typical EQ ground shaking, without slides, subsidence, or faulting in the immediate vicinity of the structure.

The seismic effect that usually concerns the structural engineer and is accounted for in EQRD provisions of building codes is the response (vibration) of a building to ground shaking that might occur at its foundation. In most cases, damage due to other effects exceeds that due to the vibration of the building. Nonetheless, procedures for gauging the probability of such hazards and coping with them are normally outside the scope of the structural engineering discipline, and so are not included in the codes. Generally, the only way to avoid damage from most of these effects is by changing the building site, a decision which rests with government officials. In spite of this, the engineer should be aware of the different seismic hazards and should advise the client of potential dangers involved in constructing buildings at certain sites.

As stated in section 3.2.2, according to the commentary of the SEAOC recommendations, structures designed in conformance with these recommendations, which are based on a one-level design EQ approach, should, in general, be able to attain the three objectives of the general philosophy. This is not only questionable, but indeed the code seismic design provisions have been developed to satisfy only the criteria involved in Objective 3 of the above philosophy. Apparently, this has been done under the assumption that if Objective 3 were met, Objectives 1 and 2 would automatically be satisfied. Recent studies show that this is not the case.

Uang and Bertero (1991) have shown that the UBC- (or SEAOC) specified seismic design procedure cannot adequately control the general demands that can be imposed by service EQGMs. Furthermore, the lecturer believes that it will be very difficult to satisfy the criteria for all three objectives of seismic design philosophy by keeping the present building code design methodology, which requires only one level of design EQ (life-safety level). It is believed that the time has arrived to move from the current code one-level design EQ methodology to a code design methodology based on at least two distinct levels of design EQs: the service-level (functional adequacy) and the life-safety level EQs.

It should be noted that the above proposed two-level design EQ methodology does not necessarily mean that the preliminary design of any EQ-resistant building system should have to be carried out considering simultaneously the specified two-level design EQs. Although this would be highly desirable, the above proposed two-level design methodology really means that the EQRD process would have to be conducted in two phases. In the first phase, the preliminary design of the building system would conform to what is considered to be the critical or controlling level design EQ. This would require thorough comparison and analysis of the two specified level design EQs. In the second phase, the preliminary design of the building would be analyzed and detailed to ensure compliance with the dual criteria involved in the two levels of design EQ, i.e., satisfactory performance at both the service level and the life-safety level EQGMs.

The idea of using two levels of design EQ is not new. In the U.S., its application and introduction into seismic codes were discussed in the 1960's [Degenkolb (1972) and Bertero (1975)]. A survey of the seismic design codes of other nations [Bertero et al. (1991)] and the World List of EQ-Resistant Regulations (1988) reveals that the 1981 Japanese Building Standard Law (BSL) explicitly specifies a two-level design EQ: moderate EQGMs, which would occur several times during the service life of the building with almost no damage, and severe EQGMs, which would occur less than once during the use of the building and would not cause collapse or harm to human lives. While buildings not higher than 31 m (102 ft) can be designed under just moderate EQGMs, buildings higher than 31 m must be designed for the two-level EQGMs.

4.2 Principal issues in the improvement of EQRC.

As discussed in section 2.1, it is well-recognized that most human injuries and economic losses due to moderate or severe EQGMs are caused by the failures of facilities, particularly buildings, many of which were presumed to have been engineered, i.e., designed and constructed, to provide protection against natural hazards and comfort to human beings. This has been dramatically confirmed during recent EQs around the world (the 1985 Chile, the 1985 Mexico, the 1986 San Salvador, the 1987 Whittier Narrows, the 1989 Loma Prieta, the 1990 Iran, the 1990 Philippines, and the 1992 Erzincan, Turkey, EQs. Therefore, one of the most effective ways to mitigate the destructive effects of EQs is to improve existing methods or develop new and better methods of designing, constructing and maintaining new structures and of repairing and upgrading (retrofitting) existing seismically hazardous facilities.

The seismic response of any facility (structure), and therefore the degree of damage that it will suffer, depends on the whole building system (soil-foundation-superstructure and nonstructural components and contents) when the EQ occurs: i.e., response depends not only on how the building has been designed and constructed, but also on how it has been maintained up to the time that the EQ strikes. Thus, the principal issues that need to be considered in order to improve EQRC, and therefore to reduce the seismic risks in our urban areas, are the ones grouped into the following categories.

- Improvement of the EQRD of the whole facility system (soil, foundation, superstructure, and nonstructural components and contents).
- Improvement of the construction of the foundation, the superstructure and the nonstructural components.
- Improvement of the maintenance (monitoring and preservation) of the whole system.

In what follows, only the main issues regarding the improvement of the EQRD of the whole system of any given facility (particularly the building system) will be discussed in detail. The main reason for this is that while issues concerning field construction and maintenance vary not only from one country to the next but even from one region to another in a given country (because they reflect the building technology available in each region), the basic problems created by EQGMs in the EQRD of our facilities are the same. However, before identifying and discussing the main issues that need to be resolved for an improvement in the EQRD of whole
facility systems, the importance of proper field construction (EQRC) and maintenance must be briefly discussed.

4.3 Importance of proper EQRC and monitoring and maintenance of facilities.

While a sound EQRD of any given structure is necessary, it is not sufficient to ensure a satisfactorily EQ-resistant structure. As discussed above, the seismic response, and therefore the performance of any facility under the effects of EQGMs, depends on how the whole system of this facility has been constructed, how its function has been monitored, and how it has been maintained. A design can only be effective if the model used to engineer the design can be and is constructed and maintained [Bertero (1975), (1982) and (1986)]. Although the importance of construction and maintenance in the seismic performance of structures has been recognized, insufficient effort has been made to improve them (e.g., through improving supervision and inspection). Design and construction are intrinsically interrelated. If good workmanship is to be achieved, the detailing of members and their supports must be simple. Field inspection has revealed that a great deal of damage and failure is due to poor quality control of structural materials and/or poor workmanship - problems that would not have arisen if the building had been properly inspected during construction. The photos in Figure 19 illustrate the severe damage that occurred in one half of a building during the 1985 Chile EQ (the two halves, divided from each other by an expansion joint, were built by two different contractors), while no damage occurred in the other half. The concrete in the severely damaged half had a compressive strength of only 100

Overview of a 4-story building: one half of the building was damaged

Figure 19. Illustration of damage due to poor quality control of material [Earthquake Spectra, EERI February 1986]

Poorly-connected precast elements

Figure 20. Illustration of damage to a precast concrete frame building due to poor detailing and workmanship [EERI Slide Library]
kg/cm². One of the main factors in the failures of several buildings during the 1985 Mexico, 1986 San Salvador, 1990 Philippines and 1992 Erzincan EQs was the poor quality of concrete and the poor workmanship in the detailing and placing of the reinforcement. Poor workmanship in the connections was the main reason for the failure of many industrialized (prefabricated) buildings during the 1988 Armenia EQ (Figure 20). In many other cases, damage may be attributed to improper monitoring of the function of the building, as in the case of several buildings during the 1985 Mexico EQ (Figure 21). Some of these buildings were built for offices or as residences, but were later used to shelter lightweight industries. Similarly, many observed failures of buildings have been due to improper maintenance during their service lives. Inappropriate alteration, repair, and retrofitting of the structure and nonstructural components can lead to severe damage during major EQ shaking. Strict enforcement of seismic codes should be included in the EQ preparedness programs, and enforcement of seismic code regulations should not be lax under any circumstances.

4.4 Principal issues in the improvement of EQRD of structures

4.4.1 Differences between design and analysis. In trying to identify the problems whose solutions need improvement in order to achieve an efficient EQRD of a facility, it is necessary to recognize clearly the differences between analysis and design. While usually (particularly for the design of engineered facilities) in order to achieve an efficient final design it is necessary to conduct analyses, in order to conduct analyses it is necessary to have a preliminary design. Design is thus more than just analysis. In order to distinguish clearly between analysis and design and at the same time to identify problems inherent in the design of EQ-resistant structures, it is convenient to analyze the main steps involved in satisfying what can be called the basic design equation.

\[
\text{DEMAND} \leq \text{SUPPLY} \quad (6)
\]

\[
\begin{align*}
\text{on} & \quad \text{Stiffness} \\
\text{Strength} & \quad \text{Strength} \\
\text{Stability} & \quad \text{Stability} \\
\text{Energy absorption and dissipation capacities} & \quad \text{Energy absorption and dissipation capacities}
\end{align*}
\]

Evaluation of the demand and prediction of the supply are not straightforward, particularly for EQ-resistant facilities. Determination of the demand, which is usually done by numerical analyses of mathematical models of the entire soil-foundation-building system, depends not only on the interaction of the system as a whole with the different excitations that originate from changes in the environment, but also on the intrinsic interrelation between demand and supply itself.

In the last three decades, our ability to analyze mathematical models of structures subjected to EQ ground-shaking has improved dramatically. Sophisticated computer programs have been developed and used in the numerical analyses of linear as well as nonlinear seismic responses of three-dimensional models of the bare structures of facilities to certain assumed EQGMs (EQ input). The time is ripe to take advantage of these improvements in analysis in the seismic design of structures. In general, however, these analyses have failed to predict the responses of real facilities, particularly at ultimate limit states. As a consequence of this, and also of the lack of reliable models to predict supplies to real structures, there has not been a corresponding improvement in the design of EQ-resistant structures. There is an urgent need to improve mathematical modeling of the whole systems of real facilities, which in turn requires integrated analytical and experimental research.

The proportioning (sizing) and detailing of the structural elements of a structure is usually done through equations derived from the theory of mechanics of continuous solids or by using empirical formulae. Except in the case of pure flexure, a general theory with reliable equations which can accurately predict energy absorption and dissipation capacities of structural elements and of so-called nonstructural elements, in the case of real buildings, has not been developed. Improving this situation will require integrated analytical and experimental research in the field (through intensive instrumenting of buildings) and in experimental laboratories (through the use of pseudo-dynamic or EQ simulator facilities or both).
4.4.2 Main issues that remain to be resolved for the improvement of the EQRD of structures. As discussed in section 3.2, the information needed to improve EQRD by improving prediction of EQ response of structures can be grouped into the following three basic elements: EQ input, demands on the structure, and supplied capacities to the structure. These three basic elements of the EQ response problem are discussed briefly below.

- **EQ input**: specification (establishment) of design EQs and design criteria. The design EQs depend on the design criteria, or the limit states controlling the design. Conceptually, the design EQ should be that EQGM, out of all probable EQGMs that can occur at the site, which will drive a structure to its critical response. In practice, the application of this simple concept meets with serious difficulties because, firstly, there are great difficulties in predicting the main dynamic characteristics of ground EQGMs which have yet to occur at the building site, and, secondly, because even the critical response of a specific structural system will vary according to the various limit states which could control the design.

Seismic codes specify design EQs in terms of a building code zone, a site intensity factor, or a peak site acceleration. Reliance on these indices, however, is generally inadequate, and methods using ground motion spectra (GMS), and Smoothed Linear Elastic Design Response Spectra (SLEDRS) based on effective peak acceleration (EPA) have been recommended. While this has been a major improvement conceptually, great uncertainties regarding appropriate values for EPA and GMS, as well as for other parameters that have been recommended, persist.

- **Estimation of reliable demands.** The major uncertainties in the estimation of reliable demands (usually done by numerical analysis) are due to difficulties in predicting the following: (1) critical seismic loading during the service life of the structure (lack of properly established design EQs); (2) the state of the entire soil-foundation-superstructure-nonstructural components and contents system when the critical EQGM occurs (proper selection of the mathematical model(s) to be analyzed); (3) internal forces, deformation, stresses and strains induced in the model (structural and stress analysis); and (4) realistic supplies of stiffness, strength, stability, and capacity to absorb and dissipate energy (i.e., realistic hysteretic behavior) of the entire facility system.

- **Prediction of supplies.** The supplies to a facility depend not only on supplies to its bare superstructural system, but also on supplies that result from the interaction of the bare superstructural system with the soil-foundation and the so-called nonstructural components of the facility. For example, masonry walls or partitions (or both) tightly packed as infill into the moment-resisting frames of a building introduce significant changes into the dynamic characteristics of that building. Changes in stiffness, strength, and deformation capacities are illustrated in Figure 22. An evaluation of the test results illustrated in this figure and implications of these results are discussed by Brokken and Bertero [1981]. It is obvious that when interaction occurs between structural and nonstructural components, neglecting such interaction in the selection of numerical characteristics for the design of the structure could lead to completely unrealistic evaluation of the demands, and consequently could result in a poor final design of the entire building system. This observation is confirmed by the fact that many RC moment-resistant space frame buildings infilled with unreinforced masonry performed poorly during the 1985 Mexico EQ. Similarly, a large percentage of the buildings that collapsed during the 1988 Armenia EQ were RC frames infilled with stone.

In considering the basic general design equation, the designer might be tempted to increase supplies in order to overcome the problems created by the uncertainties in the values of demands. However, supply must be increased very carefully, because it may contribute to and considerably increase the demand.

4.5 Directions toward solutions of the main issues in establishment of design EQs

4.5.1 General remarks regarding the need for site seismic hazard assessment. Before embarking on the design of any structure to be constructed at any given site, it is necessary to conduct an analysis of the seismic suitability of the selected site and to define the design events, i.e., what is needed is a reliable site seismic hazard assessment. Recent EQs have indicated that in order to improve the reliability of this analysis and definition, it is necessary to: first, improve the identification of all possible sources of EQs that can affect the site; second, to describe fully and reliably the dynamic characteristics of the ground motions at the source; third, to quantify how the source ground motions are modified (attenuated or amplified) as they are propagated from the source to the site (i.e., to improve
the so-called Attenuation Law); fourth, to identify the types of EQ hazards at the selected site; and finally, to estimate the return periods of EQGMs at different intensity levels.

- Identification of EQ sources. There is a need to improve identification of all possible EQ sources (faults) that can cause hazards at the site. This has been confirmed by the experience of the 1976 Tangshan EQ and the Diablo Canyon Nuclear Power Plant Studies. It will require better zonation maps. Studies conducted after the 1985 Mexico EQ show that for sites located in Mexico City it is necessary to distinguish at least four sources: local EQs; continental plate EQs; intermediate depth EQs; and subduction EQs, which can be located up to 400 km (250 miles) from the city [Rosenblueth, 1989]. It should be noted that, until a few years ago, cities and sites located at rather remote distances, say, 150 km (93 miles), had little concern about EQs. However, in the last two decades, with the advent of taller and slenderer structures, some instances of alarming damage due to EQs occurred in the long natural period structures in cities which were located considerably more than 150 km from the epicenter.

For example, the sway and cracking of many slender (particularly soft first story) medium-to-high rise buildings in Buenos Aires, Argentina, caused dramatic panic among the occupants during the 1977 El Cauite (San Juan) EQ, whose epicenter was located more than 950 km (600 miles) from Buenos Aires. Furthermore, a tall, slender steel water tank near Buenos Aires collapsed during this EQ. It can be concluded from this that:

- EQ disaster can occur at distances from the EQ source that are considerably greater than those usually assumed and reflected in present seismic zonation maps and codes. This statement is substantiated by observations of damages during the following EQs: 1957 Mexico, 1977 Cauite (San Juan, Argentina), 1985 Chile and Mexico and 1990 Philippines.

- Dynamic characteristics of ground motions at the source

- There is a need to consider the possibility of multi-events, i.e., two or more separate fault ruptures (not necessarily in the same fault), leading not only to overlapping ground motions originating from each fracture, but also, and even more importantly, the possibility of downshifting in the duration of strong motions at the epicentral region and at large distances from the epicenter. This is implied in the recorded EQGMs from the following EQs: 1985 Chile, 1985 Mexico and 1990 Philippines.

- There is a need for a more reliable definition of the total strength of an EQ at the source than is given by magnitude. This has resulted in the introduction of the concept and definition of Seismic Moment, which is related directly to the energy released to the source and to its use in what has been defined as Moment Magnitude [Hanks and Kanamori, 1979].

- Attenuation laws: ground motions at the site

- While significant improvement has been achieved in the accuracy of predicting the attenuation of peak acceleration with distance from the epicenter, focal or fault when seismic waves travel through rock or firm soil, the same cannot be said for the travel of waves through very soft soils. Peak ground accelerations are a function of both the source mechanism and the properties of the travel path. The EQGMs recorded during the 1985 Chile and Mexico EQs, and particularly those recorded during the Loma Prieta EQ, show that:

1. Due to combinations of special geological settings and local soil conditions, EQs that at their sources have the same moment magnitude as those contemplated in present seismic codes can generate EQGMs of the damaging intensity specified by the codes at considerably greater distances than those usually reflected in present seismic zonation maps.

2. Due to variations in soil conditions (soil profile), the dynamic characteristics of the induced EQGMs can differ significantly even for sites located at the same large epicentral distances.

3. The frequency content of EQGMs not only varies with epicentral distance, but also is a complex function of source mechanism, focal depth, nature of travel path and site soil topography and profile. Higher frequencies attenuate more rapidly that low ones do.

4. Duration of strong motion generated by multiple events is longer than that caused by single events. Furthermore, this duration tends to increase with moment magnitude, source distance, and soil deposits when compared to rock.

5. Directivity. Recent studies have shown that peak acceleration can vary by a factor of 10 depending on the direction from the epicenter, and that peak velocity can vary by a factor of about 5 depending on the fault rupture process.

- Types of EQ hazards at the selected site. As discussed in section 4.1, damage to human-made facilities may result from different seismic effects (hazards). They have been classified into two main groups: (1) direct effects and (2) indirect, or consequential, effects. Although present U.S. codes have been prepared to provide minimum EQ resistance against the vibration effects of typical EQ shaking, the damage during the following EQs has shown that the economic losses due to other effects can exceed those due to the vibration of buildings: 1964 Alaska (in Anchorage) and Japan (in Niigata), the 1989 Loma Prieta in San Francisco (in the Marina District), and particularly the 1990 Philippines. There is an urgent need for government officials and designers to pay more attention to these and other EQ hazards. The seismic risk resulting from these other physical phenomena should be taken into consideration through reliable regional zonation maps and microzonation of urban areas.

- EQ return period, or EQ recurrence relationship. For any given site, establishing what constitutes an acceptable seismic risk (i.e., an acceptable probability of social or economic consequences due to EQs) requires statistical information regarding the seismic activity in the region where the site is located. For each of the different levels of moment magnitude (or, even better, level of ground motion) that needs to be considered in the design of the structure (design event and design EQGM), it is necessary to establish a relationship between the damage potential of the particular motion and its frequency, i.e., the rate of occurrence of such a motion. At present, there is great uncertainty in predicting such recurrence relationships.

4.5.2 Establishment of design events and the corresponding design EQ. As discussed in sections 3.3.2
and 4.1, for any given building to be constructed on a selected site, present U.S. codes define just one level of hazard (EQGM). From analyses of the damages resulting from recent EQs, particularly the 1984 Morgan Hill, the 1987 Whittier Narrows and the 1989 Loma Prieta, it becomes clear that there is a need to consider more than one level of EQGMs for the design of structures (as is clearly spelled out in the adopted general seismic design philosophy discussed earlier). Before establishing the design EQGMs at the site, it is necessary to define the design events causing such motions.

- **Design events.** According to the EERI Committee on Seismic Risk, the design event(s) is defined as "a specification of one or more EQ source parameters, and of the location of energy with respect to the site of interest used for the EQRD of a structure." Although it is common to define a design event simply by specifying just a magnitude and a slant (focal) distance, a reliable definition of such an event requires the specification of: moment magnitude, return period, epicentral distance, focal depth, fault position, fault type and rupture area (length and depth).

- **Design EQ.** For each of the possible design events (i.e., EQs at the sources that can control the design), it is necessary to define the damage potential of the EQGMs that can be generated at the facility site, or, even better, at the facility foundation. As discussed before, seismic codes specify design EQs in terms of one or two variables. Reliance on these parameters is generally inadequate. To have a reliable definition of a design EQ, it is necessary to specify: its effective peak acceleration, velocity and displacement, its frequency content (particularly the dominant period of the shaking), and the duration of the strong motion. What is important is to have a reliable definition of each of the possible EQGMs at the facility site. The above parameters, as well as other engineering parameters, have been used to define the damage potential of EQGMs.

Since damage involves nonlinear response (inelastic deformation), the only way to estimate damage and the actual behavior of a facility under severe EQ excitation is to consider its inelastic behavior. Guided by this basic concept and by the fact that the damage potential of any given EQ ground shaking at the foundation of a structure depends on the interaction of the intensity, the frequency content, and the duration of the shaking, the dynamic characteristics of the structure, the lecturer believes that one of the most reliable ways to define the damage potential of an EQ ground shaking is to compute its energy input, Ep, to the foundation of the structure together with the other associated parameters [Bertero, 1991].

Recent EQs, particularly the 1989 Loma Prieta, clearly indicate that the level of "acceptable damage" should vary with the function (occupancy category) of the structure. For certain occupancies, there is an urgent need for code specifications that require damage control. To achieve this, it will be necessary to specify at least two of the following levels of design EQs.

- **Service-level design EQs.** During this frequent type of EQ the entire soil-foundation—superstructure—and nonstructural components and contents system should remain elastic (i.e., without any damage).

- **Functional or operational-level design EQs.** During this type of occasional EQ, the entire building could undergo some degree of nonstructural as well as structural damage (small yielding) which will not disrupt the operation of the facility.

- **Safety or survival-level design EQ.** Under this rare circumstance, the building should not collapse or suffer serious damage that can jeopardize human life.

For a reliable definition of the design EQ, it is necessary to specify at least the three translational components of the critical EQGMs at the site. This need has been clearly identified through observations of damage during the following EQs: 1979 Imperial Valley and 1985 Chile and Mexico. It has also been confirmed by experiments conducted on models of building structures using EQ simulator as well as pseudo-dynamic testing facilities [Bertero, 1986].

As pointed out previously, while the introduction of GMS and SLEDRS based on an EPA has been a major improvement conceptually (particularly for the design of essential facilities which should remain practically in their elastic range even under the extreme EQGMs), great uncertainties regarding appropriate values for EPA, GMS and SLEDRS, as well as for other parameters that have been recommended to improve this situation, persists [Bertero, V.V., 1991, Kunming, P.R. China and Bertero, V.V., 1991, Int'l Conf. on Seismic Zonation]. The uncertainties are even greater in the case of standard facilities in which structural damage under extreme EQs is acceptable. The lecturer believes that a promising engineering parameter for improving selection of proper design EQs, particularly when structural damage is acceptable and it is necessary to define the Smoothed Inelastic Design Response Spectra is the concept of Energy Input, Eo, of the EQGM, and its associated parameters, which can be obtained through the use of energy concepts.

5 USE OF ENERGY CONCEPTS

5.1 General remarks

Traditionally, displacement ductility has been used as a criterion to establish Inelastic Design Response Spectra (IDRS) for EQRD of buildings [Bertero and Uang, 1992]. The minimum required strength (or capacity for lateral force) of a building is then based on the selected IDRS. As an alternative to this traditional design approach, an energy-based design method was proposed by Housner (1956). Although estimates have been made of input energy to SDOFS [Berg and Thomasides, 1960], and even of MDOFS, (steel structures designed in the 1960s for some of the existing recorded EQGMs) [Anderson and Bertero, 1969], it is only recently that this approach has gained extensive attention [Akiyama, 1985]. This design method is based on the premise that the energy demand during an EQ (or an ensemble of EQs) can be predicted, and that the energy supply of a structural element (or structural system) can be established. In a satisfactory design, the energy supply is larger than the energy demand.

To develop reliable design methods based on an energy approach, it is necessary to derive the energy equations. Although real structures are usually MDOFS, to facilitate the analysis and understanding of the physical meaning of the energy approach, it is convenient first to derive the energy equations for SDOFS and then to derive these equations for MDOFS.
5.2 Derivation of energy equations.

Uang and Bertero [1988] give a detailed discussion of the derivation of the two basic energy equations starting directly from Eq. 7 for a given viscous damped SDOFS subjected to an EQGM.

\[ m \ddot{v} + c \dot{v} + f_v = 0 \]  

(7)

where: \( m = \) mass; \( c = \) viscous damping coefficient; \( f_v = \) restoring force (if \( k = \) stiffness, \( v = kv \) for a linear elastic system); \( \dot{v} = \nabla + \dot{v} = \) absolute (or total) displacement of the mass with respect to the ground; and \( \dot{v} = EQ \) ground displacement.

5.2.1 Derivation of “absolute” energy equation.

Integrating Eq. 7 with respect to \( v \) from the time that the EQ excitation starts, and considering that \( v = v_t - v_g \), it can be shown that

\[ \frac{m(v_t)^2}{2} + \int c \dot{v} dv + \int f_v dv = \int m \ddot{v} dv \]  

(8)

\[ E_K + E_\xi + E_a = E_I \]  

(9)

"Absolute" Kinetic Energy Damping Energy Absorbed "Absolute" Input Energy

"Absolute" strain energy, \( E_p \), and of irrecoverable hysteretic energy, \( E_H \), Eq. 9 can be rewritten as

\[ E_I = \int (\sum_{i=1}^{N} m \ddot{v}_i) dv \]  

(15)

Where: \( m_i \) is the lumped mass associated with the \( i \)-th floor, and \( \ddot{v}_i \) is the total acceleration at the \( i \)-th floor. In other words, \( E_I \) is the sum of the work done by the total inertia force \( (m_i \ddot{v}_i) \) at each floor through the ground displacement \( \dot{v}_g \). Analysis of results obtained from experiments conducted on medium rise steel dual systems indicates that the \( E_I \) to a multi-story building can be estimated with sufficient practical accuracy by calculating the \( E_I \) of a SDOFS using the fundamental period of the multi-story structure.

5.2.2 Derivation of “relative” energy equation. Eq. 7 can be rewritten as

\[ m \ddot{v} + c \dot{v} + f_v = -m \ddot{v}_g \]  

(11)

Integrating Eq. 11 with respect to \( v \) leads to:

\[ \frac{m(v_t)^2}{2} + \int c \dot{v} dv + \int f_v dv = -\int m \ddot{v}_g dv \]  

(12)

\[ E_K' + E_\xi + E_a = E_I' \]  

(13)

"Relative" Kinetic Energy Damping Energy Absorbed "Relative" Input Energy

As \( E_a = E_p + E_H \), Eq. 13 can be rewritten as

\[ E_I' = E_K' + E_s + E_\xi + E_H \]  

(14)

The \( E_I' \) that is defined as the "Relative Input Energy" represents the work done by the static equivalent external force \( (m_i \ddot{v}_g) \) on the equivalent fixed-base system; that is, it neglects the effect of the rigid body translation of the structure.

5.2.3 Difference between input energies from different definitions. Uang and Bertero [1988] discuss in detail the differences between the values of the input energies \( E_I \) and \( E_I' \). Although the profiles of the energy time histories calculated by the absolute energy equation (8) differ significantly from those calculated by the conventional relative equation (12), the maximum values of \( E_I \) and \( E_I' \) for a constant displacement ratio are very close in the period range of practical interest for buildings, which is 0.3 to 5.0 secs.

5.2.4 Input energy to MDOFS. The \( E_I \) for an \( N \)-story building can be calculated as follows [Uang and Bertero, 1988]:

\[ E_I = \int (\sum_{i=1}^{N} m \ddot{v}_i) dv \]

(15)

5.3 Advantages of using energy concepts in seismic design of structures.

Equation (10) can be rewritten as

\[ E_I = E_K + E_D \]  

(16a)

\[ E_I = E_K + E_s + E_\xi + E_H \]  

(16b)

where \( E_D \) can be considered as the stored elastic energy and \( E_H \) the dissipated energy. Comparing this equation with the design equation (6), it becomes clear that \( E_I \) represents the demands, and the summation of \( E_K + E_D \) represents the supplies. Equation (16a) points out clearly to the designer that to obtain an efficient seismic design, the first step is to have a good estimate of the \( E_I \) for the critical EQGM. Then the designer has to analyze if it is possible to balance this demand with just the elastic behavior of the structure to be designed or will it be convenient to attempt to dissipate as much as possible some of the \( E_I \), i.e., using \( E_D \). As revealed by Eq. (16b), there are three ways of increasing \( E_D \): one is to increase \( E_K \) by increasing the linear viscous damping, \( E_s \); another is to increase the hysteretic energy, \( E_\xi \); the third is a combination of increasing \( E_D \) and \( E_H \). At present it is common practice to just try to increase the \( E_H \) as much as possible through inelastic (plastic) behavior of the structure, which implies damage of the structural members. Only recently it has been recognized that it is possible to increase significantly the \( E_H \) and control
damage throughout the structure through the use of energy dissipation devices. Furthermore, as discussed by Bertero [1992] in a paper presented at this conference (in a discipline, a certain degree of damage, the Safety or Survival-Level Design EQ is defined through Smoothed Inelastic Design Response Spectra, SIDS. Most of the SIDS that are used in practice (seismic codes) have been obtained directly from SEDRS, through the use of the displacement ductility ratio, \( \mu \), or reduction factors, \( R \). The validity of such procedures has been questioned, and it is believed that at present such SIDS can be obtained directly as the mean or the mean plus different values of standard deviation of the Inelastic Response Spectra, IRS, corresponding to all the different time histories of the severe EQMs that can be induced at the given site from EQs that can occur at all of the possible sources affecting the site [Bertero, 1991, Seismic Zonation].

While the above information is necessary to conduct reliable design for safety, i.e., to avoid collapse and serious damage that can jeopardize human life, it is not sufficient. Although the IRS takes into account the effects of duration of strong motion in the required strength, these spectra do not give an appropriate idea of the amount of energy that the whole facility system will dissipate through hysteretic behavior during the critical EQGM. They give only the value of maximum global ductility demand. In other words, the maximum global ductility demand by itself does not give an appropriate definition of the damage potential of EQGMs. As discussed previously, it has been shown that a more reliable parameter than those presently used in assessing damage potential is the \( E_I \). As is clearly shown by Eq. (8), this damage potential parameter depends on the dynamic characteristics of both the shaking of the foundation and the whole building system (soil-foundation-superstructure and nonstructural components). Now the question is: Does the use of the SIDS for a specified global \( \mu \) and the corresponding \( E_I \) of the critical EQGM give sufficient information to conduct a reliable seismic design for safety?

Although the use of \( E_I \) can identify the damage potential of a given EQGM and, therefore, permits selection, amongst all the possible motions at a given site, of that which will be the critical one for the response of the structure, it does not provide sufficient information to design for safety level. From recent studies [Uang and Bertero, 1988; Bertero, 1991, Seismic Zonation] it has been shown that the energy dissipation capacity of a structural member, and therefore of a structure, depends on the energy branch and deformation paths. Although the energy dissipation capacity under monotonic increasing deformation may be considered as a lower limit of energy dissipation capacity under cyclic inelastic deformation, the use of this lower limit could be too conservative for EQRD. This is particularly true when the ductility deformation ratio, say \( \mu \), is limited, because of the need to control damage of nonstructural components or other reasons, to low values compared to the ductility deformation ratio reached under monotonic loading. Thus, effort should be devoted to determining experimentally the energy dissipation capacity of main structural elements and their basic subassemblies as a function of the maximum deformation ductility that can be tolerated, and the relationship between energy dissipation capacity and loading and/or deformation history.

From the above studies, it has also been concluded that damage criteria based on the simultaneous consideration of \( E_I \) and \( \mu \) (given by SIDS), and the \( E_{TH} \) (including Accumulative Ductility Ratio, \( \mu_{ac} \), and Number of Yielding Reversals, \( NYR \), and Number of Equivalent Yielding Cycles at \( \mu_{max, NYR,max} \), are promising parameters for defining rational EQRD procedures. The need for considering all of these engineering parameters rather than just one will be justified below by a specific example. From the above discussion, it is clear that when significant damage can be tolerated, the search for a single parameter to characterize the EQGM or the design EQ for safety is doomed to fail.

5.4 Information needed to conduct reliable EQRD.

5.4.1 General remarks. It has been pointed out previously that the first and fundamental step in EQRD of structures is the reliable establishment of the design EQs. This requires a reliable assessment of the damage potential of all the possible EQGMs that can occur at the site of the structure. Currently, for structures that can tolerate a certain degree of damage, the Safety or Survival-Level Design EQ is defined through Smoothed Inelastic Design Response Spectra, SIDS. Most of the SIDS that are used in practice (seismic codes) have been obtained directly from SEDRS, through the use of the displacement ductility ratio, \( \mu \), or reduction factors, \( R \). The validity of such procedures has been questioned, and it is believed that at present such SIDS can be obtained directly as the mean or the mean plus different values of standard deviation of the Inelastic Response Spectra, IRS, corresponding to all the different time histories of the severe EQMs that can be induced at the given site from EQs that can occur at all of the possible sources affecting the site [Bertero, 1991, Seismic Zonation].

While the above information is necessary to conduct reliable design for safety, i.e., to avoid collapse and serious damage that can jeopardize human life, it is not sufficient. Although the IRS takes into account the effects of duration of strong motion in the required strength, these spectra do not give an appropriate idea of the amount of energy that the whole facility system will dissipate through hysteretic behavior during the critical EQGM. They give only the value of maximum global ductility demand. In other words, the maximum global ductility demand by itself does not give an appropriate definition of the damage potential of EQGMs. As discussed previously, it has been shown that a more reliable parameter than those presently used in assessing damage potential is the \( E_I \). As is clearly shown by Eq. (8), this damage potential parameter depends on the dynamic characteristics of both the shaking of the foundation and the whole building system (soil-foundation-superstructure and nonstructural components). Now the question is: Does the use of the SIDS for a specified global \( \mu \) and the corresponding \( E_I \) of the critical EQGM give sufficient information to conduct a reliable seismic design for safety?

Although the use of \( E_I \) can identify the damage potential of a given EQGM and, therefore, permits selection, amongst all the possible motions at a given site, of that which will be the critical one for the response of the structure, it does not provide sufficient information to design for safety level. From recent studies [Uang and Bertero, 1988; Bertero, 1991, Seismic Zonation] it has been shown that the energy dissipation capacity of a structural member, and therefore of a structure, depends on the energy branch and deformation paths. Although the energy dissipation capacity under monotonic increasing deformation may be considered as a lower limit of energy dissipation capacity under cyclic inelastic deformation, the use of this lower limit could be too conservative for EQRD. This is particularly true when the ductility deformation ratio, say \( \mu \), is limited, because of the need to control damage of nonstructural components or other reasons, to low values compared to the ductility deformation ratio reached under monotonic loading. Thus, effort should be devoted to determining experimentally the energy dissipation capacity of main structural elements and their basic subassemblies as a function of the maximum deformation ductility that can be tolerated, and the relationship between energy dissipation capacity and loading and/or deformation history.

From the above studies, it has also been concluded that damage criteria based on the simultaneous consideration of \( E_I \) and \( \mu \) (given by SIDS), and the \( E_{TH} \) (including Accumulative Ductility Ratio, \( \mu_{ac} \), and Number of Yielding Reversals, \( NYR \), and Number of Equivalent Yielding Cycles at \( \mu_{max, NYR,max} \), are promising parameters for defining rational EQRD procedures. The need for considering all of these engineering parameters rather than just one will be justified below by a specific example. From the above discussion, it is clear that when significant damage can be tolerated, the search for a single parameter to characterize the EQGM or the design EQ for safety is doomed to fail.

5.4.2 Importance of simultaneously considering the \( E_I \), IDRS, and \( E_{TH} \) (including \( \mu_{ac} \) and \( NYR \)) for defining the safety-level design EQ. Figures 23-27 permit comparison of the values of these different engineering parameters for two recorded EQGMs San Salvador (SS) and Chile (CH); Table 5 summarizes approximate maximum values for these parameters corresponding to each of these two different recorded EQGMs. The importance and, actually, the need for simultaneously considering all the above parameters in selecting the critical EQGMs and, therefore, for defining the safety-level design EQ, is well illustrated by analyzing the values of these parameters for these two records.

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Table 5. Parameters corresponding to the Chile (CH) and San Salvador (SS) EQGMs

<table>
<thead>
<tr>
<th>EQ RECORD</th>
<th>PGA</th>
<th>EPS</th>
<th>R</th>
<th>$t_1$ (sec)</th>
<th>$C_r$</th>
<th>$E_r/n$</th>
<th>$P_r$</th>
<th>HTR</th>
<th>$C_r$</th>
<th>$E_r/n$</th>
<th>$P_r$</th>
<th>HTR</th>
<th>$C_r$</th>
<th>$E_r/n$</th>
<th>$P_r$</th>
<th>HTR</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH</td>
<td>0.67</td>
<td>0.37</td>
<td>16</td>
<td>35.6</td>
<td>0.95</td>
<td>11,200</td>
<td>11</td>
<td>20</td>
<td>0.70</td>
<td>9,600</td>
<td>12</td>
<td>53</td>
<td>0.67</td>
<td>8,800</td>
<td>133</td>
<td>121</td>
</tr>
<tr>
<td>SS</td>
<td>0.69</td>
<td>0.54</td>
<td>17</td>
<td>4.3</td>
<td>1.06</td>
<td>2,400</td>
<td>5</td>
<td>6</td>
<td>0.69</td>
<td>1,900</td>
<td>12</td>
<td>9</td>
<td>0.69</td>
<td>1,700</td>
<td>261</td>
<td>9</td>
</tr>
</tbody>
</table>

**Figure 23.** Yielding strength spectra ($C_r$) for CH and SS records (5% damping)

**Figure 24.** Input energy ($E_r/n$) for CH and SS records (5% damping)

**Figure 25.** Hysteretic energy equivalent velocity ($V_H$) spectra for CH and SS records (5% damping)

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San Salvador (SS) vs. Chile (CH) Records. From analyses of the values of Peak Ground Acceleration (PGA), Effective Peak Acceleration (EPA), and Effective Peak Velocity (EPV) given in Table 5, which are values presently used to define the seismic hazard zoning maps, it might be concluded that the damage potential of these EQGMs is quite similar. One can arrive at a similar conclusion if the values of the required Yielding Strength Coefficient, \( \gamma = \frac{V_v}{W} \), for different values of \( \mu \) are compared, or, in other words, if the IRS for different \( \mu \) are compared (Fig. 23). However, a completely different picture is obtained when the values of the \( E_p \), \( E_h \), \( \mu_h \) and NYR for different values of \( \mu \) are compared. The \( E_p \) for the CH record can be as much as 5 times the \( E_p \) for the SS record (Fig. 24). The \( E_h \) [represented by the equivalent hysteretic velocity, \( V_h = \frac{2E_h}{m} \), in Fig. 25] for the CH record is more than 3 times the \( E_h \) for the SS record when the period, \( T \), is about 0.5 secs. and nearly 2 times when the \( T \) varies from 0.5 secs. up to 1.5 secs. The \( \mu_h \) for the CH record are 2 to 4 times higher than those of the SS record (Fig. 26). The NYR for the CH record and for a \( \mu \leq 6 \) and \( T < 0.5 \) seconds are more than 10 times the NYR for the SS record (Fig. 27). For a \( \mu = 4 \) and \( T > 0.5 \), the NYR for the CH record are more than 5 times those of the SS record.

From the above comparison, it is clear that the damage potential of the CH recorded EQGM is significantly (at least 3 times) greater than that of the SS record in spite of the fact that PGA, EPA, EPV, ERS (IRS for \( \mu = 1 \)) and even the IRS for different values of \( \mu \) are very similar. Thus, the importance of evaluating the \( E_p \) and \( E_h \) (represented herein by \( V_h \), \( \mu_h \) and NYR spectra) which are functions of the duration of strong ground motions, \( t_d \), becomes very clear. While the \( t_d \) for the CH records is 36 secs., the \( t_d \) for the SS record is only 4.3 secs. (see Table 5). The importance of \( t_d \) in judging damage control is discussed in by Bertero [1991, 4th Zonation Conference]. While the above spectra are very helpful in preliminary design, for the final design (detailing of members), the ideal would be to have the time history of the \( E_h \), i.e., the time history of the load-deformation relationship of the designed structure.

The assembly of all the above spectra and time histories can be considered the ideal information for making reliable decisions regarding the critical EQGMs and, therefore, for reliable establishment of design EQs and design criteria. Thus, this basic information should be gathered in order to improve seismic codes as well as for the design of important facilities. It should be noted that all of the above spectra can be computed by an engineer who is provided with the time history of all possible EQGMs at the site of the structure.

It has to be recognized that, for practical preliminary design of most standard facilities, it will be convenient to specify the minimum possible information to keep it simple. It is believed that, for a given structural site, this minimum could be the \( E_p \) and the SIDRS for strength and displacement of all the possible EQGMs at

![Figure 26. Cumulative displacement ductility ratio (\( \mu_d \)) spectra for CH and SS records (5% damping)](image)

![Figure 27. Number of yielding reversal (NYR) spectra for CH and SS records (5% damping)](image)
that site. The $E_I$ would permit selection of the type of 
critical EQGM, i.e., the one that will induce the largest 
damage. The SIDRS, corresponding to the type of 
critical EQGM can be used to conduct the preliminary 
design of the structure. Once a preliminary design is 
completed, it will be possible to obtain all the other 
information, i.e., the $E_H$, $p_{NYR}$, $NYR$ and $NEYE_{max}$ for 
different $p_{NYR}$, from nonlinear, dynamic time history 
analyses, taking advantage of the significant advances 
achieved in the development of computer programs for 
such analysis. This will permit checking the adequacy 
of the preliminary design. While a nonlinear analysis 
of the preliminary design using a static approach (i.e., 
equivalent static lateral force) can give an idea of the 
strength and deformation capacities as well as a lower 
limit of the available $E_H$ and therefore it should be used 
if no time history of the critical EQGMs is possible, this 
type of analysis will not supply any information 
regarding the $p_{NYR}$, $NYR$, $NEYE_{max}$ or the sequence of 
damage.

From the above discussion, it becomes clear that, if 
future codes perpetuate simple procedures for seismic 
design specifying only smoothed strength response 
spectra, it will be necessary to place more stringent 
limitations on the type of structural systems that could 
be used and on how such procedures can be applied, 
and to have very conservative regulations in the siting 
and detailing for ductility and in the maximum 
acceptable deformations.

6. NEED FOR FORMULATION OF A CONCEPTUAL 
METHODOLOGY FOR EQRD OF STRUCTURES

6.1 Introduction remarks.

As discussed previously, present seismic codes fail short 
of the goals and objectives of the worldwide accepted 
philosophy of EQRD. Furthermore, these codes, in their 
tempt to be simple (as they should be), have tried hard 
to simplify the complex problem of EQRD by 
developing design procedures based on just one 
parameter. The result is codes that are not transparent, 
i.e., codes whose regulations do not present in a visible 
way the basic concepts which govern the EQRD of 
structures.

Although it is generally recognized that damage is due 
to deformation, there is no agreement regarding the main 
criterion for preliminary EQRD of structures. Perhaps as 
a consequence of past and present code requirements, 
present practice emphasizes the use of strength in the 
preliminary design of structures. More specifically, in 
most of the present codes, the preliminary design is 
based only on base shear strength, with a requirement to 
check the drift by elastic analysis. The insistence on 
using only strength as primary criterion is perhaps a 
consequence of the following two reasons: first, it allows 
the practice of trying to design for ultimate or safety 
limit state by reducing the actual inelastic design to one 
at working stress where linear elastic analysis can be 
used; and second, there is an assumption that there is a 
unique relation between strength and stiffness. This 
assumption ignores the fact that, particularly in the case 
of RC structures, it is possible to change the strength of 
a structure significantly without changing its stiffness. 
While preliminary design based just on base shear 
strength would be justified for design where 

serviceability (elastic response) controls, it cannot be 
accepted where the design is controlled by the ultimate 
(safety) limit state, where large plastic deformation is 
accepted: at this limit state, base shear strength of a 
given designed structure is insensitive to variation of 
deformation and, therefore, to damage. Once structures 
yield and deform as mechanisms develop, base shear 
strength remains constant, while the deformation can 
take any value, from its yielding value up to the 
maximum value, at which collapse (sudden significant 
drop in resistance) occurs. In view of the above 
insensitivity of the base shear strength to damage in the 
inelastic (plastic) range of response, it is perhaps 
unfortunate that in the past most of the efforts in 
improving EQRD have been expended in designing for 
strength only, without proper consideration of the role 
of deformation. Damage is a consequence of 
deformations. For any structure that is responding in the 
inelastic (plastic) range under practically a constant 
strength, the degree or level of damage depends upon 
the amount of the plastic deformation that the structure 
undergoes. Thus, to control damage it is necessary to 
control deformations. The question is how to achieve 
such control at the different levels of EQ shaking that 
can occur during the life of the structure. Bertero et al. 
[1991] discuss in detail the issues involved in achieving 
such control at the serviceability and safety limit states, 
pointing out the need for ductility and drift control. The 
need for drift control can be summarized by the 
following statement.

While displacement ductility factors generally provide 
a good indication of structural damage, they do not 
usually adequately reflect the damage to nonstructural 
elements. This is an important limitation in EQRD of 
built-up, since a significant portion of the hazard to 
occupants and of the total cost of repairing EQ damage 
is a consequence of nonstructural damage. Nonstructural 
damage is more dependent on the relative displacements 
(drift) than on the overall displacements. To obtain a 
reliable measure of nonstructural damage, maximum 
drifts must remain unnormalized or be divided by the 
value of drift corresponding to the damage threshold. 
Nonstructural damage estimates based on drift ductility 
ratios may be misleading. For example, nonstructural 
damage for relatively rigid structures may be small even 
for large values of displacement, since the yield 
displacement may be well below the nonstructural 
damage threshold. On the other hand, the nonstructural 
damage and lateral displacements for flexible structures 
may become intolerably large even before significant 
yielding develops.

To produce serviceable, safe and economical facilities, 
EQRD methods must incorporate drift (damage) control 
in addition to lateral displacement ductility as design 
constraints.

The control of the drift of a structural system under 
EQ excitation is important for at least three different 
reasons: (1) to maintain architectural integrity, thereby 
avoiding unacceptable damage to nonstructural 
components; (2) to limit structural damage and avoid 
structural instability (P-Δ) problems; and (3) to avoid 
human discomfort under frequent minor or even 
occasional moderate EQ shaking.

Story drifts and drift ductility factors may also be 
useful in providing information on the distribution of 
structural damage. Unfortunately, conventionally 
computed story drifts may not adequately reflect the
potential structural or nonstructural damage to multistory buildings. In some structures, a substantial portion of the horizontal displacements results from axial deformations in the columns. Story drifts due to these deformations are not usually a source of damage [Fig. 28a].

A better index of both structural and nonstructural damage is the tangential story drift index, \( R_T \). As schematically indicated in Fig. 28b, the intent of this index is to measure the shearing distortion within a story. For the displacement components shown in Fig. 28c and assuming that floor diaphragms are rigid in their own plane, the average tangential drift index is equal to

\[
R_T = \frac{1}{H} (u_T - u_L) + \frac{1}{L} (u_T + u_L - u_L - u_T)
\]

(17)

in which \( L \) is the bay width and \( H \) is the story height. This first term on the right-hand side of Eq. 17 is the conventional story drift index, and the second is a correction applied for each bay accounting for the slope of the floors above and below the story. It may not be appropriate to average the values of \( R_T \) for a story when the pattern of axial column deformations varies greatly across the structure (e.g., frames with structural walls).

In recognition of the noted weakness of present seismic code EQDR procedures based on base shear strength, which is insensitive to damage in the inelastic (plastic) range, there have been proposals that preliminary design be based only on lateral stiffness, i.e., only on controlling interstory drift.

### 6.2 Recommended practical methods for designing consideringIDI

A simplified method for estimating lateral drift of RC structures has been suggested by Sozen [1983]. The method is intended to be used for interpreting experience and evaluating relative merits of different structural schemes and member sizes on the basis of a tolerable damage criterion. The method is conveniently used in preliminary evaluation by simple estimates of the base shear capacity coefficient.

Shimazaki [1984 and 1988] investigated the effects of strength and stiffness and of the type of EQGM on nonlinear displacement response of SDOF systems. The results obtained show that the nonlinear displacement

![Figure 28. Computation of tangential interstory drift index (IDI)](image)

![Figure 29. Nonlinear displacement response spectra](image)
response is equal to the linear response spectral values if the system has a certain strength which is determined by dimensionless parameters for strength, initial period, and type of EQGM.

Recently Qi and Moehle [1991] and Moehle [1992] developed two simple and practical EQRD procedures based on displacement (drift) information. One uses displacement information directly, and the other, a ductility-ratio approach, uses it indirectly, establishing ductility requirements as a function of the provided strength and the strength required for elastic response.

Results obtained by Miranda [1991] have shown that the nonlinear displacements are very sensitive to the dynamic characteristics of the EQGMs and in some cases the displacement can be significantly higher than those computed from a linear elastic response, particularly if high $\mu_s$ are used in the derivation of the yielding strength (see Fig. 29 for the case of Corralitos and Hollister EQGMs recorded during the 1989 Loma Prieta EQ). This observation agrees with the results obtained by Kappos [1990]. These observations also agree with results reported by Hwang and Jaw [1990], which recommend the use of the following empirical formula for estimating the deflection amplification factor $C_q$ (defined as the ratio of absolute maximum interstory displacement to the corresponding value from a linear time history analysis).

$$\ln C_q = 0.414(\ln \mu_s)$$  \hspace{1cm} (18)

where $\mu_s$ is the maximum story ductility ratio.

Hatamoto et al. [1990] propose a newly automated seismic design method for RC frames which aims at uniform energy dissipation throughout the building frame, so that the resulting damage is uniformly distributed as much as possible over all elements.

Because of the limitations involved (due to the assumptions made to simplify the design procedure) in applying each of the practical methods on the basis of the use of just one parameter, whether strength (as in the present code), or lateral drift, and because of the difficulties in specifying very clearly the limitations on the application of these proposed methods, the method believes that a more rational approach to EQ-resistant preliminary design is one that recognizes from the beginning of the EQRD process the importance of strength and stiffness (control of deformation) and which also recognizes that these two factors, while strongly interrelated in the case of elastic response, clearly have a weaker relation to each other in the case of inelastic response. In this last case, the lateral deformation at ultimate limit state depends on the yielding strength provided to the structure. To control inelastic deformation, it is necessary to provide the structure with a minimum yielding strength. Therefore, to achieve an efficient preliminary EQRD, there is a need to consider two requirements simultaneously: the strength (based on rational use of $\mu_s$ and $C_q$), and the deformation (based on the limitation of IDI), and their combined effect on the energy capacity of the whole facility system.

The above need for a rational and transparent approach to the issue of improving EQRD procedure for new facilities and for upgrading existing hazardous facilities has motivated the lecturer and his research associates to attempt to develop and apply what can be called a conceptual methodology for EQRD of structures.

6.3 Conceptual methodology for EQRD [Bertero, R., and Bertero, V., 1992]

This proposed conceptual methodology is in compliance with the worldwide accepted EQRD philosophy and is based on well-established fundamental principles of structural dynamics, the mechanical behavior of the entire facility system, and comprehensive design. It takes into account from the very beginning of the EQRD procedure (i.e., from the preliminary design on) the simultaneous demands for strength, deformation and their combined effects on the demanded and supplied energy capacities of the entire facility system. The proposed conceptual methodology leads to a rational and transparent EQRD procedure, which is divided for convenience into two main phases. The first phase covers the acquisition and processing of the data needed to establish reliable design EQs for at least two limit states: serviceability and safety. The second phase is devoted to the design of the structure for the design EQs: it consists of an iterative procedure, starting with an efficient preliminary design followed by an analysis of the preliminary design, ending with a final design.

The main advantage of this proposed conceptual methodology is that, notwithstanding the great uncertainties in the quantification of some of the concepts involved in its codification, the numerical quantification of some of the concepts can be improved without changing the format of this codified methodology as new and more reliable data are acquired.

Figure 30. Flow chart for EQRD procedure
This conceptual methodology has been developed to improve EQRD by improving the numerical design phase of the whole EQRD procedure. Thus, this conceptual methodology should not be confused with what the lecturer [Bertero 1980 and 1982] has defined as the 'avoidance of minimization of problems created by the effects of seismic excitation through applying the understanding of mechanical behavior (performance) of the entire soil-foundation-building system rather than using numerical computations. Illustrations of this definition and guidelines for its application to improve EQRD and EQRC are given in the above references, which conclude that, "Because of large uncertainties in estimating the demands and real supplies in the EQRD of buildings, it is of paramount importance to pay more attention to 'conceptual' than to 'numerical' design.' By proper selection of building configuration, structural layout, structural system, structural material and nonstructural components and their materials, and by proper proportioning and detailing of the structural and nonstructural components, it is possible to reduce and control the uncertainties in the demands and supplies. Thus, sophistication in these selections and proportionings and detailing is more important that sophistication in numerical analysis of mathematical models to estimate demands and supplies.

The "conceptual methodology phase" (numerical design) should be applied after the "conceptual design phase" of the overall EQRD and EQRC problems has already been conducted. Thus, from the point of view of the numerical design phase, the formulation and solution of the EQRD problem can be summarized as follows.

**Given:**
- Function of building; site of building; and design of building configuration, structural layout (including foundation), structural system, structural material and nonstructural components and contents.

**Required:**
- An efficient (optimum) EQRD of the entire building.

**Solution:**
- A technically efficient and economical final solution requires an iterative procedure, starting with an efficient preliminary EQRD.

The iterative procedure required for the solution of the EQRD problem is illustrated in Figure 30. In order to attain an efficient EQRD of a structure, it is necessary that the procedure at our disposal be rational, transparent and reliable. Present seismic code design procedures do not satisfy these requirements. As illustrated in Figure 30, it is convenient in the formulation of such procedures to divide the formulation into two main phases: the establishment of design EQs and the development of an efficient and reliable design procedure for the building against the design EQs. Although it would be ideal for the designer to be involved in both of these phases, in general it is not necessary, because the design EQs can be established for different site types by a group of experts.

6.3.1 First phase: establishment of design EQs. This phase covers the acquisition and processing of data needed for the establishment of the design EQs. As discussed previously, the establishment of reliable design EQs is the key step in achieving a reliable EQRD, because to achieve any reliable prediction of the demands on a structure, it is necessary to have a reliable definition of the EQGMs against which the structure has to be designed.

**Acquisition of data.** The needed data and the problems involved in their acquisition can be summarized as follows.

**Given:**
- The site of the building (soil profile and topography).

**Required:**
- Return periods of the different levels of EQGMs that may occur at the site of the building and their damage potential for the entire building system for at least its service and safety limit states.

**Solution:**
- Conduct a reliable analysis of the site (soil profile and topography); identify all of the possible sources of EQs from which the EQGMs that could affect the building could be generated; define the seismic activity at the site due to all possible EQ sources in the form of time histories of the possible EQGMs and their recurrence periods (Tj); select Tj for the service and safety limit states.

Ideally, the acquisition of the needed data should be based on EQGM records from the site. If there are not enough recorded EQGMs for the site, the needed data can be obtained either from recorded EQGMs at sites with similar soil profile and topography or by using numerical synthesis to generate several probable EQGM time histories.

**Processing of data.** The main objective of this key step in the establishment of the design EQs is to process the data about the probable future EQGMs at the site to facilitate the reliable selection of the design EQs. According to the discussions presented above in sections 4 and 5, the problems involved in this step and their solutions can be summarized as follows.

**Given:**
- Time histories of probable EQGMs for at least service and safety limit states.

**Required:**
- For serviceability limit state: the Smoothed Linear Elastic Design Response Spectra (SLEDS) for strength (Cj) and for displacement (Sj).
- For Safety limit state: the SLEDS and Smoothed Inelastic Design Response Spectra (SIDRS) (for different values of μ) for Cj, Sj for the spectra of parameters for evaluation of the cumulative damage caused for cyclic load reversals (input energy (Ej), hysteretic energy (Eq), cumulative ductility ratio (μ), Number of Yielding Reversals (NYR) and Number of Equivalent Yielding Cycles (NEYCHmax)).

**Solution:**
- Computation of the LERS and the Inelastic Response Spectra (IRS) (for different values of μ) for Cj and for Sj for each of the possible EQGMs that can be originated at the site from the different EQ sources. From statistical studies of the LERS, find the SLEDRS and SIDRS. In smoothing the LERS and IRS, careful consideration should be given to the standard deviation as well as to the uncertainties regarding the dynamic characteristics of future EQGMs. To obtain the critical EQGMs that are to
be considered for safety level where some level of damage is tolerated (i.e., μ > 1), conceptually it is necessary to compute for each EQGM the following spectra: Ten, Eh, μ, NYR, and NEYCMax, as well as the hysteretic behavior history. Selection of critical EQGMs can be simplified by using recently proposed damage indices.

For practical application of this solution, R. Bertero and V. Bertero [1992] have used the suggestion of Fajfar et al. [1992], who have introduced the concept of an equivalent reduced ductility factor, μe, to account for the effect of cumulative damage caused by cyclic load reversals, as discussed in section 5.4.2. Three different damage models were used. The first two models, which are based on maximum displacements and dissipated energy, respectively, yield upper and lower limits for the equivalent reduced μe. The third model is based in the Park-Ang [1984] damage model, DM, which can be written in the following form:

$$DM = \frac{\delta}{\delta_u} + \beta \frac{E_h}{F_y \mu_u} \frac{\mu}{\mu_u} + \beta \frac{E_h}{F_y \Delta \mu_u}$$  \hspace{1cm} (19)

where β is a parameter that depends on structural characteristics.

Using the Park-Ang model together with the dimensionless parameter γ, introduced by Fajfar et al. [1992], the following equation for μe has been obtained for SDOFS

$$\mu_e = \sqrt{1 + 4(DM) \beta \gamma^2 \mu_e - 1}$$  \hspace{1cm} (20)

This equation points out that the reduction due to low cyclic fatigue is controlled by the parameters β and γ and the desired (or permissible). As pointed out previously, although it would be highly desirable that the designer be involved in the data acquisition and processing for establishing the design EQs, in general this is unnecessary: for any given urban area, a group of geoscientists, geotechnical engineers and structural engineers can establish the design EQs for different limit states (at least service and safety) according to the different site conditions in each urban area.

6.3.2 Second phase: design procedure. This second phase of the proposed conceptual methodology is devoted to the design (sizing and detailing of the members and their connections and supports) of the entire building system against the critical combination of the established design EQs with other excitations that can act on the building according to its site location. As illustrated in Fig. 30, in order to arrive at the desired final design it is necessary to start with a preliminary design procedure.

Preliminary design procedure. The main objective of this phase is a design which is as close as possible to the desired final design. As illustrated in Fig. 30, the preliminary design phase consists of three main groups of steps: (1) preliminary analysis, (2) preliminary design, and (3) analysis of preliminary design.

1. Preliminary analysis. The objective of this first group of steps is the establishment of the design criteria and the estimation of the acceptable maximum fundamental period (T) and the design forces (critical combinations among all of the forces that can be induced). In the conceptual methodology, preliminary analysis of the design problem can be formulated thus:

**GIVEN:**
- Function of building
- General configuration of the building, structural layout, structural system, structural materials and nonstructural components and contents
- Gravity, wind, snow and other possible loads or excitations
- SLEDRS, and SIDRS for EQGMs expected at service and safety limit states

**REQUIRED:**
- Establishment of the design criteria, the minimum stiffness (or maximum T) acceptable to control the damage (maximum deformation of the building), the design seismic forces and the critical load combinations

**SOLUTION:**
- Based on a transparent approach that takes into account from the beginning that: the structure is a Multi-Degree-of-Freedom System (MDOFS); there can be important torsional effects even under service EQGMs (i.e., in the linear elastic response) and that for safety EQGMs these effects can be different; and it is necessary to consider the desired damage index (control of damage) for selecting appropriate μ that can be used as well as the expected overstrength.

Figure 31 shows a flow chart of the steps involved in the preliminary analysis to estimate the design seismic forces. Note that although the main purpose of this step is analysis of the problem involved [i.e., what is given (general configuration, structural layout, structural system, structural material, and nonstructural components and contents), what is known (SLEDRS and SIDRS for service and safety limit state), and what is needed (to find the maximum period and design forces)], it is clear that a preliminary design of member sizes is in fact necessary in order to satisfy the deformation demand and obtain the fundamental period of the building, T1, to be used for estimating the design forces. A more detailed description of the problems confronted by the designer in carrying out this preliminary analysis is offered by Bertero R. and Bertero V. [1992].

2. Preliminary Design. The preliminary design step (assuming that preliminary sizing for stiffness was done in the preliminary analysis phase) can be as follows for a RC building:

**GIVEN:**
- Gravity, wind, snow and seismic-design loads for service and safety limit states; critical load combinations; mechanical characteristics of the structural and nonstructural materials

**REQUIRED:**
- Beam and columns sizes and their flexural reinforcement

**SOLUTION:**
- Based on the application of linear optimization theory, the beams and columns of each story are designed to minimize the volume of flexural reinforcement, using practical requirements as well as the service bending moments as constraints so that
the preliminary design simultaneously considers the demands for serviceability and safety.

3. Analysis of preliminary design. This may be stated as follows:

**GIVEN:**
- General configuration of building, structural layout, structural systems, structural material and its mechanical characteristics, and nonstructural components and their materials and mechanical characteristics;
- beams and columns sizes and flexural reinforcement;
- design EQs, critical load combinations and possible critical EQGMs for service and safety limit states.

**REQUIRED:**
- A determination of the acceptability of the preliminary design, i.e., check whether it satisfies the established design criteria.

**SOLUTION:**
- Check inter-story drift index (IDI), stress-ratios, shear stresses in the members and joints, adequacy of the foundation, and local damage index (DM) under critical GMs for each limit state using static and dynamic load analyses.

6.3.3 Discussion of the preliminary design procedure.

**Preliminary analysis.** In the numerical design of a structure, one of the most important data for attaining a reliable design is the reliable quantification of the excitations against which the structure is to be designed. Thus, the proper selection of the design EQs is an important and at the same time one of the most difficult tasks in efficient preliminary EQRD of a structure. As summarized above under “Establishment of design EQs,” it is necessary to compute a series of spectra. At present most of these spectra are computed just for SDOFS. Because a real building generally is a MDOF, as is indicated in Figure 31 it is necessary to modify the obtained SDOFS spectra. Furthermore, additional modifications are necessary from the beginning of the EQRD process to include the effects of torsion and overstrength (which in turn depends on the design method). These modifications are necessary in order to arrive at a preliminary design that is as close as possible to the desired final design.

- Modifications to account for design method. Because in EQRD the critical regions of the members and therefore the members themselves are usually provided with a large local μ, structures designed using elastic methods usually result in strength higher than that for which they were designed. The result is a designed and usually a constructed structure with significant lateral overstrength with respect to code-required strength. This overstrength varies with the $T_1$ of the structure. The larger the building (the larger $T_1$) and the fewer structural bays, the smaller the overstrength will be. It should be noted that if the designer tailors the main reinforcement so that all of the critical regions of each of the members reach their demanded strength simultaneously, the resulting structure’s overstrength will be reduced. Thus it is not only difficult, but even dangerous, to attempt to codify just one constant value for such overstrength. Similarly, if the designer uses the ACI code strength method with the redistribution due to plastic deformation allowed by this code, the overstrength will also be reduced. If the designer uses inelastic design method, i.e., methods that account for plastic redistribution of the internal forces that are demanded elastically from the structure, the resulting overstrength will decrease. However, the degree of decrease depends on how the design is conducted. If the design is based on optimization, proceeding as those suggested herein, the overstrength will be reduced to a minimum, but can still be significant. The actual dynamic overstrength during the dynamic response to recorded EQGMs is even higher than that estimated under equivalent static load (pushover test). It should be noted that in specifying the possible reduction that can be made in the ordinates of the SIDRS for $C_0$ due to overstrength, it is necessary to look not only to the possible overstrength, but also to its effect on maximum IDI. The increase in IDI beyond the acceptable limit can control the reduction. From this discussion it is clear that the final design will generally have a maximum yielding strength larger than that required by the adopted yielding strength spectra ($C_0$), and therefore that the response ordinates of such SIDRS can be reduced by a factor $R_{OVS}$. The problem is, how much can the value of $R_{OVS}$ be? As this value depends on many variables (design method, dynamic effects, $T_1$ and $T_1/T_p$, etc.), at present its selection requires seasoned judgement and should be done conservatively, at least until needed research produces the required calibration data for its proper selection.
Modifications due to torsional effects. Due to torsion, the demanded strength and IDI at certain parts of the structure increase over those required by just the translational deformation. The larger the eccentricity between the center of rigidity and the center of mass, the larger the effects of torsion. These effects will be different under service EQGMs from what they would be under safety EQGMs. At the safety level involving inelastic behavior, the effects of torsion can significantly increase depending on the initial location of the center of torsional resistance (strength) and how this center and the resulting yielding resistance eccentricity are changing in the yielding of the structure [Sedarat and Bertero, 1990]. The torsional effects are currently taken into account in the estimation of the seismic design forces by modifying the distribution of the computed total design base shear and average IDI obtained from the design spectra, or even later, in the analysis of the preliminary design. As illustrated in Fig. 31, in the proposed conceptual methodology the torsional effects are considered from the beginning by direct modification of the value obtained from the spectra. A series of practical equations for doing this have been developed [Bertero, R. and Bertero, V. 1992].

- Preliminary design. As was briefly stated in the summary of the preliminary design steps, in order to attain the required sizing of the structural elements and the amount of their reinforcement it is proposed to do this story by story, starting from the roof, and to use plastic design and optimization theory, using as an objective function the minimization of the flexural reinforcement, and introducing as constraints all practical requirements as well as the axial-flexural strength required by the service design EQ, so that the preliminary design will satisfy the demands imposed by both the service and the safety design EQs.

- Analysis of preliminary design. As indicated in the flow chart of Fig. 30 and in the summary presented previously, the main objective of this step is to determine if the preliminary design is acceptable according to the adopted design criteria. To do so it is necessary to conduct a series of analyses of the demanded values for the main response parameters used in the establishment of the design criteria, considering the main limit states through which the whole facility system could pass during its service life. The main parameters are: total weight (W); IDI; the stress ratio shear stresses in the members and their joints; the global, story and story local μ; and the local damage index (DI). These should be done for the critical EQGMs for each different limit state considered in the design criteria. To carry out the above checks it is necessary to conduct linear and nonlinear analyses. The ideal would be to conduct a 3-D dynamic analysis which would consider all of the components of the critical EQGMs acting simultaneously, or at least their two horizontal components. While at present this can be done easily for the serviceability limit state because it requires the use of dynamic linear analysis, checking the safety limit state is more difficult because of the lack of reliable 3-D dynamic nonlinear analysis computer programs that can be used in practice for multistory buildings. The use of available general 3-D dynamic nonlinear computer programs demands the use of large computers and significant amounts of computer time. Therefore, at present, attempts should be made to conduct the 3-D analysis using the static lateral load (pushover) method with a proper load pattern, or, even better, considering the probable bounds of such a pattern. If 3-D analysis programs are not available or can not be used because of the powerful computers required or computer costs, then attempts should be made to use Pseudo-3-D programs. The disadvantages of these Pseudo-3-D programs, such as DRAIN-2DX, is that they cannot estimate the effects of torsion and of multidirectional input.

Recent studies note that while the static pushover method can give an idea of the first (local) and maximum (global) yielding strengths, it can significantly underestimate the actual dynamic global yielding strength. Furthermore, the results of such pushover analysis can significantly underestimate the local cumulative plastic rotation and will not reveal the possibility of a shakedown problem, particularly in very slender, tall RC buildings. Thus, efforts should be devoted to developing practical and reliable 3-D dynamic nonlinear analysis computer programs that will enable 3-D time-history analyses to be conducted on the response of the buildings when subjected to the time-history components of the critical EQGMs.

6.4 Concluding remarks

The conceptual methodology for EQRD that has been summarized herein has been developed for the main purpose of attaining a transparent procedure, based on fundamental principles of structural dynamics and comprehensive design, that can be used as a basis for the formulation of EQRD procedure for a new generation of seismic codes which SEACO envisages for the year 2000. It has been applied successfully to the design of a 30-story RC building which has also been designed using the 1991 edition of the UBC [Bertero, R. and Bertero, V., 1992]. Comparison of the two designs shows the weakness of present UBC regulations when applied to tall buildings. It should be clearly noted that it is not the intention to propose that such a methodology as has been presented be implemented in the code for all kinds of structures. What has been proposed is that such a conceptual methodology be used for the design of irregular buildings, tall buildings, and particularly buildings on abnormal sites. For regular buildings located on standard sites, this methodology can be used to carry out case studies whose results can be used to formulate seismic code regulations that are simpler, but still more transparent and reliable, than present ones.

7 CONCLUSIONS, RECOMMENDATIONS AND PLEAS

7.1 Conclusions

- Earthquakes are a very special type of natural hazard in the sense that they are very rare, low-probability events that can result in disasters in which most of the human and economic losses are due not to the EQ mechanism (fault rupture) but to the failures of human-made facilities. Thus, although EQs are inevitable, it is in our hands to reduce their consequences to acceptable limits by controlling the built environment. This should be the key point in the
formulation of an EQ preparedness program.

- Analysis of what happened during 1988-1992 shows that even though the seismic activity (seismicity) of any urban area remains constant, and in spite of the impressive increase in knowledge in the field of EQ Engineering, the seismic risks in our urban areas have increased rather than decreased. This increase is due to the rapid and uncontrolled growth in population, urbanization, and the development of high technology industries in our urban areas has not been counterbalanced by a needed increase in EQ preparedness.

- We are far from the main goal of the work of the IAEE, which is the reduction of seismic risks in our urban and rural areas. It is taking too long to transfer knowledge gained through research and experience to the practitioners working in EQRD and EQRC of the facilities of the built environment and to the government officials in charge of the management of EQ hazard reduction programs.

- Review of what has happened during recent significant EQs reveals that either we very quickly forget the lessons learned in previous EQs or we do not take seriously their warnings regarding: (1) where not to build, and how to build so that new facilities will not fail; and (2) the urgent need to perform reliable assessments of the vulnerability of facilities similar to those that have been observed to fail.

- The issue of EQ preparedness: "The poorer the EQ preparedness, the greater the EQ disaster." The ideal solution of the EQ problem will be through prediction of the event and hazard reduction through preparedness. Prediction research should be continued, but should not interfere with efforts to solve the present and pressing problems of our built environment through the formulation and implementation of adequate EQ preparedness programs.

- The main issue confronting all of us interested in EQ Engineering is the need to control the seismic risks in our urban and rural areas. The solution lies in controlling the vulnerability of the built environment, because this allows us to control the potential sources of EQ hazards, which are consequences of the interaction of seismic activity (which we cannot control) with the vulnerability of the built environment.

- From analysis of the aspects involved in the assessment of seismic risks and the experts needed to perform such assessment, it is clear that control and reduction of seismic risks in any given urban area is a complex problem requiring the integration of knowledge and the collaboration of experts from many disciplines.

- The problem of seismic risk reduction will not be solved just by the acquisition of the required knowledge through research. Research must be accompanied by the necessary technological developments and the implementation of the knowledge and the developments in practice. What is needed is a translation of current engineering and architectural know-how into simplified options which can answer the socio-political and economic concerns. This will require not only a multi-disciplinary approach, but also a comprehensive educational program, not only for owners and future users but also for all of the different audiences that in one way or another are involved in the implementation of seismic risk reduction measures.

Good government is the key factor in preparedness, and therefore government performance is a major controllable factor influencing the impact of a disaster.

- There have not been serious attempts to integrate the knowledge and the requirements of the various disciplines involved in the general problem of EQ hazard reduction. The engineers are perhaps not asking the geoscientists the right questions, and the geoscientists are perhaps more interested in trying to predict EQs than in the problems involved in the EQRD of facilities. These attitudes have to be changed.

- The most effective way to mitigate the destructive effects of EQs is to improve and develop more reliable methods than those presently used for designing, constructing and maintaining and monitoring new structures and seismically upgrading existing hazardous facilities. Therefore, EQRD is at present the key element in coping with the problems of EQ hazard reduction.

- The principal issues that remain to be resolved for the improvement of the EQRD of structures are related to the following three basic elements: EQ input, demands on the structure, and supplied capacities to the structure.

- The EQ input element involves the following interrelated issues: design EQs, design criteria, and selection of design methodology.

- Design criteria should reflect in a transparent way the general philosophy of EQRD, which has been well established and is accepted world-wide. EQRD in practice generally follows seismic code provisions. Unfortunately, most present code EQRD methods fall short of realizing the goals and objectives of the general EQRD methodology.

- Most current seismic codes are primarily intended to safeguard against major failure and loss of life, and not to limit damage, maintain function, or provide for easy repair. In few words, current code design methodology is based on a one-level design EQ. Moreover, "the protection of life is reasonably provided, but not with complete assurance." The time has arrived to move from the current code one-level design EQ methodology to a code design methodology based on at least two distinct levels of design EQs: the service-level (functional adequacy) and the life-safety level EQs.

- Modern building codes, which try to reflect great advances in knowledge and understanding in a very simple way, are not transparent about the expected level of performance of the whole building system (soil-foundation-superstructure and nonstructural components). Expected level of performance has become an implicit, rather than an explicit, part of the codes through a series of empirical factors and detailing requirements which obscure the true nature of the EQRD problem; building performance.

- A promising engineering parameter for improving selection of proper design EQs, particularly when structural damage is acceptable and it is necessary to define the Smoothed Linear Elastic Design Response Spectra, is the energy input, $E_I$, of the EQGM, and its associated parameters.
A conceptual methodology for EQRD of facilities is proposed. It is based on well-established fundamental principles of structural dynamics, the mechanical behavior of the entire facility, and comprehensive design. It takes into account from the very beginning of the EQRD procedure (i.e., from the preliminary design on) the simultaneous demands for strength, deformation, and their combined effects on the demanded and supplied energy capacities of the entire facility system. The proposed conceptual methodology leads to a rational and transparent EQRD procedure.

The main advantage of this proposed conceptual methodology is that, notwithstanding the great uncertainties involved in the quantification of some of the concepts involved in its codification, the numerical quantification of some of the concepts can be improved without changing the format of this codified methodology as new and more reliable data are acquired.

7.2 Recommendations and pleas

- Considering that most structural engineers receive their education on the EQ problem through the code, which they have to apply in practice in the design of ordinary (standard) facilities which compose the bulk of modern construction, I believe that the decision on the part of the IAEE to undertake in the period 1980-1984, through international collaboration, the formulation of "Basic Concepts of Seismic Codes", which was accomplished with the publication of Part I, "Basic Concepts of Seismic Codes," in 1980 and Part II, "Basic Concepts for the Development of Seismic Design Criteria on Engineered Construction," in 1982, was a very good single decision. However, it is considered that the time has come to review these publications, introducing the new knowledge acquired and published in the 8th, 9th and now this 10th WCEE. The concepts of EQRD are universal. They should reflect the world-wide adopted EQRD philosophy, which is unique. Therefore, it is suggested that the IAEE, in collaboration with the United Nations as a part of this International Decade for Natural Disaster Reduction (IDNDR), appoint an international group of experts from the different fields of knowledge which are required for the formulation of the basic concepts and format in which a conceptual code for EQRD of new structures and upgrading of existing structures should be based.

Then, based on these concepts and code format together with the digestion and screening of present knowledge, an exemplary code should be formulated for a certain ideal or real seismic region.

Based on the formulated conceptual code, each country, according to its seismic hazards, the education of its engineers, its building technology and its socio-economic conditions, could decide on specific quantifications of the conceptual seismic code. The main advantages of this approach are, first, that, as the acquisition of reliable information continues, it would be easy to change the numerical values of the coefficients in the formulas given in the conceptual code so that it will not be necessary to give a new empirical approach and formulations which will confuse the engineers. Secondly, it will facilitate the interpretation of the codes for different countries, which will be very healthy for the improvement of all codes in general.

- The IAEE should do whatever is possible to encourage more practitioners and government officials working in the field of EQ Engineering to participate in the world conferences. Furthermore, an attempt should be made to find a format for the technical sessions that is better than the present format of many simultaneous sessions, in order to encourage the interchange of knowledge and ideas among the experts and practitioners from the different disciplines.

- The IAEE should encourage the development of EQ preparedness programs and of the massive educational program which is so needed for the dissemination of knowledge and the implementation of the EQ preparedness program.

The following pleas are made: firstly, to all of the experts in the different disciplines involved in solving the EQ problem, that they collaborate closely toward a timely solution to this problem; secondly, to each author of a paper to be presented to this conference, that they devote efforts to identify, among the research and/or development results that they have obtained, those that can be applied immediately in the field to reduce future EQ hazards; and thirdly, to all of the participants, that they disseminate in their communities what they learn during this conference, and collaborate in the preparation and implementation of EQ preparedness program.

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