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BUILDING CONSTRUCTION UNDER SEISMIC CONDITIONS IN THE BALKAN REGION

VOLUME 4

POST-EARTHQUAKE DAMAGE EVALUATION AND STRENGTH ASSESSMENT OF BUILDINGS UNDER SEISMIC CONDITIONS

BUILDING CONSTRUCTION
UNDER SEISMIC CONDITIONS
IN THE BALKAN REGION

UNDP/UNIDO PROJECT RER/79/015

VOLUME 4

POST-EARTHQUAKE DAMAGE EVALUATION AND STRENGTH ASSESSMENT OF BUILDINGS UNDER SEISMIC CONDITIONS

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION
executing agency for the
UNITED NATIONS DEVELOPMENT PROGRAMME
Vienna, 1985

VOLUME 4

POST-EARTHQUAKE DAMAGE EVALUATION AND STRENGTH ASSESSMENT OF BUILDINGS UNDER SEISMIC CONDITIONS

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UNIDO Backstopping Officer	Emanuel Csorba

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PREFACE

The Regional Project "Building Construction under Seismic Conditions in the Balkan Region", UNDP/UNIDO RER/79/015, has been carried out with the participation of the Governments of Bulgaria, Greece, Hungary, Rumania, Turkey and Yugoslavia, and with the United Nations Industrial Development Organization acting as Executing Agency for the United Nations Development Programme. Mr. J.G. Bouwkamp served as Chief Technical Advisor.

Within the framework of the Project, a set of seven Manuals has been produced, reflecting to a considerable extent the experience of the participating nations in earthquake resistant design and construction. These Manuals were developed by the National Delegates of the Project Working Groups, the Chief Technical Advisor and the Consultants.

The following Manuals have been prepared:

Volume 1 : Design and Construction of Seismic Resistant Reinforced Concrete Frame and Shear-Wall Buildings

Volume 2 : Design and Construction of Prefabricated Reinforced Concrete Building Systems

Volume 3 : Design and Construction of Stone and Brick-Masonry Buildings

Volume 4 : Post-Earthquake Damage Evaluation and Strength Assessment of Buildings under Seismic Conditions

Volume 5 : Repair and Strengthening of Reinforced Concrete, Stone and Brick-Masonry Buildings

Volume 6 : Repair and Strengthening of Historical Monuments and Buildings in Urban Nuclei

Volume 7 : Seismic Design Codes of the Balkan Region

ACKNOWLEDGEMENTS

The financial support of the United Nations and the Governments of Bulgaria, Greece, Hungary, Rumania, Turkey and Yugoslavia should hereby be acknowledged. Also the cooperation of the National Science Foundation of the USA is noted with appreciation.

The Project has been directed by the Project coordinating Committee. The membership of the Committee was as follows:

Bulgaria	G. Brankov
Greece	Th. Tassios and J. Sbokos
Hungary	F. Hunyadi
Rumania	V. Cristescu
Turkey	M. Erdik
Yugoslavia	N. Vukotic

From the United Nations the following individuals participated in the deliberations of the Coordinating Committee:

O.A. Nordstrand	- UNDP, Athens, Greece, Resident Representative and UNDP Principal Project Representative,
E. Csorba	- UNIDO, Vienna, Austria, Senior Industrial Development Officer, and
J.G. Bouwkamp	- UNIDO, Thessaloniki, Greece, Chief Technical Advisor.

DISCLAIMER

The material contained in these Volumes includes detailed findings in earthquake engineering - particularly, objective evaluations of causes and effects in earthquake damage - and in the seismic and geologic characteristics of the physical environment.

The material reflects the interpretation and opinions of the authors and contributors and does not necessarily represent the viewpoint of the United Nations Industrial Development Organization, the United Nations Development Programme, the participating Governments and the National Science Foundation of the USA. The above mentioned Governments and Organizations - while providing for the presentation of these Volumes in the public interest and for their obvious informational value - assumes no responsibility for any views expressed therein.

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NOTE

Post-earthquake damage evaluation and assessment of building safety is a prerequisite for emergency measures necessary for mitigating the consequences of damaging earthquakes as well as saving human life from possible aftershocks. Pre-earthquake assessment of seismic strength of buildings will be made part of rehabilitation programs that local or other authorities may carry out in regions of high seismicity and particularly in those where there are long-term predictions for high probability of occurrence of a strong earthquake.

This manual combines technical descriptions of seismic mitigation measures for structures in the Balkan region, with material and discussion on policy issues surrounding the relevant programs. The complexity of the problems addressed and the differences in socio-economic conditions in the participating countries did not allow for developing a true engineering manual on the subject. However, the material presented can be used as the basis for the preparation of manuals or guidelines by individual countries.

Considerable effort and time was spent to adopt uniform procedures for emergency post-earthquake damage inspection of buildings. To this end, a consenses was reached by all participating contries and a common "Emergency Earthquake Damage Inspection Form" has been formulated and adopted along with damage and usability classification catagories and descriptions.

Adoption of this form and the procedures recommended herein for regional use, will enhance the potential for greater cooperation between the Balkan countries, not only in the event of a major earthquake disaster - in which case technical assistance within the region could be easily utilized - but also by creating wider data bases for vulnerability studies of building ypes common to the region. For these reasons it is strongly recommended here, that these forms be adopted by the individual countries and advance training programs of professionals in the subject of post-earthquake damage inspections be initiated.

The Working Group consisted of national delegates of the participating countries with Dr. G. Serbanescu, Research Engineer, INCERN, Bucharest, Rumania, serving as Convenor. Other members of the Working Group were: Dr. Nuri Akkas, Associate Professor, Middle East Technical University, Ankara, Turkey; Dr. Stavros Anagnostopoulos, Director, Institute of Engineering Seismology and Earthquake Engineering, Thessaloniki, Greece; Dr. Tamas Karman, Institute for Geodesy and Geotechnics, Budapest, Hungary; Dr. Branko Tozija, Institute of Earthquake Engineering and Engineering Seismology, IZIIS, Skopje, Yugoslavia and Dr. L. Tzenov, Geophysical Institute, Sofia, Bulgaria.

Consultants of the Working Group were Dr. Nicolas Laszlo, Technical Director, Design Institute for Buildings and Town Planning, ISLGC, Bucharest, Rumania; Dr. Jakim Petrovski, Director, Institute of Earthquake Engineering and Engineering Seismology, Skopje, Yugoslavia; Dr. Horea Sandi, Research Engineer, INCERC, Bucharest, Rumania and Dr. John H. Wiggins, President, J.H. Wiggins Company, Redondo Beach, California, USA.

Professor Jack G. Bouwkamp, University of California, Berkeley, California, USA, served as Project Chief Technical Advisor and participated in all Working Group discussions.

1. INTRODUCTION

This manual combines technical descriptions of seismic mitigation measures for Balkan region structures with a discussion of the policy issues surrounding seismic mitigation programs. Both post-earthquake and pre-earthquake assessment programs are discussed. Chapter 1 delineates the scope and limitations of this manual while Chapters 2, 3 and 4 discuss seismic assessment and mitigation procedures. Chapters 5, 6 and 7 focus on policy considerations, including the criteria a policy-maker may employ in establishing the dimensions of a seismic mitigation program and the various methods of determining levels of acceptable risk. Chapter 8 discusses the position of the manual in the general framework of activities of the Project, and of the current state-of-the-art of knowledge in this field.

The Appendices provide additional support to the developments of the chapter referred to. Appendix A supplements Chapter 2. Appendix B completes the developments of Chapter 3. Appendices C and D complete the developments of Chapter 4. Appendix E presents the analytical background required by the analysis of risks, by the cost-benefit analysis and decision making, as discussed in rather qualitative terms in Chapters 5, 6 and 7. Appendix F provides an example of an existing ordinance intended to reduce earthquake risks for a community.

For engineers, this manual will provide the background to make decisions on whether or not to retrofit structures and to what level of seismic capacity. For public policy makers and administrators, this manual discusses the costs and benefits of an earthquake safety program.

Rehabilitation policy procedures will first be discussed at the national and community levels so that perspectives can be developed as to which, if any, cities or communities should be targeted for seismic rehabilitation programs. These initial discussions will be of special interest to policy-makers and administrators, although engineering and geoseismic judgment will be needed even at this juncture. Subsequently, more specific procedures for determining rehabilitation measures on a building-by-building basis will be outlined. These procedures will be of special interest to engineers and building officials.

1.1 Objective and Scope

This manual has two main purposes. First, it provides methodologies and procedures for assessing building losses after earthquakes. Secondly, it provides uniform pre-disaster assessment techniques for effective seismic mitigation programs.

1.1.1 Uniform Post-Disaster Assessment Procedures

One of the primary reasons for this manual is to give uniform procedures for assessing the seismic capacity of buildings and the observed damage after an earthquake. Past studies have only marginally improved general scientific information that helps the planner and policy-maker decide what actions to pursue for seismically vulnerable existing buildings in earthquake prone regions. Through coordinated efforts, more practical, transferable data can be developed that are of potential use by many nations. Moreover, lessons learned can benefit others in use of post-disaster information, such as in deciding whether or not to continue to use a building, given the potential damage from aftershocks.

Uniform post-disaster assessment procedures can thus be used to achieve the following goals:

1. reducing deaths and injuries to the occupants of buildings that have been weakened or placed in jeopardy by seismic activity and are threatened from subsequent aftershocks;

2. saving damaged or weakened structures by identifying emergency strengthening needs and measures;

3. recording damages for subsequent repair and strengthening and thus providing the basis for allowing use of as many buildings as possible, as soon as possible, and at an acceptable level of risk;

4. providing information to support emergency efforts that will help to identify reconstruction priorities, indicate transportation routes that may be dangerous because they are lined with damaged buildings, and indicate safe temporary shelter sites and hospitals, etc.;

5. properly and uniformly assessing loss in economic, social, political, and other terms so that loss estimates may be useful both for local earthquake rehabilitation programs and also for those proposed elsewhere;

6. developing seismic vulnerability relationships for pre-earthquake assessments so that sound mitigation programs elsewhere can use data developed;

7. correctly and uniformly assessing the nature of damage so that potential rehabilitation plans may incorporate such assessments;

8. providing information for practical research studies aimed at assessing mitigation alternatives, and leading to code and seismic hazard map revisions etc.

1.1.2 Uniform Pre-Disaster Assessment Procedures for Effective Mitigation Program

This manual also provides uniform and cost-effective tools for assessing the ability of buildings to withstand seismic shaking.

Pre-disaster assessment procedures are useful for:

1. helping engineers analyze the seismic vulnerability of buildings;

2. defining specific methods for making seismic assessments of buildings on a building-by-building basis if and when a community rehabilitation program is undertaken;

3. evaluating the cost, accuracy, and reliability of each seismic assessment method so that the trade-offs between assessed costs and proposed construction measures are understood;

4. defining alternative methods for making public policy decisions on general seismic rehabilitation programs, land use planning, occupancy use measures, etc.

1.1.3 Post-Earthquake Versus Pre-Earthquake Activities

The final objectives of both post-earthquake and pre-earthquake activities are the same: to control and reduce seismic risks. The immediate objectives of the two kinds of activities are nevertheless different. Carried out under emergency conditions, post-earthquake activities are intended primarily to reduce and control the risks of losses during a short period following destructive earthquakes, in relation to buildings or other works for which a high vulnerability is assessed on the basis of survey of actual damage. These post-earthquake activities secondarily provide basic data for activities of wider scope. Pre-earthquake activities are carried out under comparatively normal conditions; these activities are intended to provide a satisfactory degree of seismic protection to a certain building stock viewed from a long-term perspective.

The efficiency of post-earthquake activities is conditioned by appropriate preparedness. In the absence of adequate preparedness measures, the efficiency and correctness of emergency measures may be jeopardized and the post-earthquake surveys aimed to collect basic information for subsequent longer-term activities may become much less efficient. It is very important therefore to plan and prepare for post-earthquake scenarios of various severities that are likely, or at least credible, to occur in a given region.

In contrast, pre-earthquake activities are typically advantaged by normal working conditions. These make it possible to properly design and plan various activities, to carry out pilot studies, and to check methodologies. These conditions make it also possible to develop consistent policies for providing a satisfactory degree of protection to various systems (urban systems, infrastructure components, etc.) considered as a whole.

However fully satisfactory in relation to the immediate risks related basically to possible aftershocks, the outcome of post-earthquake activities cannot be accepted a priori as being satisfactory from the viewpoint of the pre-earthquake strategies. Post-earthquake solutions and measures must be therefore reexamined and completed under the more normal conditions governing the pre-earthquake activities.

1.2 Manual Usage

This manual is primarily addressed to the following audiences concerned with earthquake safety:

engineers
public policy makers
and public policy administrators

Reasonable cost-effective policies in earthquake safety may require data inputs from a variety of other people, such as

seismologists,
engineering geologists,
geotechnical engineers, and
building contractors

1.3 Definitions of Relevant Earthquake Policy Terms

1. Acceptable Risk - a probability of occurrence of social economic consequences from earthquakes that is sufficiently low (for example in comparison to other natural or man-made risks) as to be judged by appropriate authorities to represent a realistic basis for determining design requirements for engineered structures, or for taking certain social or economic actions.

2. Assessment (of some characteristics) - the integrated analysis of some characteristics of a system or activity and of their significance in an appropriate context (e.g., assessment of hazard, vulnerability, exposure, risk). It incorporates estimation and evaluation of characteristics referred to.

3. Class of Buildings or Structures - means a set of buildings or structures that are sufficiently similar in order to make sure that a statistical analysis of their characteristics or performances (e.g., statistical damage distribution) is significant.

4. Damage - any adverse consequence for the physical state of a building or building component caused by earthquakes. The damage inflicted to structural components and connections is referred to as structural damage, while the damage inflicted to non-structural components is referred to as non-structural damage. Damage may be apparent or hidden. Apparent damage may be quantified by means of a certain methodology.

5. Earthquake - a sudden motion or vibration in the earth caused by the abrupt release of energy in the earth's lithosphere. The motion may vary from violent at some locations to imperceptible at other locations.

6. Elements at Risk - the population, properties, and the economic activities (including public services) at risk in a given area. Often the term "exposure" is used, and one must be aware of possible double counting (in the course of a day, a person may be exposed to hazards in several buildings).

7. Estimation - (of a parameter) - the modeling and analysis leading to a quantitative characterization of a certain parameter (e.g., estimate of vulnerability or risk).

8. Evaluation (of some characteristics) - the appraisal of the significance of a given quantitative (or, when adequate, qualitative) measure of some characteristic of a system or activity, as for example, the comparison of the expected number of fatalities per year from a specified system operation, with that from a number of other, generally "accepted" causes; the appraisal of the risk of such fatalities in relation to the socio-economic benefits of its acceptance; or the appraisal of the risk in relation to the cost of its mitigation.

9. Exposure - the potential losses, expressed in economic terms, in terms of deaths or injuries, etc., representing the consequence of earthquakes, due to damage occurrence, to effects on the natural environments, or to adverse chains of events. The potential losses depend on the occupancy rates, on the value of property exposed, on the value or criticality of equipment, etc.

10. Exposure Time - the time period of interest for seismic risk calculations, seismic hazard calculations, or design of structures. For the latter case, the exposure time often equals the design lifetime of the structure. For a seismic safety rehabilitation ordinance, the exposure time will refer to the time before rehabilitation must occur, unless the building is demolished.

11. Hazard - the probability of occurrence within a specified period of time, in a given area, of a potentially damaging natural phenomenon.

12. Intensity - a measure of the severity of seismic ground motion at a specific site (e.g., MSK intensity, Modified Mercalli intensity, spectral intensity, peak acceleration or velocity, root mean square acceleration, spectral acceleration or velocity, effective peak acceleration, or velocity, equivalent ground acceleration, etc.).

13. Liquefaction - the transformation of unconsolidated or poorly consolidated water-saturated granular material (such as silt or sand) into a liquefied state (often caused by earthquake).

14. Loss - any adverse economic or social consequence caused by an earthquake.

15. Maximum Credible, Earthquake - the earthquake that would cause the most severe ground motion capable of being produced at the site under the current known tectonic structure.

16. Maximum Expected Earthquake - this term is used rather loosely to designate the largest earthquake that can be reasonably expected within a specified period of time.

17. Observed Vulnerability - vulnerability as derived from post-earthquake surveys and statistical analysis, for some definite types of buildings or structures.

18. Predicted Vulnerability - vulnerability as derived from engineering analyses the results of which are to be expressed in probabilistic terms.

19. Rehabilitation - action undertaken in order to bring a structure to an acceptable level of (seismic) protection, including repair and/or strengthening. Rehabilitation programs are aimed to bring all the structures in a community, town, region to an acceptable level of seismic protection.

20. Repair - action undertaken in order to remove the consequences of adverse events (including seismic events), upon buildings or other kinds of works. The goal of repair is to restore the initial level of resistance of structures and the function of damaged non-structural elements.

21. Risk (seismic) - the expected losses (number of lives lost, persons injured, damage to property, and disruption of social or economic activity resulting from natural phenomena (seismic activity)); consequently, a function of specific risk and elements at risk/exposure.

    A risk profile is the complement of a cumulative probability distribution function that expresses the probability that any given loss level will be attained or exceeded.

    Individual risk is the probability of a given consequence (e.g., fatality) occurring to any member of the exposed population and essentially equals the total risk divided by the number of individuals in the exposed population.

Group or societal risk is the probability distribution for the numbers of individuals who will suffer a given consequence (e.g., death).

22. Risk Management - the process whereby decisions are made to accept a known risk or hazard or to eliminate or mitigate it. Trade-offs are made among increased cost, schedule requirements, and effectiveness of redesign or retraining, installation of warning and safety devices, procedural changes, and contingency plans for emergency actions.

23. Safety - freedom from unacceptable risk (as judged by appropriate authorities).

24. Seismic Hazard - the probability of occurrence within a specific period of time, in a given area, of any physical phenomenon (e.g., ground shaking, ground failure) associated with an earthquake which may produce adverse effects on human activities.

25. Seismic Loading - the forces or stresses induced in structures during an earthquake; expressed in units of structural response (e.g., force or bending moment), or ground acceleration.

26. Seismic Zone - a typically large area within which seismic hazard is generally uniform.

27. Seismic Zoning, Seismic Zonation - the process of determining seismic hazard at many sites for the purpose of delineating seismic zones.

28. Seismic Microzone - a generally small area within which seismic hazard has been evaluated taking into account local geological conditions.

29. Seismic Microzoning, Seismic Microzonation - the process of estimating the absolute or comparative seismic hazard at many sites for the purpose of delineating seismic microzones, incorporating such local geologic and topographic effects as soil stability and liquefaction susceptibility. Alternatively, microzonation is a process for identifying relevant geological, seismological, hydrological, and geotechnical site characteristics in a specific region and incorporating them into land-use planning and the design of safe structures in order to reduce risk to human life and property resulting from earthquakes.

30. Specific Risk - (seismic) - the expected damage owing to a particular natural phenomenon (seismic activity); a function both of the natural hazard (seismic) and vulnerability.

31. Strengthening - action undertaken in order to improve the performance of a building or other artifact (damaged or undamaged) in future earthquakes.

32. Type of Building or Structure - a technical construct (solution) fully defined from qualitative and quantitative viewpoints (a type of structure may refer to an individual structure or to a set of typified or standardized structures built according to the same design).

33. Upgrading - action undertaken in order to improve the quality of a building or other artifact, including the increase of capacity to withstand earthquakes.

34. Vulnerability (seismic, of a type of building or structure) - the degree of damage resulting from the occurrence of ground motions with specified characteristics, defined under the following conditions:

a) the type or class of buildings or structures referred to must be specified both in qualitative and quantitative terms;

b) ground motions are quantified in terms of intensities with possible reference to a range of oscillation frequencies and to direction (using for observed vulnerability the information offered by macroseismic surveys and, whenever possible, by instrumental records and using for predicted vulnerability some appropriate specification of the ground motion features);

c) the distribution of damage (quantified according to some definite methodology) is expressed in statistical or in probabilistic terms, conditional on the ground motion intensity (a system of conditional distributions for discrete representations is also referred to as a "damage probability matrix").

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2. POST-EARTHQUAKE DAMAGE EVALUATION

2.1 General

In the past twenty years, governments, engineers, and scientists in the Balkan region have made significant advances in assessing and mitigating potential earthquake problems. Despite recent progress, major earthquakes during this period have caused enormous damage to the economy of these countries. In one country, direct economic losses alone approached 15 percent of the gross national product (GNP) in a single event (Skopje, 1963) and more than 1.5 percent of GNP averaged over the period.

Earthquake protection programs are well understood and implemented with improvements in seismic zoning maps, strong-motion instrumentation networks, seismic microzonation studies of urban areas, and site-specific geoseismic studies of important projects, as well as improvements in seismic design and construction codes and regulations. These improvements have mainly been associated with new buildings that are much less voluminous than facilities designed without significant seismic resistance. For the near future, the economic potential of the Balkan countries will not likely create conditions for significant seismic risk reductions to buildings, structures, and utilities. Because the seismic hazards are serious and buildings, often old, tend to be of low seismic quality, seismic risk may even increase in Balkan countries. Significant economic damage and loss in major earthquakes are expected.

Given this high risk and expected damage, the objective of this chapter is to present a uniform procedure for examining and reporting building damage both in urban and in rural areas so that a data base on earthquake effects may be established. Also, methods are presented for analyzing earthquake damage and for estimating economic losses. Use of these procedures and methods will yield an adequate volume of data to assist community and national authorities achieve the following earthquake risk reduction program goals:

- To reduce deaths and injuries to occupants of buildings that have been weakened or seriously damaged by seismic activity and that with high probability will be subjected to a series of aftershocks within several months after the principal shock.

- To obtain appropriate information on the severity of the disaster in terms of the number of usable, damaged and also dangerous buildings so that people may be immediately protected and housed and so that essential activities may continue in the affected region.

- To develop a data base for uniform estimation of economic losses so that an appropriate rehabilitation and assistance program may be devised as the affected region is reconstructed.

- To create a data base on earthquake consequences for this and also for other seismic regions.

- To provide data so that for future earthquakes the civil defense system may elaborate rescue operation plans, train staff, and organize supplies.

- To record and classify earthquake damage so that damaged buildings may be repaired and strengthened in an orderly fashion.

- To identify principal elements of earthquake damage and to develop vulnerability relationships for different categories of buildings so that pre-earthquake mitigation programs can incorporate pre-earthquake assessments in planning and implementing short- and long-term earthquake risk reduction measures.

- To improve seismic design and construction codes and regulations as well as design and construction practice.

- To improve the scientific basis for physical planning, both urban and general, especially with respect to seismic risk reduction measures in seismically active regions.

Post-earthquake damage evaluations should be organized so that teams may rapidly use a systematic methodology. Basic information from these evaluations should enable local and national governmental authorities to make critical decisions and also to employ economically justified and technically consistent seismic risk reduction measures in a uniform manner for the entire country. If coordinated efforts are made to use the uniform methodology presented in this manual, more practical and transferable data can be developed that will be of potential use in the Balkan region as a whole as well as in other seismically active regions of the world.

Principal elements of this uniform methodology and procedure for post-earthquake damage evaluation are presented in this chapter. These elements include

- damage and usability classifications for buildings

- procedures for and organization of data collection

- earthquake damage data analysis and data bank organization

- estimation of economic losses, and human fatalities and injuries

- measures for reducing adverse earthquake consequences and for mitigating seismic risk

Connected with these principal elements are earthquake damage evaluation results and relationships as developed from illustrative studies of the April 15, 1979 Montenegro, Yugoslavia earthquake. These results and relationships are found both in this chapter and also in Appendix A.

2.2 Earthquake Damage and Usability Classification

2.2.1 Nature of Damaging Earthquake Hazards

Earthquake damage to buildings, structures and utilities results from different types of seismic hazards. The main hazards posed by earthquakes may be summarized as follows:

- Ground shaking of different severities

- Differential ground settlement, landslides and mudslides, soil liquefaction, ground lurching, and avalanches

- Ground displacements within fault zones

- Floods from dam and levee failure, tsunamis, and seiches

- Fires resulting from earthquakes.

All these hazards have occurred in past earthquakes in the Balkan region -- with the dominant influence of ground shaking and also hazards associated with soil instabilities. By far, ground shaking has been the most damaging as it causes buildings and structures to collapse partially or totally and produces damage far from the epicentral area. Ground shaking affects the soil and foundation beneath structures, and much structural damage in earthquakes is a consequence of ground failure and differential ground settlements. Sometimes the ground will lurch, particularly along roadsides, culverts, river banks, and in low-lying areas. Ground shaking can also initiate devastating rock and mudslides, which themselves can produce some of greatest disasters ever experienced from seismic causes (Peru Earthquake, 1970). A very common hazard in earthquakes is the liquefaction of sandy soil, particularly in river valleys and coastal regions. During earthquake shaking, fine-grained soil and sands, saturated by water, take on a liquid character owing to alternations in shearing stress. Water-saturated sands are so widespread, particularly in flat areas where populations tends to concentrate, that soil liquefaction and resulting damage to buildings and structures have been observed in almost every damaging earthquake. Significantly, soil liquefaction effects are very frequently associated with comparatively low accelerations of ground shaking. A much more restricted hazard comes from the surface rupture within geological fault zones. Buildings that straddle fault displacements may be critically wrenched. Elimination of this hazard is more difficult in practice and depends upon adequate building codes and the availability of special geological fault maps.

Other earthquake hazards involve water and fire. Due to undersea faulting in the Mediterranean region, gigantic sea waves (tsunamis) may rush up along the coast-line and devastate coastal properties. Floods from sudden failure of dams in earthquakes is an everpresent danger that could create enormous destructive effects sometimes larger than those produced by the ground shaking itself. Fires are potential secondary effects in modern urbanized areas with the presence of chemical industry, and oil and gas supplies. Ground shaking could cause breakage of pipe-lines, failure of oil and gas tanks, and damage to chemical industries. Explosions, release of toxic chemicals and even fires in neighborhoods or entire towns could result (Tokyo Earthquake 1923; Niigata Earthquake, 1964).

These earthquake hazards are reviewed so that earthquake damage inspection teams may differentiate the influences of different earthquake hazards on damaging effects.

2.2.2 Earthquake Damage Inspection

Earthquake damage and usability classification after moderate or large-scale damaging earthquakes should be performed based on a uniformly established methodology within the country or wider region in order to create a uniform basis for assessment of physical damage and estimation of economic losses. Uniform data sets can thereby be constructed as a means to assess possible future earthquake effects.

Earthquake damage inspection as developed within this manual is based on a uniform methodology established within Working Group D of the UNDP/UNIDO project "Building Construction under Seismic Conditions in Balkan Region" and accepted for application by Balkan countries. This methodology for earthquake damage and usability classification is synthesized in the Earthquake Damage Inspection Form and developed on the basis of the experience gathered in earthquake damage and usability classification in past earthquakes in Bulgaria, Greece, Romania, Turkey and Yugoslavia.

EARTHQUAKE DAMAGE INSPECTION FORM

SKETCH OF THE BUILDING

PLAN CROSS SECTION

ADDRESS:
OWNER:

1. TOWN (NAME):
2. BUILDING IDENTIFICATION:
   2.1 SECTION NUMBER OF CONSIDERED TOWN AREA OR SETTLEMENT:
   2.2 WORKING TEAM NUMBER:
   2.3 NUMBER OF THE BUILDING:
3. ORIENTATION OF THE BUILDING PRINCIPAL AXIS (X):
   1.NS, 2.EW, 3.N45E, 4.N45W
4. POSITION OF THE BUILDING IN THE BLOCK:
   1.CORNER, 2.MIDDLE, 3.FREE
5. NUMBER OF STORIES:
   5.1 STORIES
   5.2 APPENDAGES
   5.3 MEZZANINES
   5.4 BASEMENTS
6. GROSS AREA OF THE BUILDING (m^2):
7. USAGE (SEE DESCRIPTION ON BACK PAGE):
   7.1 BUILDING:
   7.2 GROUND FLOOR:
8. NUMBER OF APARTMENTS:
9. CONSTRUCTION PERIOD (TO BE DEFINED BY EACH COUNTRY):
   1. 2. 3.

---

10. TYPE OF STRUCTURE (SEE DESCRITION ON BACK PAGE):
11. FLOORS:
    1.R.C., 2.STEEL, 3.WOOD, 4.OTHER
12. ROOF: 1.R.C., 2.STEEL, 3.WOOD, 4.OTHER
13. ROOF COVERING: 1.TILES, 2.METAL SHEETS, 3.LIGHTWEIGHT ASBESTOS CEMENT, 4.ASPHALT PAPER, 5.HEAVY INSULATION, 6.LIGHT INSULATION, 7.OTHER
14. QUALITY OF WORKMANSHIP:
    1.GOOD, 2.AVERAGE, 3.POOR
15. TYPE OF LOAD CARRYING SYSTEM (SEE DESCRIPTION ON BACK PAGE):
    1. BEARING WALLS, 2.FRAMES, 3.FRAMES WITH INFILL WALLS, 4.FRAME WITH SHEAR WALLS, 5.SKELETON WITH INFILL WALLS, 6.MIXED, 7.OTHER (SPECIFY)
16. FIRST FLOOR STIFFNESS RELATIVE TO OTHERS:
    1.LARGER, 2.ABOUT EQUAL, 3.SMALLER
17. REPAIRS FROM PREVIOUS EARTHQUAKES:
    1.NO, 2.YES, 3.UNKNOWN

---

18. DEGREE OF DAMAGE

STRUCTURAL ELEMENTS (SEE DESCRIPTION ON BACK PAGE):
1.NONE, 2.SLIGHT, 3.MODERATE, 4.HEAVY, 5.SEVERE
18.1. BEARING WALLS:
18.2. COLUMNS:
18.3. BEAMS:
18.4. FRAME JOINTS:
18.5. SHEAR WALLS:
18.6. STAIRS:
18.7. FLOORS:
18.8. ROOF:

NONSTRUCTURAL ELEMENTS AND INSTALLATIONS (SEE DESCRIPTION IN THE MANUAL):
1.NONE, 2.SLIGHT, 3.MODERATE, 4.HEAVY, 5.SEVERE
18.11. INTERIOR WALLS:
18.12. PARTITIONS:
18.13. EXTERIOR WALLS (FACADE):
18.14. ELECTRICAL INSTALLATIONS:
18.15. PLUMBING:

---

19. DAMAGE OF ENTIRE BUILDING:
    1.1 NONE, 1.2 SLIGHT, 2.1 MODERATE, 2.2 HEAVY, 3.1 SEVERE, 3.2 TOTAL
20. INDIRECT DAMAGE (FIRE, SLAMMING,ETC.)
    1.NO, 2.YES
21. OBSERVED SOIL INSTABILITIES AND GEOLOGICAL PROBLEMS
    1.NONE, 2.SLIGHT SETTLEMENTS, 3.INTENSIVE SETLEMENTS, 4. LIQUEFACTION, 5.LANDSLIDE, 6.ROCKFALLS, 7.FAULTING, 8. OTHER (SPECIFY):

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22. USABILITY CLASSIFICATION AND POSTING:
    POSTED: 1. GREEN, 2. YELLOW, 3. RED
    NOT POSTED: 4. TO BE POSTED GREEN AFTER REMOVAL OF LOCAL HAZARD, 5. SOIL AND GEOLOGICAL PROBLEMS, REINSPECTION REQUIRED
    6. UNABLE TO CLASSIFY, REINSPECTION NECESSARY, 7. BUILDING INACCESSIBLE

GREEN	1 ORIGINAL SEISMIC CAPACITY HAS NOT BEEN DECREASED	UNLIMITED USAGE
YELLOW	2 ORIGINAL SEISMIC CAPACITY HAS BEEN DECREASED	TEMPORARILY UNUSABLE LIMITED ENTRY
RED	3 BUILDING DANGEROUS AS SUBJECT TO SUDDEN COLLAPSE	ENTRY PROHIBITED

MAIN REASONS FOR YOUR CLASSIFICATION AND POSTING:

23. RECOMENDATIONS FOR EMERGENCY MEASURES:
    1.NONE, 2.REMOVE LOCAL HAZARD, 3.PROTECT BUILDING FROM FAILURE, 4.PROTECT STREETS OR NEIGHBOURING BUILDINGS, 5.URGENT DEMOLITION
24. ADDITIONAL DATA (PHOTO/SKETCHES AND COMMENTS):
    1.NONE, 2.PHOTOS ONLY, 3.SKETCH AND COMM. ONLY 4.PHOTOS AND SKETCH AND COMM.
25. ESTIMATED PRESENT VALUE OF BUILDING (MILLIONS OF........)
26. ESTIMATED LOSS (% OF ESTIMATED VALUE)
27. HUMAN LOSSES (DEATHS AND INJURIES)
    (1) NO; (2) POSSIBLY; (3) YES
    (4) IF INFORMATION AVAILABLE PLEASE INDICATE:
    NO. OF DEATHS
    NO. OF INJURIES

OPTIONAL

28. DATE OF INSPECTION: MONTH/DAY

NAMES OF INSPECTION ENGINEERS: SIGNATURES

1.
2.
3.

7. BUILDING USAGE/IMPORTANCE CATEGORIES:

USAGE

- 10 Residential: 11 Family houses, 12 Apartment Buildings
- 20 Office: 21 Entire Building, 22 Partially
- 30 Economical: 31 Trade, 32 Finance, 33 Small industry, 34 Storage and Warehouses, 35 Agricultural, Fishing, Forestry
- 40 Health and Social Welfare: 41 Hospitals and clinics, 42 Social welfare (old people homes, invalid day-care centers)
- 50 Public Services: 51 Administrative - central or local government, 52 Police, 53 Fire station, 54 Transportation (buildings: ground, rail, air, sea) 55 Communications (buildings: postal, radio, TV)
- 60 Education and Culture: 61 Education, 62 Historical and religious, 63 Cultural and entertainment, 64 Sports (gymnasium, stadium)
- 70 Tourism and Catering: 71 Hotels, 72 Restaurants, Cafe, 73 Coffee shops, pastry shops, etc.
- 80 Industry and Energy: 81 Industrial, 82 Energy (power plant, transformer station, etc.)
- 90 Other Buildings (to be described)

10. TYPE OF STRUCTURE:

1st Digit	2nd Digit	3rd Digit
1 = Masonry	1 = No belts 2 = Horizontal belts 3 = Horizontal & Vertical belts, or Diagonal braces 4 = R.C. floors or roof	1 = Adobe 2 = Stone with no mortar 3 = Stone with mortar 4 = Solid brick 5 = Hollow brick 6 = Concrete blocks 7 = Unreinforced concrete 8 = ______ 9 = ______
2 = Reinforced Concrete	1 = Cast in place frame 2 = Cast in place bearing walls 3 = Prefabricated 4 = Mixed with masonry 5 = Mixed with steel	1 = With lightweight partitions 2 = With solid brick infills 3 = With hollow brick infills 4 = With concrete block infills
3 = Steel	1 = Heavy steel structure 2 = Light steel structure 3 = Mixed with masonry or concrete	1 = With lightweight partitions 2 = With solid brick infills 3 = With hollow brick infills
4 = Wood	1 = Wood frame 2 = Bagdadi 3 = Other	1 = ______ 2 = ______

15. TYPE OF STRUCTURAL SYSTEM:

1. Vertical and lateral loads are carried by bearing walls.
2. Vertical and lateral loads are carried by frames.
3. Vertical and lateral loads are carried by frame and infills
4. Vertical and lateral loads are carried by frame-shear wall system
5. Vertical and lateral loads are carried by columns and walls but no well-defined frames are present.
6. Vertical and lateral loads are carried by combination of walls, frames, infills and/or shear walls.
7. Other systems to be described (e.g. inverted pendulum types, etc.)

18. DEGREE OF DAMAGE: (damage category)

1. NONE: Without visible damage to the structural elements. Possible fine cracks in the wall and ceiling mortar. Hardly visible nonstructural and structural damage.
2. SLIGHT: Cracks to the wall and ceiling mortar. Falling of large patches of mortar from wall and ceiling surface. Considerable cracks, or partial failure of chimneys, attics and gable walls. Disturbance, partial sliding, sliding and falling down of roof covering. Small cracks in structural members judged not to reduce the seismic capacity of the building.
3. MODERATE: Diagonal or other cracks to structural walls, walls between windows and similar structural elements. Large cracks to reinforced concrete structural members: columns, beams, R.C. walls. Partially failed or failed chimneys, attics or gable walls. Disturbance, sliding and falling down of roof covering.
4. HEAVY: Large cracks with or without detachment of walls with crushing of materials. Large cracks with crushed material of walls between windows and similar elements of structural walls. Large cracks with small dislocation of R.C. structural elements: columns, beams and R.C. walls. Definite dislocation of both structural elements and entire building.
5. SEVERE: Structural members and their connections are extremely damaged and dislocated. A large number of crushed structural elements. Considerable dislocations of the entire building and settlement of roof structure. Partially or completely failed building.

22. USABILITY RELATED TO POSTING (JUDGMENTAL)

GREEN - Buildings posted as green (damage category 1 & 2) are without decreased seismic capacity and do not appear to pose danger to human life. Immediately usable, entry unlimited.

YELLOW - Buildings posted as yellow (damage category 3 & 4) have significantly decreased seismic capacity. Limited entry is permitted, but not usage on a continuous basis before repair and/or strenthening. Need for supporting and protection of the building and its surroundings should be considered.

RED - Buildings posted as red (damage category 5) are unsafe and subject to sudden collapse. Entry is prohibited. Protection of streets and neighboring buildings or urgent demolition may be required. In case of isolated or standard buildings decision for demolition should be based on economical study for repair and strengthening.

The methodology on earthquake damage and usability classification is directly connected with the Earthquake Damage Inspection Form enclosed on the next page. Explanations are presented on the back page of the Inspection Form. These are basic instruction materials for the inspection teams in order to perform damage and usability classification in uniform manner.

The Earthquake Damage Inspection Form is prepared in a format suitable for easy data collection in the field and also for transfer to computers for detailed analysis of relevant parameters for damage and usability classification. The following groups of parameters in the Form can be used to develop a basic data set for each building:

- Identification Parameters (1-9): These describe the location of the building within the town with corresponding town section number or settlement, the building number, and the inspection team number, the position of the building in the block and its orientation, the gross building area, the number of stories, usage and number of apartments, and construction period. When the building plan-view and cross section are sketched and the address of the building and owner are transcribed (left side of Form), then the basic identification parameters are completed.

  Town codes, (town or settlement) section numbers, number of the building, working team number, and other identification parameters together with suitable town and section maps can be prepared in advance during the training process of the inspection teams. The position of the building in the block and the orientation of the building are important both to separate possible collision effects or failure of adjacent buildings and also to examine the dominant direction of earthquake motions. Particular attention should be given to classification of usage in accordance with codes given on the back page of the Inspection Form. Construction period is an identification parameter which is left to be defined by each country; this parameter is usually connected with type of structure and quality of construction. (For the Balkan region, possible differentiations may be made as follows: 1. Before 1920 - Dominant traditional construction of adobe, stone masonry, and brick masonry; 2. 1920-1950 - Dominant construction of brick and stone masonry buildings with R.C. slabs; 3. After 1950 - Dominant construction of R.C. frame buildings and other modern types.)

- Structural and Quality Parameters (10-17): Codes for the type of structure are described on the back of the Form. The first digit describes the predominant type of structural system. The second and third digits describe subcategories of each predominant category of structural system. Additional parameters include structural systems of the floors, and of the roof, respectively, roof covering, type of load carrying system (with seven basic subcategories presented on the back page), quality of workmanship, first floor stiffness relative to other floors, and repairs from the previous earthquakes.

  All these parameters are of basic importance for classifying damage and usability of the buildings and for extrapolation of these data for economic loss analysis as well as for improvement of future seismic design and construction practice and requirements. Damage evaluation data that lead to empirical vulnerability and damage cost functions (Figures 2-1 through 2-6) should be associated with structural types and usage categories. Particular attention should be given during the training process of the in-

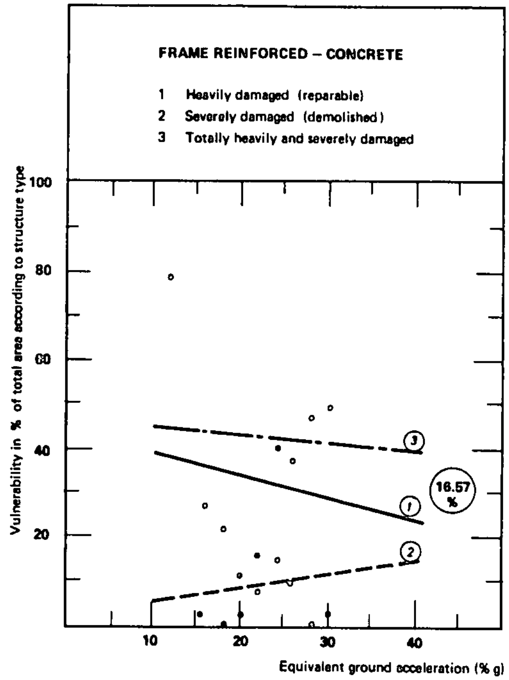
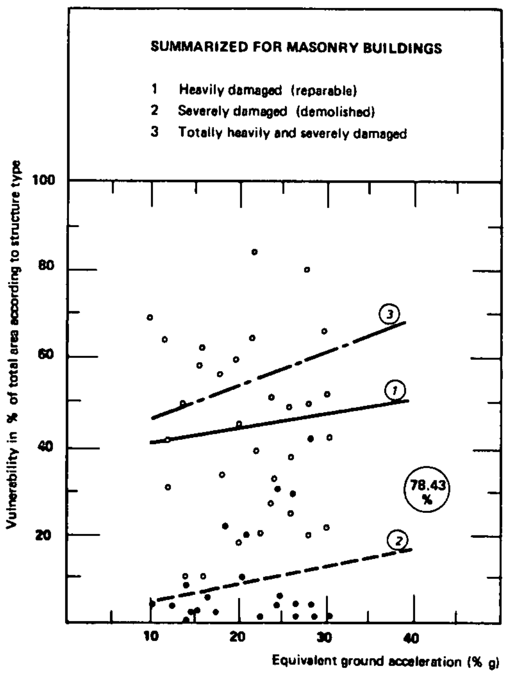


Figure 2-1. Vulnerability Empirical Functions for Reinforced - Concrete Frame Buildings (Source: Petrovski, J. et al., 1983)



'e 2-2. Vulnerability Empirical Functions Summarized for Masonry Buildings (Source: Petrovski, J. et al., 1983)

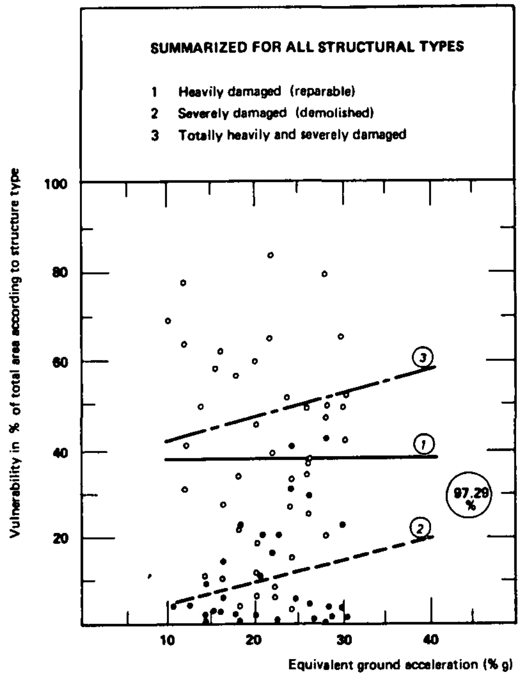


Figure 2-3. Vulnerability Empirical Functions Summarized for All Structural Types (Source: Petrovski, J. et al., 1983)

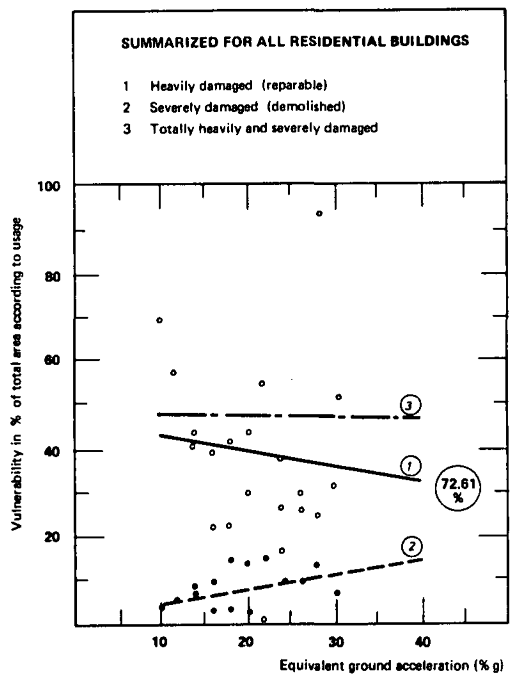


Figure 2-4. Vulnerability Empirical Functions Summarized for Residential Buildings (Source: Petrovski, J. et al., 1983)

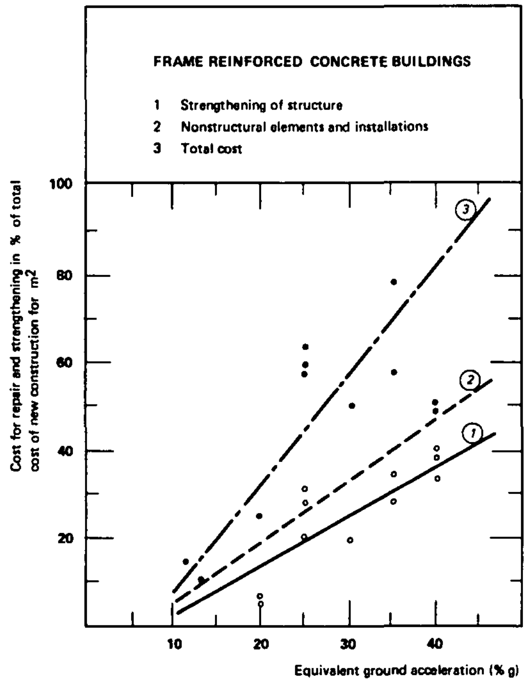


Figure 2-5. Function of Cost for Repair and Strengthening of Frame Reinforced Concrete Buildings (Source: Petrovski, J. et al., 1983)

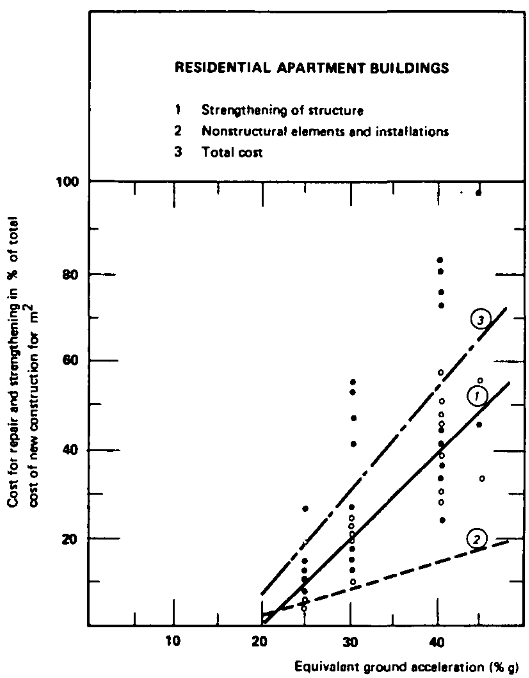


Figure 2-6. Function of Cost for Repair and Strengthening of Residential Apartment Buildings (Source: Petrovski, J. et al., 1983)

spection teams to the assessment of the quality of workmanship and relative stiffness of the floors, assessment that will be based mainly on engineering experience and judgment. Repairs from previous earthquakes are an extremely important parameter. Evaluation of previous repairs for each building should be made during training process. Use of the parameter for repairs may ultimately lead to an improved general strategy for repair and strengthening of earthquake damaged buildings or to an extremely reduced number of casualties which would otherwise occur from failure of inadequately repaired, previously earthquake-damaged buildings (Romanian Earthquake, 1977).

- Damage and Usability Classification (18-23): Damage both to structural and to nonstructural elements is described in five basic categories; damage to the entire building is described by six basic categories; damage due to local soil instabilities and geological problems is described in eight categories; indirect damage due to fire and/or collision (etc.) is described in five categories; finally, on the basis of described levels of damage usability, classification and posting is summarized in three categories related on the back page of the form to five categories of damage degree.

All these parameters are of fundamental importance for any further damage and usability classification and analysis of the entire stock of data. In the event of a major earthquake, many buildings will be damaged to varying levels and possibly a large number of them will collapse. The overriding consideration for usability classification of the buildings will depend on the damage level of the structural elements and the integrity of the structural system. Earthquake damage of the structural system is dependent on the type of load carrying system, lateral-load-resisting system, age and construction quality of the building, severity and duration of ground shaking, and such associated seismic hazards as differential ground settlements, soil liquefaction, and landslides. Since severe aftershocks may occur after a major earthquake and cause further weakening of a damaged structural system, it is of paramount importance to make an immediate damage inspection in order to assess the degree of damage and the potential of the structural system to resist further aftershock shaking. Other nonstructural elements may be damaged and removal of hazards resulting from their failure can be done within a shorter period of time. But the structural system is of primary concern to the safety of the occupants. If the structural system has been damaged, the building should be posted as being unsafe for occupancy. Damage of structural elements and posting for all five damage categories is given in sufficient details on the back page of the Inspection Form, along with comments on safety and usability of each relative to damage category.

Damage of nonstructural elements and installations should be estimated with equal care in five basic categories similar to those of the main structural system and its elements. Most damage to nonstructural elements and installations will depend on damage degree of or integrity remaining in the structural system. Examples of nonstructural damage are cracked or else partially demolished or shattered partitions, interior or exterior walls, cracked or fallen ceilings, fallen light fixtures, cracked and fallen chimneys, attics and gable walls, broken glass, dislodged mechanical and electrical equipment, broken plumbing lines and water heaters, broken gas and water lines, and elevators coming out of their guide-rails. Damage of building service systems (supplying water, gas, electricity, sanitary services etc.) may render a building unusable or dangerous. The critical need for buildings in certain usage categories (hospitals, schools, gymnasiums, cafeterias, food warehouses, power stations, transformer stations, pumping stations for water and sewage, communication facilities, etc.) to be returned to operation as soon as possible makes early evaluation of service systems (installations) almost mandatory. Damage categories of nonstructural elements and installations should follow basically the same damage categories of structural elements because nonstructural damage depends on the dislocations created in the integrity of the structural system. For very flexible structures, nonstructural damage classification could be considered to be one damage category higher than respective damage to the structural elements.

Damage to the entire building should be classified according to the same five damage categories that are used with respect to structural elements; the category of "total" may also be used for the entire building. Observed soil instabilities and geological problems, if pronounced, could be classified by the regular inspection team. In case of any doubts, the regular inspection team should require reinspection by soil engineers and geologists. Indirect damage from fire, slamming, and other causes is quite possible in modern urban regions. Thus, damage could generally be reduced significantly with protective measures and also through earthquake response training programs.

Finally, on the basis of the damage classification, usability classification and posting can be made in accordance with the description given on the back page of the Inspection Form. Posting should be made unless firm reasons exists for responses 4, 5 or 7 under posting in category 22 (Usability Classification and Posting). Explanations of main reasons for usability classification and posting should be brief and related to classifications of structural and nonstructural damage.

Possible recommendations of the inspection team for emergency measures to be undertaken include removal of local hazards primarily consisting of damaged nonstructural elements in order to make the building usable to the occupants, and also measures for protection of the streets and neighboring buildings from sudden failure of severely damaged buildings or from their demolition.

- Photographs, Other Sketches, etc. (24): Photographs should be taken on damage of structural and nonstructural elements in order to augment the existing evidence and data set on earthquake damage. Data recorded by photographs will otherwise disappear within a short period of time. The photographs will also assist the supervisors and governmental authorities in the emergency as well as being of basic importance in scientific and applied research. On the back of each photograph should be the code number of the sector or settlement and the building identification number. Photographs on nonstructural and installation damage should be taken where this damage represents a hazard to building occupants. Other sketches or background material may also be identified for purposes of scientific applied research.

- Optional Items, Estimated Losses, Deaths and Injuries (25-27): These items are optional, depending on governmental policies set for damage inspection teams. Identification of deaths and injuries is usually undertaken by health departments, the civil defense, and the military during emergency operations. Usually, earthquake damage inspection teams are organized and operational within several days after large-scale earthquakes. Thus, the inspection team may use data on human losses supplied by health departments; there is no need for the involvement of the inspection team in rescue operations. Data on human losses may be very important and are to be collected together with other data on damage and usability classification in order to develop more reliable data base for assessment of human losses relative to structural types and usage of the buildings.

  Described by the estimated loss category is percent of building damaged as a function of replacement value. This is one measure of damage that may assist officials in determining approximately how much direct economic damage has taken place. Later surveys can be used to refine estimates made here, but, unless legal difficulties or governmental prohibitions exist, the team should try to make rough estimates of reconstruction costs based on the amount and type of damage that the building has incurred.

2.3 Earthquake Damage Data Collection

In order to achieve rational and rapid performance of each inspection team, comprehensive training programs should be organized continuously within civil defense organizations of communities, districts, and the entire country. Special ordinances should be issued by local and national government authorities for implementing the described methodology on earthquake damage and usability classification.

2.3.1 Information Required and Procedure for Data Collection

Earthquake damage data collection based on the uniform methodology as described should be organized. Procedures and organizational modes should be suitable for rapid data collection, day-to-day reporting of by inspection teams, and summary data presentation to damage inspection headquarters of the sectors, communities (communes), and the entire earthquake affected region (which in the case of large-scale earthquakes could be extend to more than one community or even district). The community and regional inspection headquarters should be able to report on a daily basis to the responsible government authorities and also to prepare the final report based on inspection team reports. To achieve these needs in advance of a major earthquake, a sufficient number of well-trained inspection teams must be established in each of the communities of the seismically active regions within the country as well as in the large cities of the country. During the training process for damage inspection teams, organizational preparatory measures should be undertaken in each community and outlined in the form of the Earthquake Damage Inspection Plan which should contain the following information and materials (prepared before any earthquake):

- Topographic maps on the scale of 1:10,000 or 1:5,000 of the community, with a detailed presentation of each individual sector for inspection along with its designated code number. (One sector should cover no more than 1,000 buildings of average height in the region, since three to four inspection teams in that sector will perform damage and usability classification within one month.)

- Topographic maps of each sector on the scale of 1:1,000 with the names of the streets, the number of buildings, and the code number for each building. The code numbers of the buildings, if they are different from the regular numbers, should also be permanently and clearly marked on the buildings.

- A detailed analysis and plan of organization, and the number of the inspection teams along with the headquarters of the sectors and the community. Each inspection team should be identified by the name of the local specialists and the number of the additionally required inspection teams.

- Earthquake Damage Inspection Forms in triplicate for each building along with completed identification and structural parameters (1 through 9 and 10 through 18 on the Inspection Form). These data should be collected during the training process.

- Forms for summary and final presentation of the inspection results of earthquake damage and usability classification in three basic categories (green, yellow, and red) by number and gross area of classified buildings.

All the abovementioned maps and forms should be prepared for each inspection team and headquarters in separate files and in a format suitable for easy handling under specific field conditions after damaging earthquakes. They should be kept within civil defense headquarters or other organizations responsible for holding the training program and for actual damage and usability classification. These pre-disaster preparations are very essential for training and rapid performance of damage classification in order to ensure implementation of the uniform methodology. When these preparations are not made, a significant number of instructors and supervisors should be used for one-week training courses, during which inspection teams should make trial classifications. The most difficult problems will be the preparation of maps and forms as well as mobilization of the inspection teams under extremely difficult post-earthquake conditions.

Success of the data collection procedure depends very much on the level of preparation and training undertaken before a damaging earthquake. If these preparations take place in accordance with the recommendations mentioned above, then the following procedural data collection steps should be performed after the actual damaging earthquake:

- Mobilization of the staff of the inspection teams and headquarters.

- Distribution of the already prepared files for earthquake damage inspection to each inspection team and headquarter.

- Completion of Damage Inspection Form on a building by building basis and posting of the buildings with the color corresponding to damage and usability classification.

- Preparation of cumulative daily and weekly reports as well as final reports of each inspection team and also headquarter of the sectors and the communities.

- Submittal of the cumulative reports on earthquake damage and usability to the responsible authorities in the community, district, and country.

- Archiving of one copy of the complete set of performed damage and usability classification within civil defense headquarters of the community. Submittal of the two other copies to the regional and country headquarters responsible for further actions related to evaluation of economic losses and reduction of earthquake consequences.

2.3.2 Organization of Data Collection

Basic organization for data collection on earthquake damage and usability classification should be developed within an Earthquake Damage Inspection Plan for the community. All details on the number of required inspection teams and headquarters by sectors and the entire community should be specified based on the assumption that the entire process of data collection should be performed within one to two months after the damaging earthquake. Here, for the sake of a single basic planning organization of inspection teams, section and community headquarters will be considered together with the required equipment and also major topics of the training course.

- Organization and Duties of the Members of Inspection Team: Each earthquake damage inspection team should be composed of at least three members: a structural engineer, head of the team; a civil engineer or architect; and a technician-driver.

  The duties of the head of the inspection team are to inspect the building together with the other team members, to instruct on completion of the Inspection Form, to prepare daily and weekly reports and a final report where reports contain summary findings on all inspected buildings, to submit reports to the sectional headquarters, and to make the final decision on the posting or reinspection of the building. He is responsible for both performance and also safety of the inspection team. The second member (civil engineer or architect) transcribes on the Inspection Form and assists the head of the team in damage evaluation and preparation of the reports, takes photographs, and together with the technician takes basic measurements of the building outside of the building. The third member (technician) assists in data collection, draws sketches, takes measurements, and posts the building with its determined color. He is also the driver of the team.

  The equipment and supplies of the inspection team should minimally consist of the following: a complete inspection team file with maps and inspection forms, a hard hat for each member, a camera with an adequate supply of black-and-white film, a flashlight, notebook, hammer, measuring band, meter, and red, yellow, and green colors, and paint brushes.

- Training of Inspection Teams: The training of inspection teams should cover on the following topics: mobilization procedure, team organization, use of inspection forms and reporting procedure, on-site determination of structural systems for buildings without benefit of plans, assessment of the quality of the materials, evaluation of structural and nonstructural damage, hazard identification of nonstructural elements and of adjacent buildings with respect to usability and safety of occupancy, and temporary bracing methods.

- Duties of Sectional and Community Headquarters: Sectional and community headquarters should be directly associated with civil defense headquarters and supported by two to three structural engineers and also several technicians. The community headquarters should perform the following tasks:

- prepare the overall Earthquake Damage Plan for the jurisdiction,
- organize and execute the training program,
- compile relevant maps, forms, data etc. in advance of any disaster,
- mobilize inspection teams and also the office force,
- establish communication with sectional and regional headquarters,
- retrieve equipment and supplies from storage and distribute them to inspection teams,
- arrange transportation, food, and housing of personnel,
- organize and supervise the work both of inspection teams and also sectional headquarters,
- request protection of streets, removal of local hazards, and urgent demolition where needed,
- respond to citizens' requests for inspection,
- organize the work of reinspection and specialist teams,
- prepare reports for other agencies and for the news media,
- guide inspection teams through damaged areas, and
- archive final reports and all data on earthquake damage inspection.

The sectional headquarters should perform the following functions:

- organize the work of inspection teams in accordance with the Earthquake Inspection Plan for the corresponding section of the community
- supervise the work of the inspection teams
- prepare daily, weekly and final reports for the community, organize the work of the reinspection teams.

2.4 Organization of Data Base and Earthquake Damage Data Analysis

At the community headquarters, one copy should be maintained of the following:

- basic data sets on earthquake damage and usability classification,
- maps and final reports of inspection teams and also of sectional and of community headquarters.

Two copies of the same materials should be submitted to that governmental agency responsible for further reduction of adverse earthquake consequences, evaluation of economic losses, and planning of short- and long-term seismic risk reduction measures.

All completed basic data sets should be immediately computerized and analyzed for initial presentation in the form of tables presenting each category of usage and structural types with reference to the five basic categories of damage, usability in terms both of number of buildings and also the gross area (as a percent of the total constructed building area). These tables should be developed for each section, community. and also affected regions, respectively.

Cumulative damage and usability evaluation with the assessed seismic intensities and recorded earthquake ground motions could be immediately presented to the government authorities and the scientific community in the form of graphs and tables. These can exhibit evaluated physical damage (like the examples in Appendix A) and concentration of damage within both the community and the affected region.

2.5 Estimation of Economic Losses

From the completed earthquake damage evaluation and summary analysis of building damage, earthquake damage could be related directly with reference to structural types, usage categories of the buildings, and also gross area. For estimation of economic losses, the first strategic decision should be made on the level to which earthquake damaged buildings should be repaired and strengthened. Two basic decisions are in general possible:

- buildings should be repaired and strengthened to be aseismic structures with possible updated functioning, or

- buildings could be repaired to pre-earthquake conditions. (Many Balkan countries are implementing the first decision because large earthquakes occur often and the stock of non-aseismic construction is large.)

After the decision on the basic strategic approach is made, summary relationships on observed damage (empirical vulnerability functions) with respect to ground shaking intensity could be prepared for each usage category and structural type of the building (Figures 2-1 to 2-4). Depending on the distributions for each usage category and structural types in the total gross area, a number of representative samples will be selected for detailed cost-estimate analysis of repair and strengthening of each category and for at least five levels of ground shaking. For each of the selected sample buildings, detailed analysis of design and detailing should be performed prior to the cost-estimate analysis. Based on the analysis of a sufficient number of selected samples, functions for estimation of the cost for repair and strengthening of the structural system, nonstructural elements and installations, including improvement of the building function, could be developed similarly to those shown in Figures 2-5 and 2-6 from the Montenegro earthquake. The cost of repair and strengthening could be presented as a percent of the total cost of new construction per unit area.

Once these preliminary tasks are accomplished, summary of direct economic losses on buildings will be rather simple. In addition to building losses, direct as well as indirect economic losses for local and regional infrastructures should be assessed by specialized inspection teams.

2.6 Human Fatalities and Injuries

Human losses will range from light injuries to deaths. Naturally, human losses create the greatest concern about an earthquake and will typically characterize the extent and severity of the disaster. It is therefore extremely important to collect human loss data. These data can be used to correlate human losses to observed damage in various building types, and ultimately to build safer structures. These data and correlations may be used to make casualty predictions for emergency planning purposes. Methods for applying correlations of human losses to damage to obtain estimates of expected casualties for predisaster preparedness planning are discussed in Chapter 4 (see Anagnostopolous and Whitman, 1977).

2.7 Measures for Reduction of Earthquake Consequences and Mitigation of Seismic Risk

The primary purpose of the described methodology and procedure for post-earthquake damage evaluation is to assure an adequate volume of data with sufficient accuracy for the needs of seismic rehabilitation efforts and also seismic risk reduction programs.

Owing to recent catastrophic earthquakes in the Balkan region, severe damage has occurred to a large number of residential buildings, schools, hospitals and other public, administrative and industrial buildings, as well as other facilities in local and regional infrastructures. Most damaged buildings are in an unusable condition until their basic structural system and also nonstructural elements are adequately repaired and strengthened. In order to assure appropriate safety and normal use of damaged buildings, it will be important to recognize that these buildings will be exposed in the future to a large number of small and moderate earthquakes and, with significant probability, to the catastrophic large magnitude earthquakes similar to those in the past. So that the requirements for economic development and aseismic design may be met, systematic scientific and applied research should be carried out for the purpose of seismic risk evaluation, definition of economically justified and technically consistent design criteria, and improvement of structural systems capable of withstanding expected earthquake effects. During the stage of general and physical and urban planning, of design and construction of new facilities, as well as repair and strengthening of damaged buildings, it will be essential to take into account the expected seismic hazard and its influence on seismic stability of the structures and installations. Safety criteria should be elaborated based on determined acceptable seismic risk levels. The safety criteria will assume that seismic protection is economically justified and that future earthquake damageability levels will permit safe and undisturbed use.

For the purpose of reduction of earthquake consequences and mitigation of seismic risk, short- and long-term studies and actions should be organized by responsible government authorities and professionals. The basic steps of these studies and actions are summarized for short-term and for long-term needs, respectively.

2.7.1 Short-term Studies and Actions for Reduction of Earthquake Consequences

- Classification of buildings, structures, and local and regional infrastructures according to the usability and level of damage based on the uniform methodology developed here for damage classification.

- Planning temporary housing, organizing of medical centers, supplies, schools and other public activities.

- Studies of earthquake effects and damage distribution.

- Seismological studies based on existing and temporary installed seismic stations.

- Immediate installation of strong-motion accelerographs and seismoscopes for recording stronger aftershocks.

- Seismic records data collection and analysis for elaboration of seismic design criteria for repair and strengthening of damaged buildings and structures.

- Elaboration of requirements and instructions for repair and strengthening of damaged buildings and structures.

- Reconsideration of physical and urban plans with mapping of the spatial distribution of earthquake effects.

- Estimation of earthquake damage losses, and planning of financial and legal actions for reduction of earthquake consequences.

- Urban planning for construction of new settlements for housing, medical centers, schools and public utilities based on existing immediate needs, existing usable buildings, and anticipated future urban development.

- Actual repair and strengthening of damaged buildings and demolition of heavily damaged buildings, with associated site investigations and designs for repair and strengthening.

2.7.2 Long-term Studies for Physical and Urban Planning, Design and Construction in Highly Seismic Regions

- Improvement of the network of seismological stations with telemetered and computerized systems for rapid collection and analysis of earthquake data.

- Statistical studies on instrumental and historical seismological data and preparation in a format for rapid computer analysis used for scientific and applied research.

- Development of a neotectonic map with dynamic evaluation of the neotectonic processes within the seismic regions of the country.

- Development of a seismotectonic map of the country.

- Development of seismic hazard map of the country for different levels of seismic risk for planning, design, and construction of different types of structures.

- Development and installation of a strong-motion network.

- Physical planning of seismic regions based on damage evaluation and vulnerability studies.

- Evaluation of expected vulnerability and acceptable seismic risk levels along with requirements for seismic protective measures.

- Elaboration of codes, instructions, and manuals for aseismic design and construction of different types of structures, retrofitting of existing structures and other specific needs.
- Development of seismic microzonation maps for significant urban areas.
- Planning, design, and construction studies for structures of vital importance.
- Elaboration of laws and regulations for counter-measures against large-scale earthquakes.

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3. BASIC ELEMENTS IN THE DECISION REGARDING POST-EARTHQUAKE REHABILITATION

3.1 Purpose of this Chapter

This chapter outlines some of the main elements in deciding whether or not to rehabilitate damaged buildings after an earthquake.

It is here assumed that this decision procedure begins once a preliminary damage survey has been made. Those buildings that are undamaged will be considered in latter chapters, since such undamaged buildings are in the same policy position as pre-earthquake structures. Nevertheless, elements of the procedures explained in this chapter may also be applied both to post- or pre-earthquake decisions pertaining to rehabilitating undamaged individual buildings or else building populations. Actual technical procedures for repairing and strengthening structures are discussed in another manual, Volume 5, "Repair and Strengthening of Reinforced Concrete, Stone and Brick-Masonry Buildings."

3.2 Pertinent Results of Preliminary Damage Survey

Based on the preliminary damage survey found in Chapter 2, buildings are given the following damage ratings and typical postings:

DAMAGE TO STRUCTURAL ELEMENTS	TYPICAL POSTING
NONE SLIGHT	GREEN (USABLE AFTER LOCAL HAZARDS ARE REMOVED)
MODERATE HEAVY	YELLOW -- ENTRY LIMITED (TEMPORARILY UNUSABLE BEFORE REPAIR AND STRENGTHENING)
SEVERE	RED -- ENTRY PROHIBITED (UNSAFE WITH POSSIBLE SUDDEN COLLAPSE)

These ratings typically refer to the building immediately after the earthquake. Over time, the building condition may worsen owing to slow plastic deformations in some structural members having a new state of stress created by damage, or owing to such secondary effects as additional foundation settlements caused by soil saturation resulting from damaged water supply and drainage systems. These possible post-disaster effects must also be taken into consideration in the final strengthening design.

In addition, damage to non-structural components, such as equipment, cabinets, and bottles, may have resulted in large losses. This chapter does not directly concern itself with policy decisions on these non-structural losses.

Moreover, this chapter does not concern itself with emergency measures to be taken for structures that need to be protected against aftershocks or else effects of gravity loads immediately after the earthquake.

Instead, of special concern are structures with damaged structural systems where the following decisions may be applicable:

(1) repair or strength the structural system to a given level of seismic capacity;

(2) discontinue use of the building, or continue with a reduced use after repairs, or

(3) demolish the building, and replace it with a new building, if needed.

Hence, this chapter primarily pertains to those buildings with slight to severe structural damage and that may be able to be seismically strengthened rather than demolished. Almost no severely damaged buildings will fall into this category, since they will tend to be irreparable. This chapter is intended to include specially hazardous elements that create public safety risks. Later chapters will discuss policies pertinent to undamaged buildings or to buildings with only superficial damage.

3.3 Main Elements in the Decision Procedure

Decisions on what to do to damaged buildings have engineering, economic, social, political, and legal repercussions.

In most cases, especially for temporarily usable buildings, or for highly critical buildings that are currently unusable, decisions will need to be made fairly promptly. In addition, if numerous buildings have suffered significant structural damage, then the economics of demolishing rather than seismically rehabilitating them may become a major issue. To resume business as usual, some institutions may require very rapid decisions as to what buildings they may use. Since public safety issues are at the forefront of what to do with damaged structures, social and legal issues will also be salient. For instance, there may be a strong desire by many former building occupants not to reenter a building that has previously been structurally damaged even though its structural damage has been seismically rehabilitated. For another instance, decisions to seismically rehabilitate a structure may face adverse public and legal reaction if the same structure is later seriously damaged by an aftershock or a new earthquake. It is in this highly emotional context that the structural engineer must make his assessments and the public administrator must make his decisions.

Even though public safety issues are at the forefront of decisions as to what to do with damaged structures, risk aspects of the decision also will be present if only because earthquake-proof structures do not exist and the economics of repairing buildings will be significant. In the haste of such decisionmaking, it will here be assumed that the current seismic code constitutes the guideline for acceptable risk practices. Decisions will be made with this seismic code as a reference. In practice, reasons may be forwarded for using either higher or lower standards than the code implies.

For the policy administrator, general discussions of risk methods in later chapters, especially Chapter 6, will augment the brief discussion of policy issues here. The remainder of this chapter is devoted to the broad conceptual framework of the structural engineer as he makes his assessment of the damaged structure. In making such an assessment, he shall be assumed to convey to non-technical parties information not only on the safety of the building but also both on the technical concept and on the costs of repairing the structure or strengthening it to a given level of seismic resistance.

3.4 Technical - Economic Feasibility Assessments by the Structural Engineer

The approach of the structural engineer to the rehabilitation problem of an existing structure contains four steps:

1. Examination of the existing structure.

2. Construction of alternative rehabilitation concepts.

3. Examination of the technical feasibility and cost estimate for each alternative and selection of the final concept.

4. Final design.

This chapter deals with the first three steps. The fourth step is governed by pertinent technical regulations and is partially covered in Manual 5, "Repair and Strengthening of Reinforced Concrete, Stone, and Brick-Masonry Buildings."

3.4.1 The Examination of the Existing Structure

Structural rehabilitation of an existing building structure is unanimously acknowledged as one of the most difficult problems that the engineer has to face. His knowledge of the structure is generally very incomplete. Even if he possesses its execution drawings, he cannot be sure that all design provisions have been strictly followed. He may even find significant differences between those provisions and how they have been followed. Both the safety level of a damaged structure and also the accepted ways to demonstrate this level are not settled by any technical regulations. He therefore has to decide on his own about the safety level and how it is to be demonstrated. He has also to consider the economic efficiency of the possible rehabilitation measures.

This very complex situation has sometimes led to extreme approaches that, on the one hand, consider rehabilitation as a matter of very sophisticated analysis, more sophisticated than the design of new structures, and, on the other hand, consider rehabilitation as an art dependent only on intuition or experience. Both extremes have too little relation to the accepted and well-known provisions for new structures. This is unsatisfactory because the practicing engineer is familiar with the way of conceiving seismic safety as formulated in the codes and regulations.

Therefore, generally speaking, we think that the basic assumptions and analytic methods for the engineering problems and safety requirements of rehabilitation should be the same as those accepted in the codes for new structures of the same kind. Certainly code provisions may sometimes prove to be incomplete or too schematic to reflect the complexity of a specific rehabilitation problem. But these are special cases and should be treated as such.

It is very important to note that in the following when speaking the expression "existing structure" will be taken to refer to the structure considered as though undamaged. By the term "repair," we shall mean the simple restoration of the integrity of the visibly damaged structural members, partitions and finishes. Through this restoration the structure approximately regains its pre-earthquake seismic resistance. By the term "strengthening" we shall mean the upgrading of that resistance, including, of course also the repair of damaged members. These conventions emphasize that one rehabilitation alternative is simple repair of the structure to its pre-earthquake state, and that a distinct rehabilitation alternative is strengthening to ensure an acceptable degree of seismic protection in the future. From this perspective, the damage evalution provides only very valuable information about the seismic behavior of the structure and a guide in establishing the strengthening concept and the cost estimate of the rehabilitation.

In principle, the aim of intervention should be to upgrade the existing structure in order to fulfill the respective code requirements completely or as much as is reasonably possible. Thus, the technical examination has to establish how far the actual characteristics stray from the required ones and to what extent and by what means these characteristics could be improved in order to reach the required or at least an acceptable safety level. Seismic safety criteria as specified in codes are based on the following damage control requirements:

- prevention of important non-structural damage and significant structural damage in a moderate earthquake;
- prevention of excessive non-structural damage and important structural damage in a strong earthquake;
- avoidance of collapse in a very severe earthquake.

In general, codes contain the following specifications in order to meet these requirements:

- some rules regarding the location and distribution of the structural elements bearing the seismic loads, a possibly uniform distribution of the stiffness on the building surface, and a smooth variation of the stiffness on the height of the building;
- the values of the conventional design forces in the form of coefficients and dynamic amplification factor spectra;
- the allowable deformations (drifts), to protect non-structural elements and prevent excessive damage;
- some rules of good detailing to ensure ductile behavior and sufficient deformation capacity of the structural members in the postelastic range.

In other words, (as in Figure 3-1(a)), codes specify:

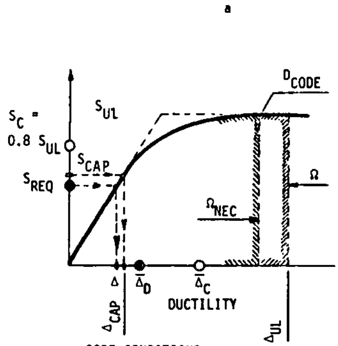
- the principles of a good structural concept;
- the necessary strength in the elastic range;
- the allowed deformability;
- the necessary ductility measures.

These four aspects mentioned above have to be examined for an existing structure (see Figure 3-1(b)).

3.4.1.1 The Quality of the Structural Design

Structures are classified from the viewpoint of the quality of their concept and layout as:

- good - the structural scheme is clear; there are redundant lateral resistant structural elements in both horizontal directions;



- CODE REQUIREMENTS
  - S_{REQ} = REQUIRED BASE SHEAR FORCE CAPACITY
  - $\bar{\Delta}_D$ = DAMAGE THRESHOLD DRIFT
  - MEASURES GRANTING DUCTILITY
- ADDITIONAL LIMITS SUGGESTED
  - S_c = 0.8 S_{ul}, CONVENTIONAL CONDEMNATION BASE SHEAR FORCE
  - Δ_c = CONDEMNATION THRESHOLD DRIFT

NOTATIONS:

D = SCHEMATIC S-Δ DIAGRAM FOR A STRUCTURE [ENVELOPE OF THE ACTUAL HYSTERETIC S-Δ DIAGRAM]

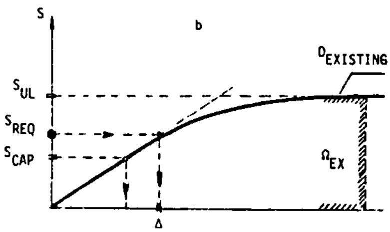
Ω = AREA ENCLOSED BY C, REPRESENTING ENERGY ABSORPTION CAPACITY OF THE STRUCTURE

CODE CONDITIONS

$S_{CAP} > S_{REQ}$

$\Delta < \bar{\Delta}_D < \bar{\Delta}_C$

$\mu = \frac{\Delta_{UL}}{\Delta_{CAP}} > \mu_{REQ.}$ OR $\Omega > \Omega_{NEC.}$



- CHARACTERISTICS TO BE ASSESSED FOR THE EXISTING STRUCTURE
  - S_{CAP} = ACTUAL BASE SHEAR FORCE CAPACITY
  - S_{REQ} = SHEAR FORCE CAPACITY REQUIRED
  - Δ = DRIFT CORRESPONDING TO S_{REQ}
  - CONDITIONS FOR DUCTILE BEHAVIOUR (ADEQUATE REINFORCEMENT AND DETAILING)

Figure 3-1. Schematic S-Δ Diagrams

a) NEW STRUCTURE - DESIGN BASED ON ASEISMIC CODE

b) EXISTING STRUCTURE

- these elements are clearly shaped as frames or shear walls;
- they are distributed with convenient spacing, on the surface of the structure, and with no excessive differences of stiffness;
- there are no significant sprung-like stiffness differences between stories; (see example in Figure 3-2);
- <u>acceptable</u> - as a whole, the structure may be classified as good, except for some weak points such as:
- evident large eccentricity of the stiffness centroid in relation to the centroid of masses;
- weak, flexible, stories in an otherwise stiff structure, stiffness concentrations at large distances, and so on (see examples in Figure 3-3);

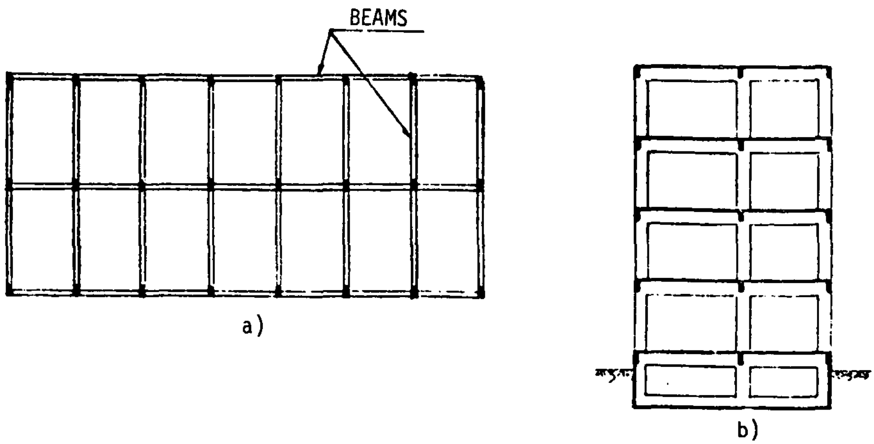


Figure 3-2. Example of a "Good" Structural Layout

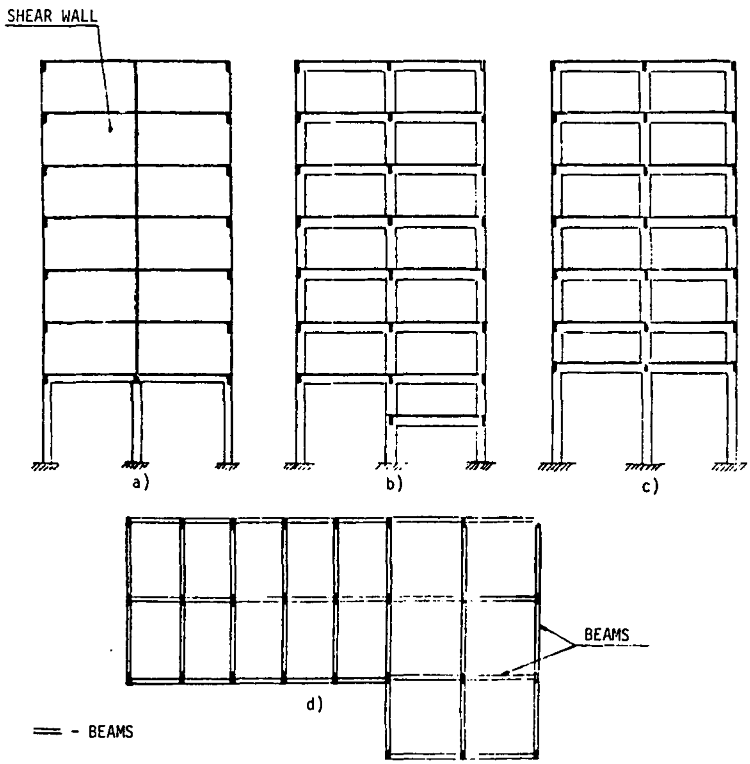


Figure 3-3. Example of "Acceptable" Structural Layouts

- unclear - the lateral force resistant elements are not clearly shaped as frames or walls, or their location is irregular, that is, the state of stress and deformation under lateral loads cannot be clearly established by usual analytic methods (see example in Figure 3-4);

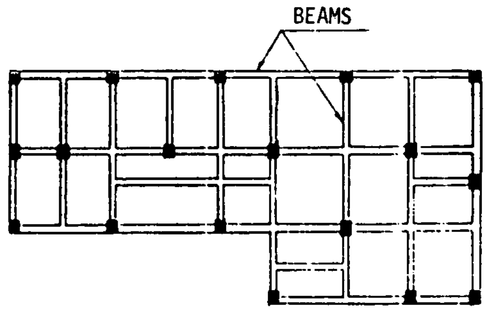


Figure 3-4. Example of "Unclear" Structural Layout

- inadequate - the structural system is eventually unsatisfactory even for gravitational loads and is obviously inadequate for lateral loads (see example in Figure 3-5).

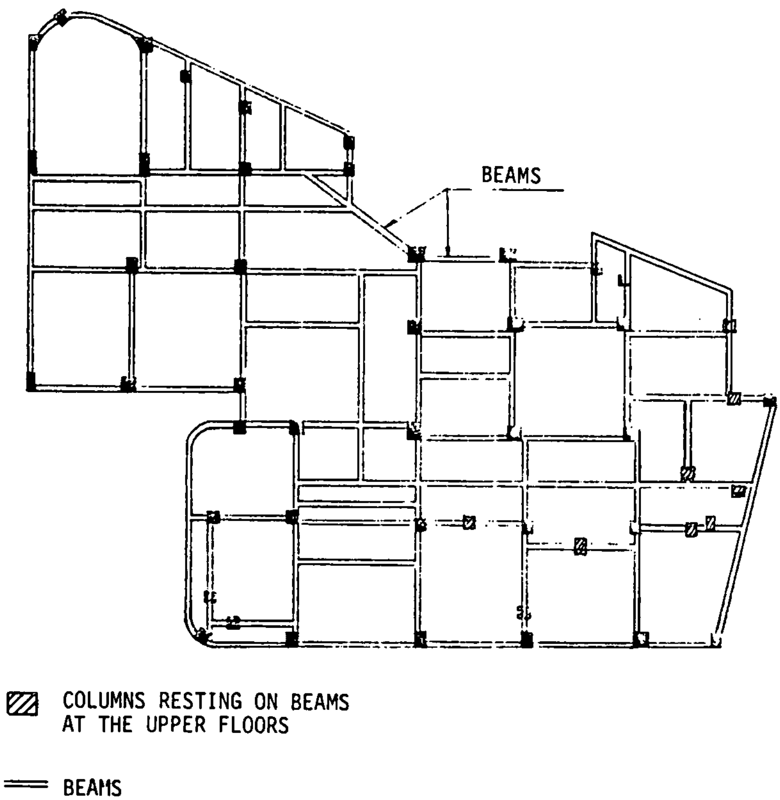


Figure 3-5. Example of "Inadequate" Structural Layout

3.4.1.2 The Strength of the Structure

The seismic performance capability of the structure is defined by Scap., the base force capacity in the elastic range, that is, the shear force which causes the first column at ground floor level to reach its design limit strength.

This shear force capacity is compared to $S_{req.}$, the required shear force capacity according to the codes for a building of the same size and function and with the same kind of structure. The ratio $R = \frac{S\ cap.}{S\ req.}$, called here the strength index, is then compared to specified limit values of this index. The limit values are to be considered as guidelines, not as strict limit conditions.

For decisionmaking needs, three intervals of values of the strength index R have been defined:

$R > 0.8$	in this interval, the strength can be considered as being satisfactory, with the probability of somewhat deeper incursions into the postelastic range during the seismic action, without coming too close to the condemnation threshold;
$0.5 < R < 0.8$	in this interval, the opportunity still exists to compensate for the lack of strength by good ductility in assuring the non-collapse condition in a strong earthquake, although this type of structure may approach its condemnation threshold. Therefore, either the structure could be accepted as it is, with a lifetime limitation depending on the seismic risk, or else the structure could be strengthened;
$R < 0.5$	in this interval, the safety of the structure is clearly unsatisfactory.

Some methods of assessing $S_{cap.}$ and $S_{req.}$ are given in Appendix B.

Combining the estimated structural layout quality and the value of the strength index R, five categories of comprehensive characterization of the actual structural quality are defined from A to E in Table 3-1.

Table 3-1. Five Classes Based on Structural Layout and the Strength Index

STRENGTH INDEX	QUALITY OF THE STRUCTURAL LAYOUT		
	GOOD	ACCEPTABLE	UNCLEAR
$0.8 < R$	A	B	C
$0.5 < R < 0.8$	B	C	D
$R < 0.5$	C	D	E

3.4.1.3 The Deformability of the Structure

The computed deformation generally in form of the drift at the ground floor level has to be compared with two limit values: Δ_D, the damage level and Δ_C, the condemnation level. Δ is the expected drift for a shear force equal to $S_{req.}$, i.e., the computed elastic deformation divided by the structural factor of about 0.2 - 0.3, as specified in the codes for each type of structure (see Figure 3-1).

The Δ_D limit, intended to ensure protection of non-structural elements, is therefore independent of the structural characteristics and has a fixed value of about 0.7 - 0.75 adopted in most codes.

Δ_C, a limit intended to ensure the protection of the structure itself, should be determined from a condemnation displacement spectrum. Since these spectra are generally not available, estimated values of

Δ_C = 1% for shear wall structures, and

Δ_C = 1.5% for frame structures

could be adopted.

3.4.1.4 The Ductility Requirements

In accordance with most current seismic design regulations, a structure is assumed to have satisfactory ductility if:

- any member has an appropriate amount of transverse reinforcement so that ductile failure in bending will occur before brittle failure by shear;
- in any member, the concrete in the compression zone is confined by adequate transverse reinforcement;
- the stiffness and ultimate strength of structural members are adequately proportioned, so that the vertical load-bearing elements, i.e., columns and walls, are the last to reach ultimate strength.

For the needs of an initial overall decision, no special ductility analysis is to be carried out. Only an estimation needs to be made of the fulfilment of these basic requirements, namely, the amount and spacing of the column stirrups confining the thin shear wall edges, and the relative stiffness and steel ratio in beams and columns at the joints of frame structures.

This estimation can be made only by examining the execution drawings. If these are not available, then only the probability of having these ductility conditions fulfilled can be estimated, based especially on the year of design and completion of the structure and the prevailing analysis and detailing concepts and also regulations at that time. Figure 3-6 provides a flow chart of the decision procedure.

3.4.1.5 A Simplified Approach for Frame Structures

The amount of analysis needed for the above procedure can be considerably simplified if the following matters are taken into account:

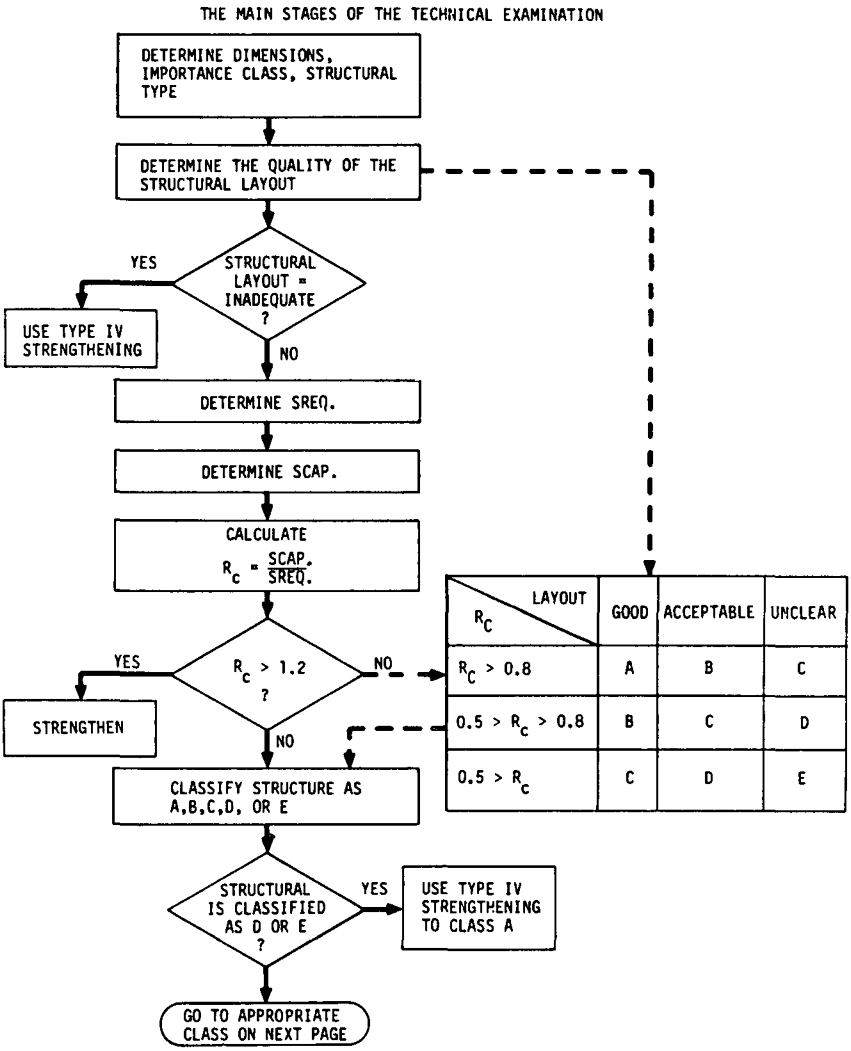


Figure 3-6. Development of the Rehabilitation Concept for Making Suitable Seismic Restoration Feasibility Studies (Page 1 of 3)

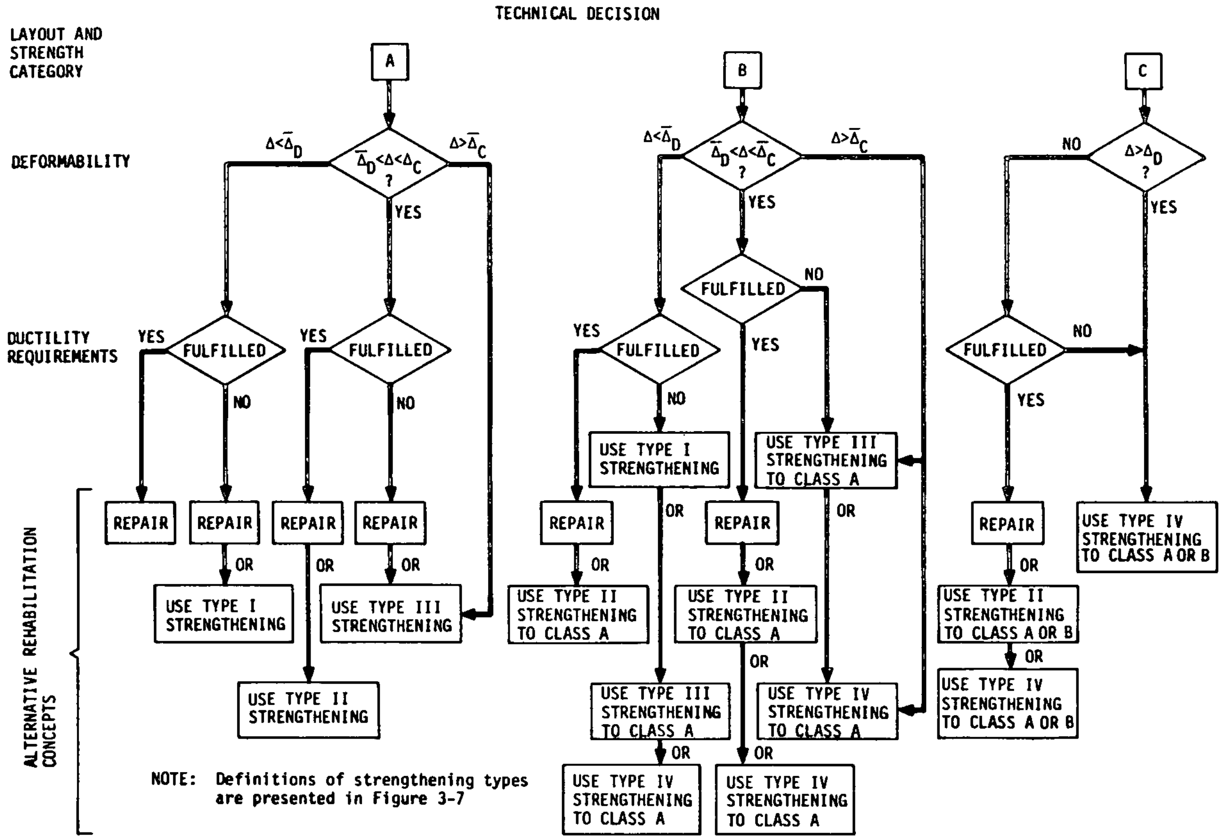
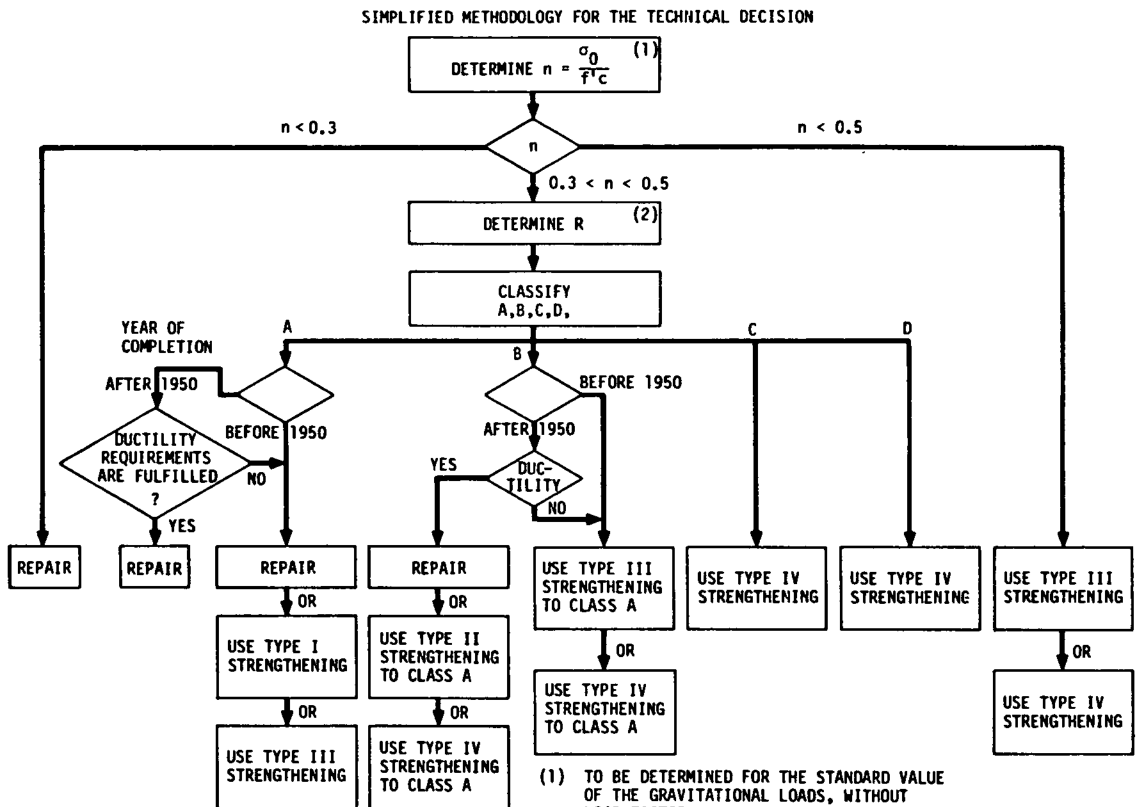


Figure 3-6. Development of the Rehabilitation Concept for Making Seismic Restoration Feasibility Studies (Page 2 of 3)



(1) TO BE DETERMINED FOR THE STANDARD VALUE OF THE GRAVITATIONAL LOADS, WITHOUT LOAD FACTOR

(2) ON THE BASIS OF DIAGRAMS AS IN Figure 3-11.

Note: Definitions of the strengthening types are presented in Figure 3-7.

Figure 3-6. Development of the Rehabilitation Concept for Making Seismic Restoration Feasibility Studies (Page 3 of 3)

1) the state of stress in a column under gravitational loads is expressed by

$$n = \frac{\sigma_o}{f_c} = \frac{N}{bhf_c}$$

where

N = the axial load;

b,h = cross-section dimensions;

f_c = the cylindrical or prismatic compression strength;

σ_o = the axial compression stress.

the ratio n, associated with the interaction diagram of the column section, allows for the direct assessment of the shear force capacity and consequently of the strength factor R, as illustrated in Figure 3-7.

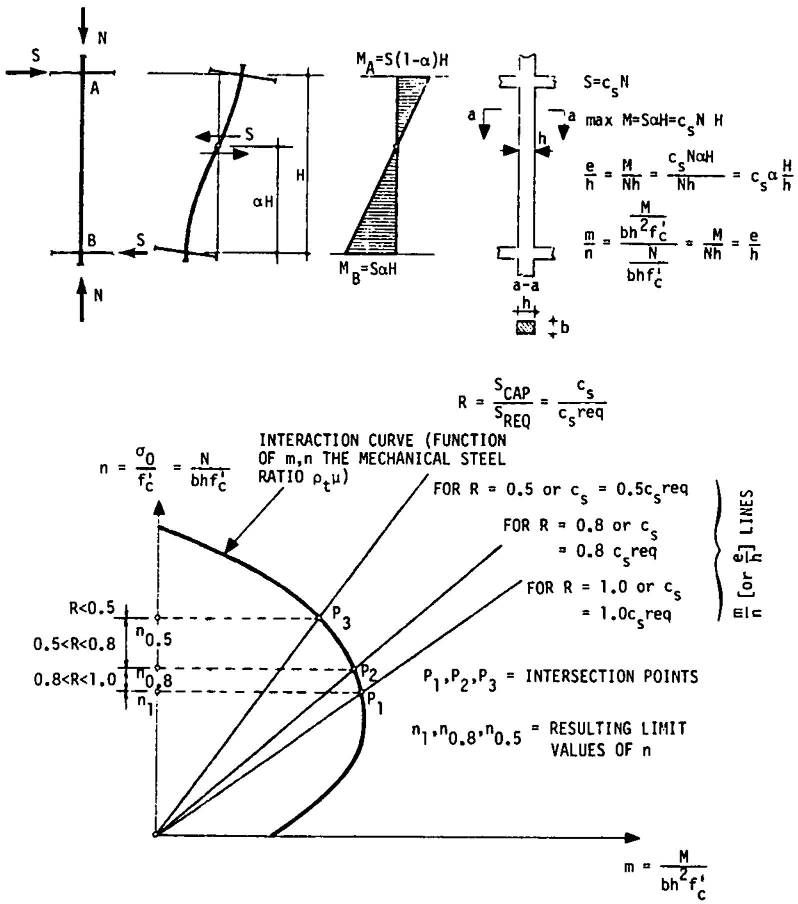


Figure 3-7. Relations Between the Axial Compression Stress, the Required Base Shear Coefficient and the Resulting R for a Reinforced Concrete Column

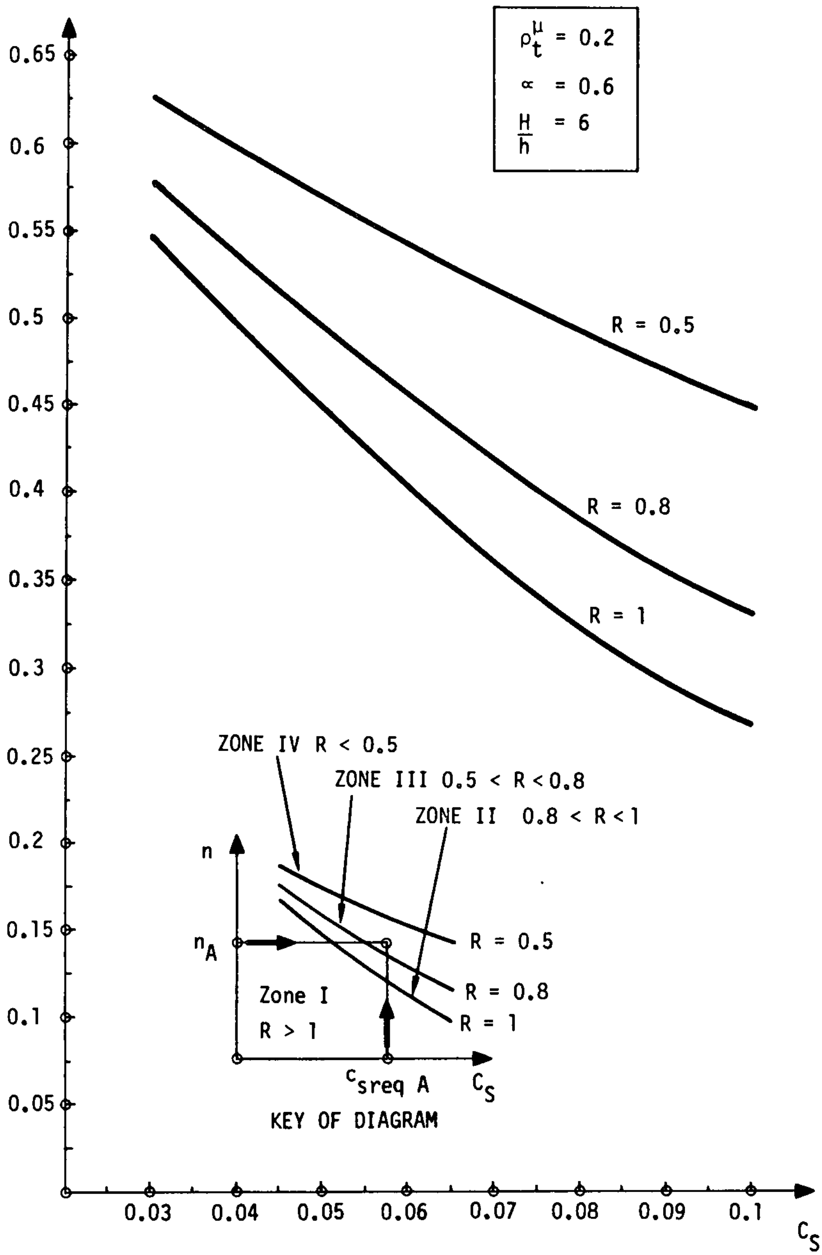

$\rho_t^\mu = 0.2$
$\alpha = 0.6$
$\frac{H}{h} = 6$
0.65
0.6
0.55
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
R = 0.5
R = 0.8
R = 1
ZONE IV R < 0.5
ZONE III 0.5 < R < 0.8
ZONE II 0.8 < R < 1
n
n_A
Zone I
R > 1
R = 0.5
R = 0.8
R = 1
c_{sreq} A
c_s
KEY OF DIAGRAM
0.03
0.04
0.05
0.06
0.07
0.08
0.09
0.1
c_s

Figure 3-8.

2) (with respect to that figure) the dispersion of the values of the factors influencing the C_s(cap.) -- the restraint degree at the columns ends, the column slenderness and the mechanical steel ratio -- is rather low in practice, so that it may be worked with average values without risk of large errors.

3) it may be assumed that there is little, if any chance that a structure designed before 1950 - 1955 has satisfactory ductility, even if the structure is well-engineered in accordance with reinforced concrete codes valid at that time. These codes allowed excessive axial compression stresses and prescribed inadequate transverse reinforcement. The importance of ductility was fully realized only later.

As a result, diagrams such as those in Figure 3-8 can be drawn, allowing for the direct determination of the strength factor R from the stress under gravitational loads and the required base shear coefficient C_s(req.) for a given column.

Such diagrams are valid for the structure as a whole if the columns have nearly the same thickness and values close to the gravitational load stress. In such cases, one may use the average compression stress,

$$\sigma_o = \frac{\text{Total weight of the building}}{\text{Total area of the columns}} = \frac{W}{\Sigma A_c}$$

and an average $n = \dfrac{\sigma_o}{f_c}$ for the entire structure.

Based on the interaction diagram in Figure 3-9, some conclusions may be drawn:

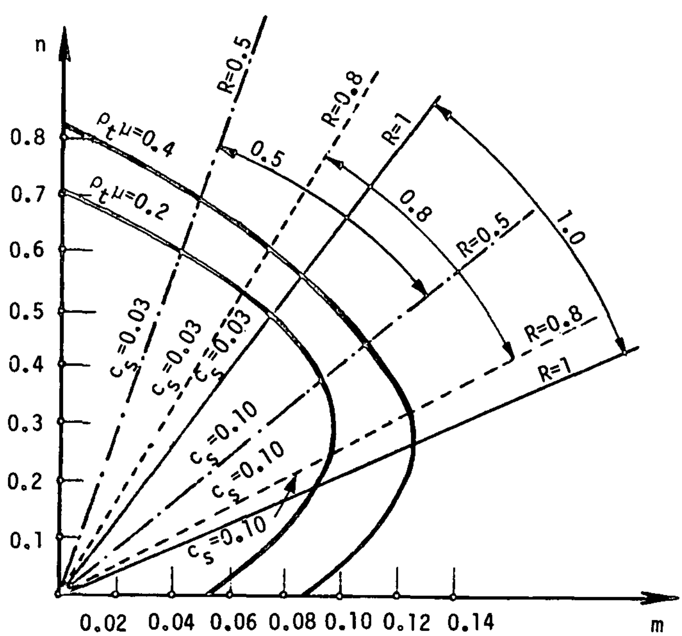


Figure 3-9. The Probable Range of Values of $\rho_t\mu$ and c_s and the Resulting Limit Values of n and R

- if $n > 0.60$ the structure has no chance to achieve a strength factor $R > 0.5$, and so obviously needs strengthening;

- if $n < 0.20$, then $R > 0.8$ and the type of failure in eccentric compression is always ductile, so that only repair is probably needed;

- if $0.20 < n < 0.60$, then R may have values between 0.5 and 1.0 depending on the average column slenderness expressed by H/h and the diagram corresponding to the respective value of H/h (see Figure 3-8) has to be used to estimate R directly from n and c_s(req.).

If stiffness varies significantly from column to column, then it is unavoidable to determine R_i for each column i or for each group of roughly identical columns and deduce R for the whole structure as

$$R_{structure} = \text{minimum significant } R_i$$

Based on the above considerations regarding the assessment of the strength and ductility of an existing structure, the simplified procedure illustrated by the flow chart in Figure 3-6 (third page) may be adopted.

In Appendix B, a more detailed explanation of the fundamentals of this procedure as well as some examples of its use are presented.

3.4.2 Decision Procedure and Construction of Alternative Rehabilitation Concepts

A full structural rehabilitation consists of improving the structural layout and upgrading strength, deformability, and ductility to the level required by the codes. Technical difficulties of full rehabilitation or else economic or social considerations may lead to the acceptance of a lower level for one of these characteristics if adequate compensation is provided for by the other characteristics. Consequently, a very limited intervention on the structure may be decided on, so that only repair of visibly damaged structural members is made. For instance, one may accept less strength and higher deformations, if the structure has a good layout and good ductility. This means that extensive non-structural and rather significant structural damage but no collapse is to be expected in a future code design earthquake. However, there are limits to acceptability. A strength factor of about $R < 0.5$ is unacceptable because chances are poor or non-existent that ductility could compensate for such low strength in order to protect against collapse. A deformability higher than the condemnation drift is unacceptable because the state of stress becomes unpredictable owing to high second-order effects.

For another instance, one may accept less ductility than required by the codes, if enough strength in the elastic range is available.

If neither existing strength nor ductility offer a satisfactory probability of survival of the building in the relevant code design earthquake, then structural rehabilitation or else an appropriate change in building use should be decided on.

Structural rehabilitation may consist of:

- an intervention in the existing structural members so that their individual strength and/or ductility are improved and in this way the respective characteristics of the structure are influenced (e.g., jacketing of the columns), even though the structural scheme is unmodified, or

- an intervention in the structure as a whole and that improves its lateral force resistance, stiffness and ductility through the addition of new structural members to increase the respective characteristics of the structure (e.g., bracing in a frame or skeleton structure or new shear walls in a shear wall structure in order to reduce the eccentricity of the masses) or even through introduction of a new lateral force resistant structure to take up integrally the seismic action (e.g., stiff shear walls introduced in a flexible frame or skeleton structure). Such an intervention produces significant changes of the stress distribution in the structure as well as in the structural layout.

The first sort of intervention may be called strengthening and the second, which implies rethinking the entire structure, may be called redesign. Certainly, no sharp limits can be drawn between these two sorts of interventions, because any intervention improves the stiffness as well as the strength and ductility of the whole structure and any intervention concept implies some thinking and design. Therefore, in order to avoid discussions about terminology, the term strengthening has been adopted to cover any intervention.

In order to point out the main characteristics of the type of strengthening needed, four strengthening types are differentiated:

Type I - improving mainly the ductility and energy absorption capacity in the post-elastic range.

Type II - adding strength and stiffness in order to diminish the deformability.

Type III - adding strength and systematically improving the ductility.

Type IV - reshaping the structure through the addition of new structural elements or even a replacement structure for lateral loads.

Figures 3-10, 3-11 and 3-12 explain some of the concepts involved and illustrate graphically, through schematic S - Δ diagrams, some examples of rehabilitation decision alternatives.

3.4.3 The Technical and Economic Feasibility Study

3.4.3.1 Contents of the Study

The feasibility study forms the basis for the final decision regarding the concept and the economic acceptability of the rehabilitation. This study should contain the following:

a) The technical portion: preliminary design, estimate of amount of construction, cost estimate, estimate of the means and materials, special tools and machinery);

b) The economic part (the efficiency index of each variant);

c) Comparison, conclusions, and recommendations.

3.4.3.2 The Preliminary Design

The extent of the preliminary design is highly dependent on the particular case in discussion. In principle, this design should contain at least a general structural layout and the amount of structural analysis and details needed to demonstrate the design's validity, the correct outline dimensioning, and the technical feasibility of the respective rehabilitation variant. The degree of seismic protection to be achieved is either established through appropriate regulations developed by competent authorities or else examined through alternatives for each case. Based on the preliminary overall concepts assessed as a result of the technical examination, various degrees of seismic protection could be achieved. For instance, a structure having a strength factor $R < 0.5$ could be upgraded to a strength factor in the range $R = 0.5$ to 0.8 or else in the range $R = 0.8$ to 1.0. In some cases, this poses only a problem of adequate dimensioning and a difference in the quantities of materials needed. In other cases, this may pose a problem of selecting a different type of strengthening. For instance, if by a type III strengthening only a value of $R = 0.6$ to 0.7 could be reached, to achieve a value of $R = 0.9$ to 1.0 a type IV strengthening would be needed. Finally, in some cases, even the most radical type IV strengthening--introduction of a replacement lateral force resistant structure - may achieve only a limited improvement of the strength factor because the preservation of the building function limits the possibilities of shaping and dimensioning the new structure.

Given the wide range of possible situations, standard values of the desired degree of seismic protection can hardly be established. We can only suggest that in principle three degrees of seismic protection should be taken into consideration, (with the most appropriate strengthening type selected among those established according to the flow charts in Figure 3-6) namely:

- simple repair
- strengthening to an R factor of about 0.6 to 0.7
- strengthening of an R factor of about 1.0

3.4.3.3 The Economic Efficiency Index

The economic efficiency index EF may be roughly defined as the ratio between the rehabilitation cost and the cost of a new replacement building of the same kind or satisfying the same social functions.

Certainly, in a more comprehensive account of economic efficiency, many considerations should be involved and compared, including the achieved degree of safety and the technical feasibility of the two structures, the maintenance costs of the building in the two cases, and the efficiency of land use. For such a thorough definition of efficiency, we refer to specialized literature. But since obviously the approach could not be the same in all cases-for the central zone of a metropolis or a small town somewhere in a seismic zone - and since these considerations are subject to insignificant variations in common cases, for the purpose of a rehabilitation feasibility study such an preliminary simple definition may be accepted.

In order to provide more accurate information about efficiency, the cost of the rehabilitation should be subdivided as follows:

K_1 = the cost of the structural repair and strengthening

K_2 = the cost of the nonstructural damage repair

K_3 = the cost of restoring the otherwise undamaged nonstructural elements and finishes, affected by the intervention in the structure

K_4 = cost of other nonstructural repairs that are independent of the intervention in the structure and are needed to ensure normal functioning of the building.

Consequently, one may allow $K_T = K_1 + K_2 + K_3 + K_4$ = the total cost of the rehabilitation and

V_{ex} = the cost of a new building functionally identical to the existing one

V_{rep} = the cost of a replacement building having the same social functions, but not necessarily with an identical architectural layout

K_D = the cost of the demolition of the existing building.

Then an efficiency index EF of the rehabilitation may be defined as

$$EF = \frac{K_T}{V_{ex} + K_D}$$

or alternatively as

$$EF = \frac{K_T}{V_{rep} + K_D}$$

Both definitions are valid, depending on the specific case. For a building having some specific feature, for instance special finishes, the comparison to V_{ex} is advisable. For an average building, the comparison to V_{rep} is acceptable and simpler.

The efficiency index is to be compared to specific limit values. The admissible or reasonable limits of EF are dependent on the economic, social or architectural value of the building. Buildings may be divided according to their functional and architectural value, and to the quality and state of their nonstructural components, into four classes of importance that are related to social rehabilitation decisions:

Class 1 - special importance. To be maintained at any cost.

Class 2 - functionally good and up-to-date, architectural valuable, good finishes, materials, and workmanship. To be maintained.

Class 3 - functionally acceptable, architecturally neutral average finishes, non-structural materials, and workmanship. To be maintained, if maintenance is not inefficient.

Class 4 - functionally worthless or obsolete, poor materials and workmanship. To be maintained only for a limited lifetime as long as possibly needed to respond to some social needs (for instance, in case of a housing shortage).

For each class, upper limits of each intervention's efficiency may be defined in terms of limit values of the efficiency index EF. Certainly, these limits are informative, although highly dependent on specific conditions and susceptible to variation in time. Proposed limits are found in Table 3-2. In some situations, even as much as EF = 80% could be accepted.

Table 3-2. Tentative Upper Limit Values of EF

CLASS 1	NOT LIMITED
CLASS 2	50%
CLASS 3	35%
CLASS 4	20%

Furthermore, one may take into account the following:

a) By a new low- or middle-rise building, in normal conditions the cost shares could be assumed as in Table 3-3:

Table 3-3. Cost Breakdowns in Low- and Middle-Rise Buildings

	RC FRAMES	BEARING BRICK MASONRY
FOUNDATIONS AND INFRASTRUCTURE	8 - 10%	8 - 10%
STRUCTURE	30 - 40%	40 - 50%
NONSTRUCTURAL MEMBERS AND FINISHES (WITHOUT EQUIPMENT AND INSTALLATIONS)	60 - 50%	50 - 40%

b) For the cost of structural rehabilitation, some usual values based on previous experience could be assumed as in Table 3-4:

Table 3-4: Typical Values of Structural Rehabilitation Costs

STRUCTURAL CATEGORY	TYPE OF STRENGTHENING	AVERAGE OBSERVED COST	
A	I	K1	K1 + K2
		3%	5%
B	II - III	8-10%	15%
C	III - IV	10-12%	25%
D	III - IV	12-15%	35%

c) Estimation of the degree of damage (by earthquake or other causes) of nonstructural members and finishes can be categorized as:

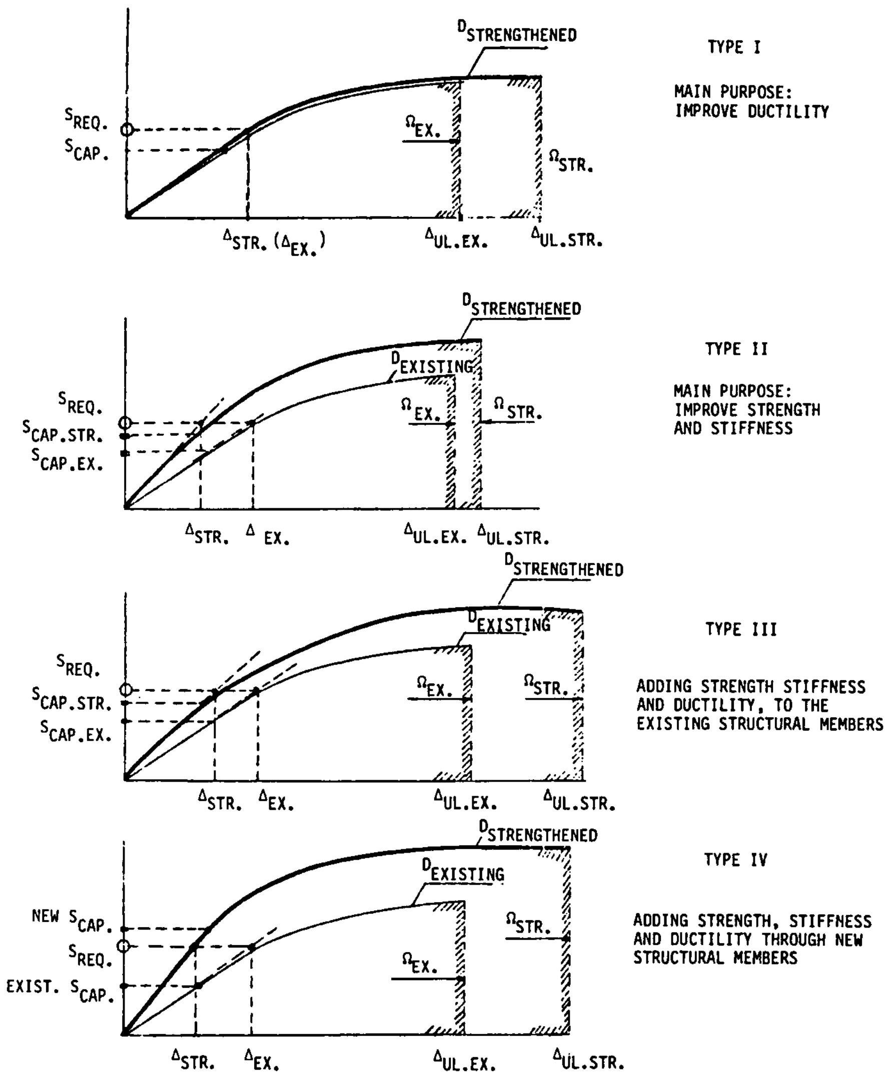
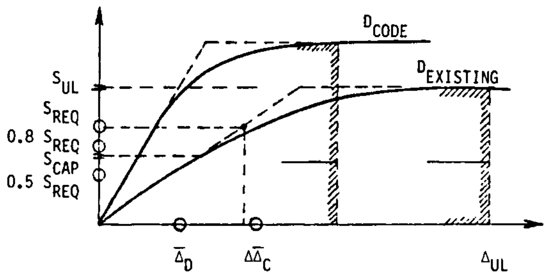


Figure 3-10. Diagrams Illustrating the Strengthening Types

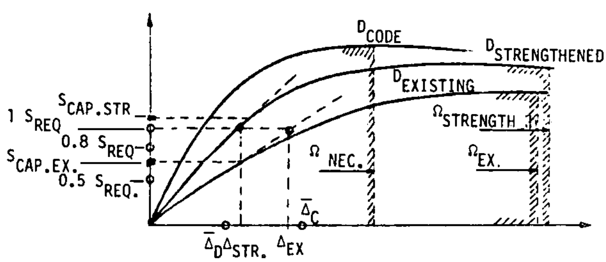


CHARATERISTICS OF THE CASE

1. CATEGORY OF THE STRUCTURE: B (GOOD LAYOUT, 0.5 R 0.8)
2. $\bar{\Delta}_D < \Delta < \bar{\Delta}_C$
3. DUCTILITY CONDITIONS FULFILLED (GOOD CHANCES TO HAVE $\Omega_{EX} > \Omega_{NEC}$).

(a) EXAMINATION OF THE EXISTING STRUCTURE [RELATIVE POSITION OF $D_{EXISTING}$ COMPARED TO D_{CODE}]

ALTERNATIVE 1: ONLY REPAIR

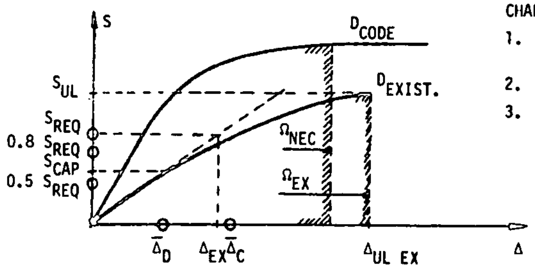


ALTERNATIVE 2: STRENGTHENING TYPE II — ADDING STRENGTH AND STIFFNESS TO THE EXISTING MEMBERS. [$S_{CAP.}$NEW > $S_{CAP.}$ EXISTING AND $\Delta_{STR.} < \Delta_{EXIT.}$] WITH NO SPECIAL CONCERN FOR IMPROVING DUCTILITY

ALTERNATIVE3: STRENGTHENING TYPE IV — ADDING STRENGTH AND STIFFNESS BY ADDITIONAL STRUCTURAL MEMBERS

(b) ILLUSTRATION OF POSSIBLE STRENGTHENING CONCEPTS

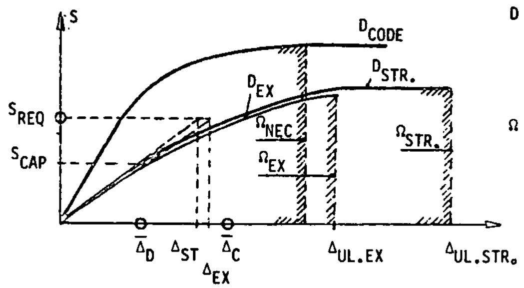
Figure 3-11. Illustration of Alternative Rehabilitations Decisions in a Given Case



CHARACTERISTICS OF THE CASE:

1. CATEGORY OF THE STRUCTURE B (GOOD LAYOUT, $0.5<R<0.8$)
2. $\bar{\Delta}_D<\Delta<\bar{\Delta}_C$
3. DUCTILITY CONDITIONS NOT FULFILLED (POOR CHANCE TO HAVE $\Omega_{EX}>\Omega_{NEC}$)

(A) RELATIVE POSITION OF $D_{EXISTING}$ COMPARED TO D_{CODE}

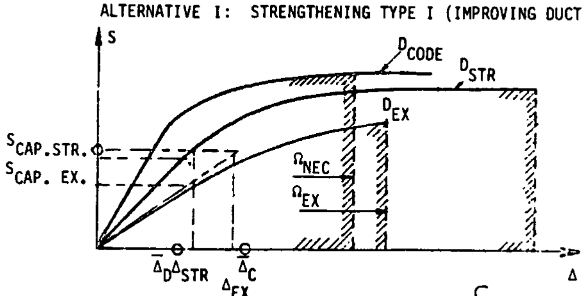


LEGEND:

D = SCHEMATIC DIAGRAMS
- D_{CODE} = CODELIKE STRUCTURE
- D_{EX} = EXISTING STRUCTURE
- D_{STR} = STRENGTHENED STRUCTURE

Ω = AREA ENCLOSED BY THE S-Δ DIAGRAM (REPRESENTING ENERGY ABSORBTION)
- Ω_{NEC} = NEEDED, ACCORDING TO CODE
- Ω_{EX} = CAPACITY OF THE EXISTING STRUCTURE
- Ω_{STR} = CAPACITY OF THE STRENGTHENED STRUCTURE

ALTERNATIVE I: STRENGTHENING TYPE I (IMPROVING DUCTILITY)



ALTERNATIVE 2: STRENGTHENING TYPE III — ADDING STRENGTH AND STIFFNESS IMPROVING DUCTILITY OF EXISTING STRUCTURAL MEMBERS

ALTERNATIVE 3: STRENGTHENING TYPE IV — SAME: BY ADDITIONAL STRUCTURAL MEMBERS

Figure 3-12. Illustration of Alternative Rehabilitation Decisions in a Given Case

insignificant:	if damaged in a proportion of	1/10 - 1/20
low:	if damaged in a proportion of	1/5 - 1/6
middle:	if damaged in a proportion of	1/3
extensive:	if damaged in a proportion of	1/2

The table in Figure 3-13 could be used to estimate the order of magnitude of the efficiency index and consequently the chances of a rehabilitating a specific building within the limits established for its class of functional importance.

NONSTRUCTURAL DAMAGE / STRENGTH CATEGORY	INSIGNIFICANT	LOW	MIDDLE	EXTENSIVE
A	10%	15%	20%	30%
B	15%	20%	30%	40%
C	20%	30%	40%	50%
D	30%	40%	50%	60¢

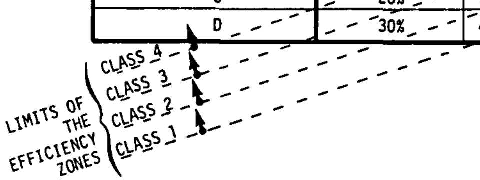


Figure 3-13. Probable Average Values of the Efficiency Index EF

This table (in Figure 3-13) provides the following sorts of information:

- a building of the strength category D, with middle or extensive non-structural damage, has little or no chance of being rehabilitated in the limits set up in Table 3-2, or

- a building from the class of importance 4 has little or no chance to be rehabilitated in the defined limits, if the building is not of the strength category A or B (or C with insignificant damage), or

- a building of the strength category C and of class 3 in importance could be efficiently rehabilitated only if non-structural damage is low.

The figures mentioned in this paragraph provide average estimates. The purpose is not give precise quantitative information which is very dependent on specific conditions. Rather, these figures illustrate a method to provide rapid analysis of the economic efficiency of rehabilitation.

3.4.3.4 Alternatives to Structural Rehabilitation

The technico-economic study should also consider two alternative decisions to rehabilitation. The strength factor $R = S_{cap}/S_{req}$ can be increased either by increasing S_{cap}, i.e., structural rehabilitation, or else by reducing S_{req}. S_{req} may be reduced in two ways. First, the usage of the building of its occupancy level may be changed resulting in a lower importance category. Second, partial demolition may be made as of upper stories or of heavy non-structural building components. Changing the building usage may be in some cases a social rather than an economic decision. Nevertheless, some costs may be incurred for damage repair or architectural adjustment or even be associated with a small-scale intervention in the structure.

3.4.3.5 Use of the Feasibility Study for Decision Making

On the basis of the feasibility study, decisions have to be made for individual buildings with regard to the acceptable value of the strength index R and the selected rehabilitation concept for the final design.

For building populations, the sum of the conclusions of individual feasibility studies, or selected conclusions regarding some typical building structures, may serve as considerations in the development of a rehabilitation policy by the policy administrator.

A socially accepted seismic risk should be specified in terms of the accepted limit values of the strength factor R. These limit values provide a guide for the structural engineer in making the final design and at the same time serve as a delimitation of his social liability.

3.4.3.6 Limitations on the Life-span of the Rehabilitated Building, as a Function of the Seismic Protection Obtained

In the process of decision making, any rehabilitation achieving a strength factor $R < 1$ implies the acceptance of one of the following alternatives:

- a limitation on building life-span as compared to the reference life-span of structures designed according to code provisions (if the same seismic risk as accepted as a basis for the code provisions is desired), or

- a higher seismic risk than that accepted in the codes, if no such limitation on life-span is defined.

Based on the theoretical assumption explained in Sections E.8.3 and E.8.4 of Appendix E, the following relation could be used in assessing the life-span limitation

$$\frac{T_{service}}{T_{code}} = (R)^{1.5 \ \ldots \ 2}$$

where

$T_{service}$ = the (shorter) life-span of the rehabilitated structure, subsequent to rehabilitation;

T_{code} = the reference life-span of the structures designed according to the code provisions;

R = the strength factor that could be obtained by a given structural rehabilitation (repair or strengthening).

Note that the same formula could be used to assess the permissible delay of completing the rehabilitation of a damaged structure. In this case, the strength factor R is to be determined for the damaged structure in its actual state before rehabilitation.

3.4.4 The Final Design

After selection of the rehabilitation solution, the engineer has to proceed to the final design. The design is governed by existing generally valid design rules and regulations, with due regard for the specific details of strengthening techniques, as explained in manual 5. In cases where low values of the strength factor R have to be accepted, the engineer may resort to more complex analytic methods or mathematical models. For instance, assessment of the ultimate strength and energy absorption capacity may be made by nonlinear analysis, or by considering also the contribution of such typical nonstructural elements as infill masonry, in order to demonstrate the seismic performance of the rehabilitated structure.

Note that according to the methodology previously explained in this chapter, the strength factor R was determined for the initial structure in its pre-earthquake state, i.e., considered as undamaged and with the initial strength and deformability of the materials. In this stage, the purpose of the analysis has been only

- to explain the damage pattern;
- to decide between repair and strengthening (i.e., to assess whether or not the return to the initial structure would be acceptable as a rehabilitation concept);
- to establish in principle alternative strengthening concepts.

In the subsequent stages, the feasibility study and the final design, the analysis is to be based on a complete and appropriate mechanical model of the rehabilitated structure. The model considers the actual stiffness of the strengthened structural members, the role and influence of the new structural members, and the actual residual strength and flexibility characteristics of the non-strengthened or simply repaired structural members which may still have a significant role in assuring the lateral force resistance of the strengthened structure.

As regards these residual characteristics, the effect of the earthquake on reinforced concrete and masonry members will decrease their initial stiffness - owing to very fine internal cracking - and will diminish the initial ductility of reinforced concrete members, even though these members may appear to be undamaged.

This does not significantly affect the ultimate capacity of the respective members, but is of concern when computing their contribution to the stiffness and ductility of the strengthened structure. The quantitative values of the residual stiffness and ductility are hardly to be exactly assessed. These values may be based largely on engineering judgment. Generally speaking, for the computation of deformations (e.g., story drifts), an average reduction by 10-30% of the concrete deformation-modulus may be assumed. The loss of ductility may be compensated for by assuring a higher ductility for the strengthened members, for the newly introduced structural members, or for the structure as a whole.

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4. METHODS FOR PRE-EARTHQUAKE EVALUATION OF DAMAGE POTENTIAL

4.1 General Considerations

There are many methods of gathering the information needed for pre-earthquake rehabilitation programs. Those methods differ in cost, scope, precision, and sophistication. The planner or engineer may begin with simpler methods for initial screening and then proceed to more complicated analyses. All simpler methods can be used as potential screening techniques for more in-depth work. For some structures, such as older concrete highrise buildings, initial screening may be bypassed if detailed assessment is obviously needed. Initial screening is not needed for structures already known to be as seismically resistant as desired.

Risk is a function of the vulnerability of a structure and the severity of the seismic hazard. Four general methods of assessing building vulnerability and two general methods of assessing seismic hazard are presented below.

4.1.1 Methods for Assessing Building Performance Under Seismic Excitations

- Categorization Methods: These methods classify structures with special reference to the structural framing system and to the date of construction as it bears on seismic codes and construction practices.

- Inspection and Rating Methods: These methods rate structural elements critical to earthquake resistance.

- Analytical Methods: These methods analyze the expected resistance of a structure in response to assumed ground motions.

- Experimental and Analytical Techniques: These methods combine analytical techniques with data from vibration tests or other experiments.

4.1.2 Methods of Assessing Local Hazard

Seismic hazard assessments can be made with varying degrees of sophistication. Seismic hazards may be divided into those that are purely vibratory, including amplification effects, and those that also involve surface ground displacement, including effects of faulting, liquefaction, lurching, compaction, foundation settlement and landslide. Other secondary effects may include tsunami or seiches. Figure 4-1 summarizes a range of such factors.

Two principal ways exist for assessing seismic hazards. First, one may postulate a given earthquake scenario as it affects the site in question. Accordingly, ground motion parameters for the postulated earthquake may be derived for the site. This scenario may be a "design" earthquake for design purposes. Or, it may be a "maximum credible" earthquake, for estimating maximum expected losses in a given region. Second, one may provide a statistical account of the entire range of earthquakes expected to affect the site in question. Using this method, one is able to estimate the overall risk to the structure. This second method is the primary one to be referred to in this manual.

To use a statistical method in conjunction with categorization techniques generally requires some measure of ground motion at the site, such as vibratory measures (primarily indicated in intensity scales). Overall risk to the building in question requires frequency estimates for the entire range of ground motions expected at the site. If the risk analyst limits loss estimates to expected losses from only one earthquake scenario, he will underestimate annual expected losses from the entire range of possible earthquakes. Use of regional or site amplification factors can increase the accuracy of overall loss estimates. Quantification of other hazard factors, such as potential faulting or other ground failures expected at the site, would indicate likely increase in risk for the building in question. It is believed that through post-disaster assessment surveys, quantifiable vulnerability models will eventually be developed for buildings to account for ground failures. Presently, risk analysts can only make "educated guesses" on the effects of ground failure on buildings.

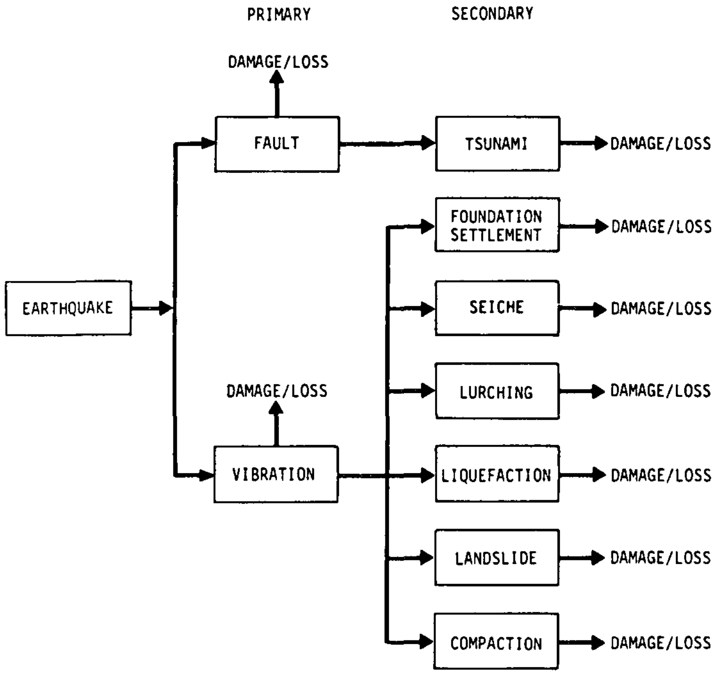

Figure 4-1. Primary, Secondary and Higher Order Earthquake Hazards Leading to Damage and Losses (from Wiggins et al., 1981)

Inspection and rating techniques may use information on regional seismicity only implicitly. Such techniques are generally developed for regions of fairly high seismicity. Criteria for action resulting from ratings may be adjusted to accord with the seismicity of the region in question, along with economic, legal, and other factors. Whereas loss estimates may be calculated from categorization techniques, no such estimates are required by inspection and rating techniques. However, in passing, these latter techniques may promote more accurate definition of the "categories" to which specific structures belong, and so modify previous loss estimates made based on more limited building data.

Analytic techniques typically model details of the building response and expected damage relative to specific seismic inputs. Expected building response to the entire range of expected ground motions could be analyzed through discrete analyses for a number of expected ground motions. However, costs of each analysis tends to be high, and expected overall damage to the structure can more cheaply be scaled relative to estimates based on one or two specific analyses.

Experimental data can be used to refine analytical predictions of building performance. Once such techniques are employed, analyses can be revised. Additional other experiments can be made to determine deterioration, welding, mortar quality, cracking and other structural data pertinent to analyses.

4.2 Categorization Methods

4.2.1 Uses

The use of categorization techniques enables public officials to make the following types of decisions:

1. whether specific categories of buildings should be examined in greater detail, and by what methods,

2. whether specific categories of buildings should be ignored,

3. whether prompt seismic rehabilitation measures (or other mitigations such as occupancy reduction measures) should be applied to specific buildings,

4. whether more limited seismic mitigation actions only should be considered and possibly researched, and

5. what priorities for further examination and/or seismic mitigation are desirable.

Categorization techniques yield general loss estimates that can be used to evaluate the costs of rehabilitation programs. Other uses include determination of losses for emergency preparedness programs, determination of probable maximum losses for insurance programs, examination of the expected distribution of losses, and determination of overall expected losses including secondary effects of damage.

Categorization techniques are thus potentially extremely useful in setting priorities for earthquake rehabilitation efforts, from either a national, regional, or a community level. They may be used, for instance, to determine the comparative earthquake risks in various regions of a country. In some cases, those risks may warrant further examinations of buildings in order to determine which should be retrofitted. In other cases, those risks may be found to be so low as to preclude any further analysis.

4.2.2 Classification to be Used

In line with attempts to provide a uniform method for developing rehabilitation programs in the Balkan regions, a possible categorization scheme could be that derived from damage evaluation forms. Those damage evaluation forms contain, for example, the following relevant categories:

TYPE OF STRUCTURE

1st DIGIT	2nd DIGIT	3rd DIGIT
1 = Masonry	1 = No belts 2 = Horizontal belts 3 = Horizontal and Vertical belts, or Diagonal braces 4 = R.C. floors or roof	1 = Adobe 2 = Stone with no mortar 3 = Stone with mortar 4 = Solid brick 5 = Hollow brick 6 = Concrete blocks 7 = Unreinforced concrete 8 = ____________ 9 = ____________
2 = Reinforced Concrete	1 = Cast in place frame 2 = Cast in place bearing walls 3 = Prefabricated 4 = Mixed with masonry 5 = Mixed with steel	1 = With lightweight partitions 2 = With solid brick in-fills 3 = With hollow brick in fills 4 = With concrete block infills
3 = Steel	1 = Heavy steel structure 2 = Light steel structure 3 = Mixed with masonry or concrete	1 = With lightweight partitions 2 = With solid brick in fills 3 = With hollow brick in fills 4 = With concrete block infills
4 = Wood	1 = Wood frame 2 = Bagdadi 3 = Other	1 = ______________ 2 = ______________

Subcategories or other categories are also useful in defining relevant groupings, as are data on construction year, number of stories, gross area, and roof, floor, and foundation characteristics. Usage type is relevant only in cases where it indicates expected building response. It should be noted that these classifications ignore wood frame structures except for those included under local types. For building rehabilitation programs, this omission is satisfactory insofar as wood frame structures are not likely to receive high priority in such programs.

Alternatively, buildings may be rated according to quality from, for example, 1-7 (with 1 being the least resistant structures). Using such quality ratings, structures can be compared based on framing system, construction year, number of stories, etc.

A much longer account of existing vulnerability analyses is given in Appendix C. A brief summary of existing techniques is given in Table 4-1.

4.2.2.1 Loss Estimates by Expert Panels

In order for engineers and planners to use categorization techniques, expert panels must first determine the expected losses to various types of buildings from earthquakes of varying intensity. Tables 4-2 through 4-5 illus-

Table 4-1. A Brief Listing of Existing Categorization Techniques

A. LOSSES TO BUILDINGS	ESTIMATES OF MEAN LOSS (AS A PERCENT OF REPLACEMENT COST) X CLASS OF BUILDING X INTENSITY (AND/OR OTHER SHAKING PARAMETER).
(1) SHAH et al. TECHNIQUES	BASED ON AN EXAMINATION OF A LARGE NUMBER OF PREVIOUS CATEGORIZATION TECHNIQUES, SUMMARY MEAN LOSS CURVES ARE DEVELOPED FOR 10 CLASSES OF STRUCTURES. NUMEROUS OTHER TECHNIQUES ARE REFERRED TO HERE, INCLUDING (2), (3) AND (4), AND ALSO JOHN BLUME TECHNIQUES.
(2) WHITMAN et al. TECHNIQUES	LOSS ESTIMATES ARE PROVIDED FOR CLASSES OF UBC STRUCTURES (UBC 0, ETC.). PROBABILITY DISTRIBUTIONS ARE DEVELOPED. EMPHASIZED ARE BUILDINGS WITH FIVE OR MORE STORIES.
(3) STEINBRUGGE et al. TECHNIQUES	LOSS ESTIMATES ARE BASED ON ISO CLASSES. ALSO INCLUDES ESTIMATES ON WOOD FRAME AND MOBILE HOME RESIDENTIAL STRUCTURES.
(4) WIGGINS et al. TECHNIQUES	QUALITY GRADINGS ARE GIVEN, AND ARE DIVIDED INTO RESIDENTIAL AND COMMERCIAL-INDUSTRIAL CATEGORIES.
(5) ROMANIAN STRUCTURES TECHNIQUES	BASED ON ROMANIAN STRUCTURES DAMAGED IN AN EARTHQUAKE, MEAN AND ONE-SIGMA (PLUS OR MINUS) ESTIMATES ARE GIVEN. EXTRAPOLATION IS NEEDED FOR LOWER AND HIGHER INTENSITIES.
B. DEATHS	
(1) STEINBRUGGE et al. TECHNIQUES	MEAN LIFE LOSS IS ESTIMATED AS A PERCENT OF OCCUPANTS BY UBC SEISMIC DESIGN, BUILDING HEIGHT, AND MMI.
(2) WHITMAN et al. TECHNIQUES	MEAN LIFE LOSS IS ESTIMATED AS A PERCENT OF OCCUPANTS BY UBC SEISMIC DESIGN AND MMI.
(3) BAYULKE AND AKKAS TECHNIQUES	ESTIMATES ARE MADE AS A PERCENT OF COLLAPSES EXPECTED TO TURKISH BUILDINGS.
(4) ANAGNOSTOPOLOUS & WHITMAN TECHNIQUES	DEATHS ESTIMATED AS A FUNCTION OF DEGREE OF DAMAGE AT VARIOUS MMI LEVELS AND OCCUPANCY RATE FOR UNREINFORCED MASONRY STRUCTURES.

trate the formats experts may use to provide the data needed for categorization techniques. First, experts must describe the types of buildings in the area to be analyzed by categorization techniques (Table 4-2). Then, they must estimate losses as a percent of replacement costs (Table 4-3), the percent of buildings sufficiently damaged to cause deaths at various levels of earthquake intensity, and the expected number of deaths per 100 people exposed (Tables 4-4 and 4-5).

Table 4-2. Building Categories

	DESCRIPTION
A	
B	
C	
D	
E	
F	
G	
H	
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.	

Table 4-3. Mean Losses as Percents of Replacement Cost MMI or MSK Scale

BUILDING CATEGORY	MSK OR MMI INTENSITY						
	VI	VII	VIII	IX	X	XI	XII
A							
B							
C							
D							
E							
F							
G							
H							
.							
.							
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Table 4-4 illustrates the percent of buildings of each category that would be sufficiently damaged to cause deaths following an earthquake of a given intensity.

Table 4-4. Estimates of Percent of Buildings Sufficiently Damaged to Cause Deaths

BUILDING CATEGORY	MSK OR MMI INTENSITY						
	VI	VII	VIII	IX	X	XI	XII
A							
B							
C							
D							
E							
F							
G							
H							
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Table 4-5 can be used to indicate the expected deaths per 100 people exposed.

Table 4-5. Expected Mean Deaths Per 100 People Exposed in Building Types Affected at Various Levels

BUILDING CATEGORY	MSK OR MMI INTENSITY						
	VI	VII	VIII	IX	X	XI	XII
A							
B							
C							
D							
E							
F							
G							
H							
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4.2.3 Steps to be Used in Analysis by Categorization Techniques

The following steps indicate some of the ways in which categorization analysis can be used. Such steps will generally require judgment as to the degree of seismic rehabilitation possible at a given cost (or cost per square meter). In addition, such steps will typically require hazard analysis which provides the annual expected frequency of a given intensity at a given site. Within regions of fairly homogeneous seismicity averages may be used or site-specific hazard analyses may be developed. The amount of detail required by hazard analysis will depend on the degree of accuracy obtainable and desirable.

Step 1 - For a site or many sites, derive expected annual losses (as percents of replacement cost) and also expected annual deaths per 100 people exposed for each of the categories of structures.

Here one uses Tables 4-3 and 4-5 in conjunction with estimates of expected annual frequency of intensities at given sites. If, for instance, L_j is the expected loss (as a percent of replacement cost) at a given intensity j for a given category of building and N_j is the annual expected occurrence of intensity j at a given site, then

$$\text{Expected annual losses to a building of that category at that site} = \sum_{j=6}^{12} N_j L_j \qquad (4\text{-}1)$$

Only Tables 4-3 and 4-5 and appropriate hazard analyses are required for these calculations. However, these calculations can be used in conjunction with results of building surveys. But, if the analyst has some knowledge of the mix of building classes in the area, a building survey is unnecessary for these calculations.

After initial calculations are complete, various risk criteria can be used to determine whether surveys and/or indepth inspections are needed, and if so, for what potential categories of structures. It may, for instance, be useless to survey structures built after adequate seismic codes have been enacted, or to survey structures in a region of very low seismicity. Acceptable risk criteria may be applied to any category of structures in order to estimate whether any structures pose unacceptable risks. If such is not the case, for a given region, then it is likely that no further studies are needed (unless hidden factors are creating potential gross underestimates of loss). Risk criteria available are discussed in Chapter 6. If rough estimates of rehabilitation costs are available at this stage, benefit-cost or risk-benefit criteria can be used in addition to or rather than acceptable risk criteria. (Alternative suggestions for determining what regions to ignore are found in Section 5.1.)

A second step involves use of a rapid survey on the basis of the risk criteria chosen.

Step 2 - Make a rapid survey of certain categories of buildings in order to refine results of Step 1.

Importance factors (analogous to those defined in Appendix C) and the results of Step 1 will indicate if a rapid inventory of existing structures should be undertaken by engineers. This survey is not intended to be a full field inspection of each structure, but rather to give a preliminary account of existing structures so that broad policy decisions can be made on rehabilitation priorities.

Items of special interest in such a broad survey include a building-by-building account of:

1. structural category to which a building belongs (A, B, etc.),
2. maximum occupancy for building,
3. number of square meter in building (approximate),
4. number of stories,
5. construction date of building, and dates of any modifications,
6. seismic code in effect at time of construction date, and modifications,
7. any special contents in building (computers, vaults, explosives, etc. whose damage may give rise to considerable losses beyond those to the building itself),
8. occupancy density characteristics (is the building used throughout the day, what is the mean use per square meter, etc.), and
9. importance factor for the building (see Appendix C), or general function of building (apartment dwellings, etc.).

These factors will provide overall data for general policy analysis. Of special interest, of course, is the structural category to which the building belongs. Other factors are intended to allow for adjustments in loss estimates or policy considerations. For instance, determination that special contents are located in a structure or that the building has a high exposure content such as expensive equipment may increase loss estimates.

Two separate occupancy measures can be used for estimating life safety risks. Maximum occupancy is the traditional measure for earthquake design codes. However, for estimating life safety risks, it is also possible to use density of occupancy (mean occupancy rates per square meter) in order to obtain more precise estimates of expected deaths. Some structures may have very low maximum occupancy but be very densely occupied (apartment buildings with small apartments housing large families, offices used throughout the 24 hour day, etc.). In contrast, some structures such as older auditoriums may be rarely used and so have large occupancy potential but very low mean occupancy rates. Determination of mean occupancy per square meter is a helpful measure if potential seismic rehabilitation measures are also calculated on a square meter basis. When considering occupancy reduction to minimize earthquake risks, it is necessary to take into account actual (including mean) occupancy. Use of maximum occupancy as a measure will tend to correspond to risk criteria based on avoidance of larger losses from single buildings. Use of mean density occupancy as a measure will tend to correspond to treating all life safety risks on an equal basis regardless of the possible (often low probability) risks of numerous deaths on one occasion or at one site.

Once such a survey is complete, benefit-cost analyses, risk-benefit analyses, or catastrophic risk analyses may be made (see Chapter 6). Generally speaking, these analyses require preliminary engineering judgments on the potential costs of seismically upgrading structures and on the comparative value of using certain funds for seismically upgrading a given facility. In addition, as previously pointed out, length of time for expected continued usage of a building (whether the building is to be seismically retrofitted or not) will need to be estimated. Benefit/cost analyses also require that expected reductions in deaths and economic losses from proposed seismic mitigation measures be calculated.

4.3 Visual Inspection of Critical Elements and Rating Methods

4.3.1 Uses

Rating methods incorporate more data into the assessment of particular structures than categorization techniques do, but they do not involve actual analysis of expected structural response. These methods use visual inspection and concentrate on structural elements and characteristics critical to seismic response. Inspection and rating techniques are used to make the following decisions on particular structures:

1. total retrofitting or rehabilitation,
2. partial retrofitting,
3. no retrofitting, and
4. further study.

Appendix C discusses in detail two of the many rating methods available: the Field Evaluation Method and the Wiggins et al. Method. Such rating methods are difficult to compare. First, the items emphasized for visual inspection and rating may be grouped differently. Second, the weights given to various items may not have obvious derivations. Third, the emphases may implicitly be different depending upon whether a rating technique stresses death and injury risk, or potential economic losses, or both.

Table 4-6 below provides a brief listing of available rating and inspection techniques.

Table 4-6. A Brief Listing of Available Rating and Inspection Techniques

A. INSPECTION AND RATING	
(1) FIELD EVALUATION METHOD (CULVER et al.)	A CAPACITY RATIO RATING IS ESTIMATED BASED ON SEISMICITY, A GENERAL STRUCTURAL RATING, AND RATING OF PRESENT CONDITION, ANCHORAGE (NON-STEEL STRUCTURES) ETC. OF RESISTING ELEMENTS.
(2) WIGGINS et al. METHOD	A RISK SAFETY FACTOR RATING IS ESTIMATED BASED ON SEISMICITY AND A GENERAL STRUCTURAL RATING OF FRAMING SYSTEM, ETC.
(3) DECISION FACTOR ANALYSIS METHOD	A SIMILAR RATING IS PRODUCED, BUT THE RATING IS SPECIFICALLY DESIGNED FOR SAN FRANCISCO BAY AREA STRUCTURES ONLY.
(4) SHAH et al. METHOD	PENALTY INCREMENT FACTORS ARE USED BASED UPON SIX STRUCTURAL FACTORS. SEISMICITY FACTORS ARE IGNORED.

Appendix C explains how both the Field Evaluation Method and the Wiggins et al. Method can give rise to various comparative risk ratings for structures. Hence, buildings in their seismic environment will be rated either "very poor," "poor," "fair," or "good," according to the Field Evaluation Method, and "Highest Risk," "Next Highest Risk," "Third Highest Risk," and "Lowest Risk" according to the Wiggins et al. Method. These qualitative assessments based on detailed field inspections will roughly correspond to risk criteria.

The risk analyst may use comparative outcomes or may calibrate vulnerability totals (CR•IR for the Field Evaluation Method, the score from 180 total points for the Wiggins et al. Method) using categorization techniques developed in the previous section. In order to make such calibrations for Wiggins et al. techniques, it is necessary to find those categories expected to approximate 180 points, 120 points, 50 points, and 0 points, and from those correlations to develop risk criteria for seismic environments.

Table 4-7 shows the format that an expert panel could use to make correlations between building types and point totals.

Table 4-7. Correlations Between Building Types and Point Totals (F+D+P+PC+SH), Wiggins et al. Method

BUILDING TYPES	POINT TOTALS EXPECTED FOR BUILDINGS IN VARIOUS CATEGORIES (0-180)
A	
B	
C	
D	
E	
F	
G	
H	
.	
.	
.	

4.3.2 Steps for Visual Inspection and Rating Techniques

These steps nominally follow the previous steps outlined under the discussion of Categorization Methods.

Step 3 - Select structures to be inspected based on the outcome of categorization surveys.

It is necessary to have ordinances that make seismic inspection of structures legally permissible or mandatory. Appendix H contains a model ordinance.

The choice of which inspection and rating techniques should be used depends on expert evaluations. The discussion of the next step clarifies how to evaluate what actions may be regarded as permissible and/or mandatory following inspection.

Step 4 - Based on time and personnel considerations, inspect and rate buildings and determine mandatory/permissible actions to be taken.

Once structures are rated, the results should be translated into various actions within a specified amount of time including

(1) seismic rehabilitation of the structure,

(2) occupancy reduction,

(3) reduction of building life-span,

(4) seismic removal or bracing of special hazards,

(5) an additional inspection of the building by additional qualified observers if results of the initial rating are contested,

(6) more detailed analysis of the building and/or experimentation on the building either to define its seismic response characteristics more accurately or to define more accurately (and with possible surveys) seismic rehabilitation measures actually needed, or

(7) no action whatsoever.

Risk criteria (discussed in Chapter 6) can be used in such decisions. For instance, based on Wiggins et al. methods, policy makers may require that all buildings with an r_s rating greater than 2.0 be rehabilitated within 10 years, and that those with an r_s rating greater than 4.0 be rehabilitated within 2 years. Alternatively, the policy maker may state that a score of 150 points or greater requires rehabilitation in 2 years, etc. In deciding on what measures to be taken, the tables previously developed in this chapter can be used in conjunction with risk criteria.

In developing such requirements through use of risk criteria, one may allow for building ratings to be checked by independent experts, and may also allow for other considerations to modify the decision on what is to be done with the structure. For instance, for some structures, seismic rehabilitation costs may approach total replacement costs. In such cases, either demolition, total replacement, or alternative measures such as occupancy reduction may be suggested.

4.4 Analytical and Quasi-Analytical Methods

4.4.1 Uses

These methods give detailed analysis of seismic response of structures. They are more expensive, but also more thorough than visual inspection and rating techniques. Analytical techniques are employed when visual inspection and rating techniques do not yield sufficient information on what seismic mitigation measures should be undertaken. Also, analytical techniques may be used for a particularly valuable or structurally complex building.

4.4.2 Procedures for Analytical and Quasi-Analytical Methods

Table 4-8 contains a brief summary of methods, while Appendix C discusses analytical methods in more detail.

As Table 4-8 indicates, only in rare cases are analytic models combined with empirical estimates of losses. These include the Detailed Analytical Evaluation Method and DAMAGE No. 1 and DAMAGE No. 2. In general, such methods provide further information for engineers in determining expected response characteristics of buildings; only in some cases is this information combined with data for seismic risk assessment.

Table 4-8. Brief Listing and Summary of Analytic and Quasi-Analytic

(1) APPROXIMATE ANALYTICAL EVALUATION METHOD (CULVER et al.)	A CRITICAL STRESS RATIO (STRESSES PRODUCED BY SEISMIC LOADING TO LIMITING STRESSES OF CRITICAL BUILDING ELEMENTS) IS CALCULATED FOR STRUCTURAL AND NON-STRUCTURAL ELEMENTS. NOT FOR ONE- AND TWO-STORY RESIDENTIAL BUILDINGS.
(2) DETAILED ANALYTICAL EVALUATION METHOD (CULVER et al.)	A MORE DETAILED ANALYSIS IS PERFORMED FOR CONFIGURATIONS AND FOR STEEL FRAME STRUCTURES, CONCRETE SHEAR WALL AND SHEAR WALL PLUS FRAME BUILDINGS, BEARING WALL BUILDINGS, AND LONG SPAN ROOF STRUCTURES. DAMAGE ESTIMATES ARE YIELDED. A FINITE ELEMENT MODEL IS USED TO COMPUTE INTERSTORY DRIFT, WHICH IS THEN RELATED TO DAMAGEABILITY. NOT FOR ONE- AND TWO-STORY RESIDENTIAL BUILDINGS.
(3) H. BONCHEVA & L. TZENOV METHOD (BULGARIA)	A DAMAGE POTENTIAL INDEX IS PRODUCED EQUAL TO SPECTRAL ACCELERATION DIVIDED BY CALCULATED STRUCTURAL FACTORS.
(4) ATC-3 METHOD (SIMILAR TO HUNGARIAN METHOD)	AN EARTHQUAKE CAPACITY RATIO (SEISMIC SHEAR FORCE CAPACITY FOR EXISTING SYSTEM DIVIDED BY A SELECTED STANDARD SEISMIC SHEAR FORCE CAPACITY) IS CALCULATED.
(5) BLUME et al. METHOD	A DAMAGE FUNCTION IS CALCULATED AS A FUNCTION OF EXISTING DUCTILITY, ULTIMATE DUCTILITY, AND AN ECONOMIC SCALE FACTOR.
(6) OLIVEIRA et al. METHOD	A DAMAGE FUNCTION IS DEVELOPED AS A FUNCTION OF DISPLACEMENT FACTORS AND MATERIAL AND STRUCTURAL PARAMETERS.
(7) YAO et al. METHOD	A DAMAGE FUNCTION IS CALCULATED AS A FUNCTION OF CYCLIC COMPRESSIVE CHANGE IN PLASTIC STRAIN, SUBSEQUENT TENSILE CHANGE IN PLASTIC STRAIN, AND NUMBER OF TENSILE LOADS.
(8) DAMAGE NO. 1	DESIGNED FOR HIGH-RISE STRUCTURES, THIS METHOD PRODUCED DAMAGE ESTIMATES FOR FRAME, WALL, GLASS, AND NON-STRUCTURAL COMPONENTS BASED ON INTERSTORY DRIFT CALCULATIONS. MEAN AND STANDARD DEVIATIONS ARE GIVEN FOR DAMAGE RATIOS.
(9) DAMAGE NO. 2	IN ADDITION TO FLOOR-BY-FLOOR STIFFNESS MODELING, BEAMS, COLUMNS, SHEAR PANELS, AND PARTITIONS ARE ALSO MODELED. OUTPUTS AS FOR DAMAGE NO. 1 ARE PRODUCED BASED ON MORE REFINED ANALYSIS.
(10) UMEMURA et al. METHOD (JAPAN) (ALSO VERYET PAPER)	FOR REINFORCED CONCRETE STRUCTURES UP TO SIX STORIES, A PERFORMANCE INDEX IS DEVELOPED BASED ON ULTIMATE HORIZONTAL STRENGTH, DUCTILITY, BUILDING SHAPE, DETERIORATION, AND SEISMICITY (THREE LEVELS OF PROCEDURES).
(11) KALEVRAS METHOD (GREECE)	BASED ON THREE RECENT GREEK EARTHQUAKES, UNDERSTRESS AND OVERSTRESS PARAMETERS ARE USED TO INDICATE DAMAGEABILITY. THIS METHOD HAS NOT YET BEEN CONNECTED TO EMPIRICAL LOSS MODELS.
(12) M.I.T. METHOD(S) (ANAGNOSTOPOLOUS, BIGGS, WONG, ZARNECKI, LAI, BANON AND BIGGS)	ANALYTIC MODELS ARE DEVELOPED FOR A VARIETY OF BUILDINGS, ESPECIALLY HIGHRISE AND REINFORCED CONCRETE FRAMES.

Step 5 - For buildings in which more refined analysis of resistance (and expected loss) is judged necessary and worthwhile structural engineers should use analytic or quasi-analytic techniques.

4.5 Experimental Techniques

4.5.1 Uses

Experimental techniques can be used to define more exactly the material properties of the structure in question or of its structural components, and the dynamic characteristics of the soil-structure system. In some cases, these techniques can be used to reduce seismic rehabilitation costs.

Appendix D provides an account of available experimental techniques.

4.5.2 Procedure

Step 6 - In order to lower possible seismic rehabilitation costs, or to define better the material properties of the soil structure system, experimental analyses on specified components may be undertaken.

In most cases, elaborate experiment will be much too costly to be used here. However, Step 6 may be very useful in increasing the confidence level of the evaluation methods and in lowering mitigation costs.

4.6 Choosing a Potential Damage Evaluation Method

Each of the four techniques, categorization, visual rating and inspection, analytical, and experimental, has strengths and limitations. Table 4-9 outlines the factors that bear upon a choice of method for undertaking pre-earthquake damageability studies.

First, professional validity refers to the acceptability of techniques within the professional community. Many scientific limitations exist to all techniques mentioned (Appendix C outlines general limitations of available categorization techniques). Reliability and accuracy of techniques also may depend on whether or not they are applied to individual structures or only to large groups of structures. Categorization techniques, for example, tend to be much less reliable when applied to individual structures. Moreover, there are many techniques available that may give results that satisfy general scientific curiosity but have little or no bearing on decisions to rehabilitate structures.

Secondly, costs of research per unit examined relate to the obvious fact that there is a tradeoff between ongoing study, or further study, and actual reconstruction. Also, economies of scale may be present if costs per unit are conceived of as part of an overall assessment program in which a very large number of structures are examined. Such costs have been broken down into costs of labor, equipment, administration and overhead. Some assessments may have adverse environmental or safety effects. As an extreme example, shaking a building may require occupants to stay out of the structure and may lead to potential damage. However, other techniques, such as categorization, have no such adverse impacts.

Thirdly, legal, political, social, and technological considerations may enter into choice of a method. For instance, the legality of large-scale inspection programs may be called into question by those who do not want such information to be obtained, or by those who do not want any interference with their businesses or residences.

Also, political considerations affect a damage assessment if budget-conferring bodies for whatever reason choose not to support it or its practical results. Favorable political considerations may enter if such bodies strongly encourage such programs. However, even if appropriations-granting bodies are favorably disposed to earthquake assessment programs, they are likely to be affected by modern bureaucracies. In such bureaucracies, it is not unknown for office holders to have goals separate from those of earthquake safety. Hence, assessment programs may be aided or harmed by bureaucratic implementation and modification.

Social considerations may vary among special interest groups. Certain groups may see little gain from rehabilitation programs. For example, elderly people in Los Angeles may not favor rent increases when they are faced with tight monthly budgets. Commercial institutions may favor or oppose expenditures in safety to the extent that a variety of factors (potential construction costs, business interruptions, default on mortgages, time spent in allowing assessments, etc.) are perceived to be important. Likewise, industrial institutions may regard damageability assessments positively or negatively depending on their ability to allow time for them, the quality of safety engineers already employed, potential costs, etc. In

Table 4-9. Factors in Choosing a Damageability Evaluation Method

I. PROFESSIONAL VALIDITY

A. ACCURACY AND RELIABILITY AS APPLIED TO INDIVIDUAL STRUCTURES.

B. ACCURACY AND RELIABILITY AS APPLIED TO LARGE GROUPS OF STRUCTURES.

C. RELEVANCE OF RESULTS TO A REHABILITATION PROGRAM (NON-SUPERFLUOUS: PROVIDES ADDITIONAL DATA USEFUL FOR ASSESSMENT AND/OR POSSIBLE MITIGATION, PROVIDES OVERALL PERSPECTIVE, ETC.).

II. COSTS/STRUCTURE ANALYZED AS PART OF A REHABILITATION ASSESSMENT PROGRAM

A. LABOR COSTS/UNIT (INCLUDE CONSIDERATIONS ON AVAILABILITY OF LEVEL OF EXPERTISE AND/OR TRAINING REQUIREMENTS TO ENGAGE IN PROGRAM).

B. EQUIPMENT COSTS/UNIT (INCLUDE CAPITAL OUTLAYS, OPERATIONS AND MAINTENANCE, AND AVAILABILITY OF NEEDED EQUIPMENT).

C. ADMINISTRATIVE AND OTHER OVERHEAD COSTS.

D. COSTS OF LOST PRODUCTIVITY, ETC. FOR THOSE IN STRUCTURES ANALYZED BY METHOD.

1. RESIDENTIAL
2. COMMERCIAL
3. INDUSTRIAL
4. GOVERNMENTAL

E. COSTS OF ADVERSE IMPACTS OF DEPLOYMENT OF METHOD.

1. ADVERSE ENVIRONMENTAL IMPACTS
2. ADVERSE SAFETY RISKS

III. LEGAL, SOCIAL, POLITICAL, AND TECHNOLOGICAL ACCEPTABILITY OF USE OF METHOD AND POTENTIAL RESULTS OF METHOD

A. LEGAL ACCEPTABILITY

1. ARE THERE LAWS THAT MIGHT PROHIBIT USE OF METHOD TO INDIVIDUAL STRUCTURES OR GROUPS OF STRUCTURES, OR THAT IN GENERAL MIGHT FORBID SUCH RESEARCH?

2. IS IT POSSIBLE TO DEVELOP ORDINANCES THAT MIGHT BE IMPLEMENTED BASED ON RESULTS OF METHOD?

B. POLITICAL ACCEPTABILITY OF RESEARCH AND RESULTS.

1. BY BUDGET-CONFERRING BODIES

2. BY ADMINISTRATORS AND BUREAUCRATS OF RELEVANT GOVERNMENTAL PROGRAMS

C. SOCIAL ACCEPTABILITY OF RESEARCH AND RESULTS.

1. TO RESIDENTIAL DWELLERS
2. TO COMMERCIAL INSTITUTIONS
3. TO INDUSTRIAL INSTITUTIONS
4. TO GOVERNMENTAL INSTITUTIONS

D. TECHNOLOGICAL ACCEPTABILITY OF RESULTS.

1. TO MANUFACTURERS AND SUPPLIERS
2. TO BUILDING OFFICIALS, INSPECTORS, ETC.
3. TO ARCHITECTS, ENGINEERS, ETC.

Table 4-10. Rating Criteria for Rehabilitation Assessment Techniques

TYPE OF METHOD	SPECIFIC TECHNIQUE	PROFESSIONAL VALIDITY			COSTS/STRUCTURE AS PART OF AN OVERALL ASSESSMENT PROGRAM					
		ACCURACY AND RELIABILITY		PRACTICAL RELEVANCE OF RESULTS	LABOR COSTS/ UNIT	EQUIPMENT COSTS/ UNIT	ADMINISTRATIVE AND OTHER OVERHEAD COSTS/UNIT	COSTS OF LOST PRODUCTION FOR THOSE IN STRUCTURE/UNIT	ADVERSE ENVIRONMENTAL OR SAFETY IMPACTS/UNIT	OVERALL COSTS/ UNIT
		TO SINGLE STRUCTURES	TO GROUPS OF STRUCTURES							

Table 4-10. Rating Criteria for Rehabilitation Assessment Techniques (cont'd)

LEGAL, SOCIAL, POLITICAL, AND TECHNOLOGICAL ACCEPTABILITY OF METHOD OR RESULTS										
LEGAL		POLITICAL		SOCIAL				TECHNOLOGICAL		
RESEARCH IS ALLOWABLE	RESULTS CAN BE DEVELOPED INTO ORDINANCES	BY BUDGET CONFERING BODIES	BY GOVERNMENTAL ADMINISTRATORS AND BUREAUCRATS	TO RESIDENTIAL DWELLERS	TO COMMERCIAL INSTITUTIONS	TO INDUSTRIAL INSTITUTIONS	TO GOVERNMENTAL INSTITUTIONS	TO EDUCATION AND TRAINING INSTITUTIONS	TO ARCHITECTS, ENGINEERS, ETC.	TO MANUFACTURERS, SUPPLIERS, ETC.

governmental institutions, rehabilitation programs may either further extend or retard existing programs for land and building development and use.

Finally, technological considerations strongly impact the viability of an earthquake rehabilitation program. Training and educational institutions may need to add new courses to provide enough personnel for such a program. Architects and engineers may not be properly trained to implement earthquake assessment. Material or other shortages may be expected should a large rehabilitation program be undertaken, and diversion of resources from other programs may also occur.

Table 4-10 can be used to decide which methods of pre-earthquake damageability assessment should be used in a seismic mitigation program.

The table is to be filled out by experts in the Balkan region. (Ratings are to range from 1, the lowest, to 10.) In addition, weights are to be assigned to each rating criterion in Table 4-9. (These weights are to be assigned by the same experts.)

4.7 Relevant Bibliographies

4.7.1 Bibliography of Categorization Techniques

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5. FACTORS DETERMINING THE NEED FOR SEISMIC MITIGATION AND THE NATURE OF SOLUTIONS

5.1 General

This chapter discusses generally the conditions under which government agencies should undertake a seismic mitigation program and the nature of actions to be undertaken. Whereas previous chapters emphasize technical matters, this chapter approaches the seismic policy problem from a more general standpoint, considering technical, economic and social factors of interest. The next chapter then details criteria to be considered. Additional information is given in Appendices C and D, and the analytical treatment of seismic risk is dealt with in Appendix E.

The main factors considered are:

a) the seismic risk;
b) the intervention alternatives;
c) the costs and benefits of service;
d) the correlation with development projects.

5.2 The Seismic Risk

The seismic risk, which represents the expectancy of earthquake-inflicted losses, represents the first major factor to be considered in determining the need for mitigation programs. The seismic risk is determined basically by:

a) the seismic hazard;
b) the vulnerability of the artifacts of man;
c) the nature and exposure of elements at risk.

The seismic hazard, the primary source of seismic risk, is as a rule defined in terms of macrozonation and also microzonation maps and studies. Macrozonation maps define broad zones or contours assumed to be homogeneous with respect to the hazard (comparable maximum intensities with comparable return periods). Microzonation maps or studies pertain to restricted areas that are densely built and populated or else represent the locations for some particularly important structures. These maps provide detailed zones and, wherever possible, specify the expected spectral contents of future strong motions.

For particularly important built environments, it may be suitable to carry out special hazard studies. These examine potential source activity and attenuation laws and specify, on a refined basis, the expected spectral contents of future ground motions. With regard to built ensembles (urban systems, lifelines, infrastructure components), it may be important to carry out hazard studies designed to predict the simultaneous occurrence of strong motions at different sites. Use of earthquake scenarios in order to estimate the possible consequences of the largest earthquakes that are likely to occur will be especially appropriate in order to carry out group risk analyses. The degree of microzonation desirable may depend to a great extent on the severity of hazard. Unless ground failure hazards tend to be especially high, their detailed examination is generally unnecessary in zones affected by lower shaking hazards. As the shaking hazard increases, the identification of other secondary hazards may become more important. The importance of appropriate microzonation for the assessment of risk affecting underground conduits must be stressed here, too.

The vulnerability of buildings and other structures, that is, their proneness to damage when subjected to strong seismic action, represents a dual factor generating seismic risk. Vulnerability depends on the degree of engineering, the quality of materials and workmanship, the gradual deterioration of structures, the degree of damage from past earthquake, other overloadings, or corrosion, and the unsuitable interventions (e.g., removing of walls or columns) that may have occurred previously. As a rule, older buildings tend to be more vulnerable than more recently built ones. Both the likelihood of heavy damage (basically structural damage that may involve collapse), and also the likelihood of apparently light, non-structural, life-threatening damage should be considered. Both aspects are important for subsequent risk estimates and both kinds of likelihood must be expressed in principle in a conditional form, as function of ground motion intensity. The vulnerability characteristics, determined for various types or classes of structures, are not used in current engineering design calculations. However, their use is necessary whenever risk estimates are to be made.

Seismic hazard and vulnerability data make it possible to estimate the specific risk, i.e., the damage expected to occur during a certain subsequent time period. It is important to provide, on one hand, a picture of the likelihood of the most severe damage, and, on the other hand, of the likelihood of less severe damage that may be reasonably expected to occur rather frequently. Earthquake scenarios already referred to make it possible to develop corresponding damage scenarios that lie at the basis of the estimation of losses of various types.

The third category of output data required by risk analyses is represented by the nature and exposure of elements at risk. In comparison with the first two categories, seismic hazard and vulnerability, this category is less tractable especially owing to the various facets of the exposure and possible chain effects, to the psychological effects of disaster impact, etc. It is necessary to include in this category lives of occupants, various kinds of property, activities that can be disrupted, and cultural values. It is also necessary to consider here potential chain effects corresponding to various scenarios. Those losses that can be expressed in monetary terms may eventually be discounted. In contrast, monetary values at risk can significantly vary in time, owing to decrease of functionality or to obsolescence. As a rule, the exposure of elements at risk is not considered explicitly in codes in force. Some differentiation is nevertheless available, given the importance factors specified by most codes. Code specifications customarily pay attention not only to the exposure, but also to buildings or systems whose function is essential for post-disaster response and recovery operations (e.g., hospitals, fire stations, lifelines, critical communication systems).

Data on specific risk and on elements at risk make it possible to estimate the seismic risk, i.e., the losses of various sorts that are more or less likely to occur during a certain future time period. The expectancy of losses should be considered separately for the various kinds of potential losses that are not commensurable, such as losses of lives, monetary losses, losses represented by disruption of vital activities, and losses of cultural values.

The expectancy of losses must be evaluated for each possible strategy or alternative, including the present situation of a building, of an urban system, etc. Risk analysts should thus finally attach some numbers, characterizing the expectancy of losses for the various components referred to (lives, property, etc.), to each alternative.

Calculations required for estimates of specific risk and of risk may be carried out by means of more or less refined technique, using more or less refined data. It is reasonable to adopt a level of sophistication for calculations that agrees with the degree of accuracy and certainty for the various categories of basic data. In order to obtain realistic estimates, it is more important to cover all qualitative aspects than to invest serious efforts in overly sophisticated calculations. The basic assumptions, the basic scenarios referred to, should be the subject of in-depth judgement by experienced staffs.

5.3 The Intervention Alternatives

In relation to the eventual intervention on the existing building stock, decision-makers have two basic variables to consider:

a) the function/occupancy of buildings
b) the seismic resistance of buildings

Measures related to either variable may be used in order to mitigate seismic risks. Changes in function/occupancy will affect basically the elements at risk, whereas such intervention as retrofitting and upgrading will affect basically the vulnerability. Changes in function/ occupancy can include cancellation of use (possibly provisionally), reduction of occupancy, and increase in functionality (correlated with eventual structural upgrading). Change in seismic resistance of buildings may include demolition, removal of some particularly vulnerable parts, repair, strengthening, or even more profound reconstruction (in conjunction with an upgrade in functionality/ occupancy). Since seismic risks are functions of the lifetime of buildings, the timing (more properly the deadlines) of any intervention must be considered simultaneously with the nature and extent of the intervention.

Once carried out, interventions can affect two of the three factors determining the seismic risk, namely, the vulnerability and the exposure of elements at risk. At this stage, it is unrealistic to consider any influence on seismic hazard, which is governed by natural forces that cannot be controlled by man. Any intervention will ultimately influence the seismic risk and it is desirable to attach to each possible solution (including the solution of no intervention) numbers characterizing the expectancy of various losses (of lives, of property, etc.). Note again that any intervention solution is defined also by its timing and that the seismic risk will be directly influenced by the time parameter.

In relation to the intervention alternatives, it is necessary to consider those resources required and the feasibility from several viewpoints: technical (disposal of trained staffs), financial, and material (allocation of materials, components etc. required). Feasible solutions will be characterized not only by certain costs, that will be expressed in financial terms, but possibly also in terms of some material resources (e.g., energy or steel) that may be scarce under certain circumstances.

5.4 Costs and Benefits of Service

The use and even the mere existence of buildings implies some costs (apart from the losses involved by the risk) and some benefits. Maintenance costs and the value of land occupied contribute to the costs. In contrast, benefits may be expressed in monetary terms (such as rent), in terms of lives saved (like the activity of a hospital or of a vital communication line), etc.

The costs and benefits of service will be different for the present state of a building as contrasted to the post-intervention state. It is necessary to attach numbers also from this viewpoint to the various possible strategies or solutions (including the solution corresponding to continuing the present state of a building). These numbers should correspond to the various subcategories of costs and benefits.

5.5 Cost-Benefit Analyses

The data provided by the seismic risk analysis, along with data on costs and resources required by the various intervention strategies and costs and benefits implied by the service of various buildings and other elements of urban systems, regional networks etc., must be synthesized in order to evaluate the various possible strategies of mitigation of seismic risk as compared to the present situation. It is reasonable to consider the different incommensurable elements, like human lives (expectancy of lives lost due to seismic events/expectancy of lives saved due to the appropriate functioning of some buildings like hospitals, nurseries etc.), monetary losses or income (expectancy of the direct and indirect losses due to seismic events/expectancy of benefits from rent, productive activities etc.) and possibly other elements. The estimated losses/benefits expressed separately for the different elements (human lives, money etc.) should be attached as numbers to each of the possible strategies of action, as determined for each of them with explicit consideration of the timing of action. Some statements on the degree of accuracy and confidence of these estimates may be also required.

The estimates referred to will provide an important basis for decision making at the level of cities, regions etc. Staffs involved in decision making should use the cost-benefit estimates along with judgment, including the consideration of social attitudes and reactions towards losses, benefits, and the different trends of development.

Seismic design codes in force do not include an explicit concern for cost-benefit analyses. The results of these analyses, carried out more or less explicitly, nevertheless lie at the basis of the values selected for design forces, safety factors, etc. Code provisions may be amended in relation to mitigation activities, given factors that tend to alter the assumptions on which codes are based (modification of subsequent lifetime of structures, existence of apparent or hidden damage for existing buildings, special features of exposure and importance of buildings, etc.).

5.6 The Correlation with Development Projects

Seismic risk mitigation activities must be closely linked with the general development projects within a region, system, etc. Systematic mitigation involves huge resources and those resources cannot be allocated unless the spending is justified by long-term nonseismic benefits as well. In the pre-earthquake situation, seismic risk mitigation projects are reasonable elements of development projects. The implications of actions related to mitigation of losses must be considered in the allocation of resources for the successive steps of the development projects. Resources allocated for general development activities must be planned in a way that is compatible with the needs of intervention on the highest priority elements, such as highly vulnerable structures, critical facilities, etc. Conversely, solutions adopted for intervention must be harmonized with the more general strategies of development, through consequent adoption of the required solutions of demolition, upgrading, change of function or occupancy, etc.

Post-earthquake situations always have a considerable impact on development plans. A post-disaster situation should be a major incentive for rethinking general patterns. The need to remove much of the building stock, along with eventual major aid from national or international resources, may provide opportunities to modernize radically the building stock and so to affect the economic and social fabric of a region.

5.7 Management of Mitigation Activities

The management of mitigation activities must cover the successive steps of study and action and also provide for and adequately schedule the resources needed.

The first major step of mitigation activities is represented by decision making considered in a broad sense. This includes definition of the general scope of activities, appropriate planning of initial tasks, activities intended to provide basic information regarding seismic hazard, vulnerability and exposure or elements at risk, analyses required by estimation, evaluation and assessment of specific risk and risk (individual risk, group risk), gathering complementary data required by cost-benefit analysis, and selection of strategies and solutions of intervention, including priority indices and timing.

The second major step of mitigation activities is implementation, considered again in a broad sense. This includes detailed design and planning activities (which must be carried out in an integrated manner within the framework of general design and planning activities required by development projects), construction work, and the necessary control activities.

The basic resources to be provided for the various activities referred to consist essentially of qualified personnel, information and regulatory bases, building materials and manufactured components, and construction equipment. The most specific and sometimes scarce resources are the personnel and also the software required.

The following sorts of qualified people are needed: seismologists and geologists, architects and engineers, mathematicians and analysts, economists and sociologists, policy administrators and policy makers, building officials and inspectors, and construction technicians and workers. Emphasis must be placed on the special training needs raised by the complex problem of mitigation of seismic risks and by the need for interdisciplinary work.

A few comments were made previously in relation to the differences between decision-making based on code provisions as opposed to decision-making based on cost-benefit analyses.

In some cases, current earthquake-resistant design codes may be used as reference for rehabilitation activities. This may become necessary when no special provisions are available.

Code provisions may be amended in order to allow a reasonable degree of safety through rehabilitation measures. In this regard, Japanese standards for evaluation and rehabilitation of existing buildings prescribe increased design factors for the verification of existing structures in order to account for the unfavorable effects of hidden damage. In contrast, a diversity of design factors should be reasonably adopted, considering the different planned subsequent lifetimes of different structures. Some guidelines in this connection are given in Appendix E (Section E.8.4).

Use of adequate earthquake resistant design codes in new construction is a much more cost-effective way to implement earthquake safety programs than seismic retrofit of existing structures. Still, seismic rehabilitation programs are needed, given the high risks that currently affect much of the existing building stock in various countries.

5.8 Conclusion

The multifaceted and complex nature of those activities required by the decision making in relation to mitigation activities and by the implementation of decisions adopted is clear throughout this chapter. More detailed data on some aspects of the problem are provided by the next chapter and by the Appendices, as referred to in Section 5.1.

The lack of broad experience in these activities must be emphasized here along with the pioneering nature of such projects designed to fulfill needs at a national, regional or urban level. Nevertheless, seismic risk mitigation activities should be considered basically as permanent, ongoing activities, given general development, and also changes in technical knowledge and in the economic and social environment, and the need to update concepts, strategies and plans.

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6. SOCIO-ECONOMIC ANALYSES FOR MITIGATION OF SEISMIC RISK

6.1 Introduction

This chapter, to be used in conjunction with Chapters 4 and 5, outlines methods and criteria for determining tolerable/acceptable risk levels. Those levels so determined may be used to decide whether or not a given building should be seismically retrofitted, and to what level. Likewise, these methods may be used in conjunction with other criteria to determine whether or not further assessments are needed.

6.2 Cost-Benefit Methods

These methods compare the monetary costs of given projects with their expected monetary benefits. Costs of a seismic rehabilitation program are primarily those costs to repair buildings to a given level of seismic resistance. Costs of seismic assessment procedures have been indicated in Chapter 4. Benefits of a seismic rehabilitation program are expected reductions in (not eliminations of) economic losses and deaths from building damage or from associated losses such as lost production, business interruption, alteration of services in a given community, and unemployment.

There are two almost identical ways to perform benefit-cost analyses. First, the risk analyst may develop benefit-cost ratios comparing the costs of rehabilitation with expected benefits. Those ratios that fall below unity generally suggest that alternative investments would be economically superior. When such ratios exceed unity, programs may be compared with others to determine priorities. The second method regards costs as "losses," along with expected damage and economic losses from business interruption. Using this method, the analyst finds the mix of actions that minimizes losses. Figure 6-1 illustrates this type of analysis. As levels of seismic resistance increase, expected losses decrease, and at some point, the total costs reach a minimum. In Figure 6-1, a star indicates that optimal point. However, in the actual construction of such curves, it is important to recognize threshold effects. It is possible that losses would not continuously decline, but would only be expected to decline once a significant amount had been spent on increased seismic resistance.

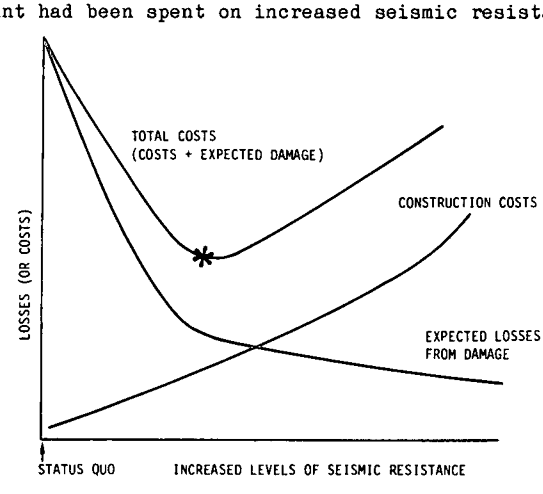


Figure 6-1. Illustrative Curves for Minimum Loss Analysis

Numerous objections to cost-benefit analyses exist, but only two are significant here. First, some benefits are not clearly reducible to monetary terms. Especially suspect is the "monetary" value of life. However, if the analyst precludes assessing the value of a life in monetary terms, he should either not use benefit-cost analyses where safety is a significant issue or should recognize that benefits are thereby underestimated. In contrast, if the analyst lets the value of life be "infinite," the resulting proposed expenditures for health and safety programs would bankrupt the nation. The second objection to cost-benefit analysis is that choice of discount factors is very controversial, especially when the relevant exposure period is long. In analyzing the present value of benefits, one may either choose to add all benefits (over a limited time period) or to discount future benefits. If benefits are discounted, choice of discount rates may determine whether or not a program is judged to be economically sound.

6.3 Risk/Benefit Methods (Individual Risk Criteria)

The methods used in risk/benefit analysis tend to be the same as those used in benefit/cost analysis except that there is no explicit monetary value placed on lives. Instead, various actions are evaluated based on costs per lives saved. Once a cost per life criterion is adopted, various levels can be set at which an activity is deemed very worthwhile, acceptable, tolerable, or unacceptable. Table 6-1 contains a list of activities that have been evaluated by such criteria.

For purposes of evaluating earthquake safety programs, the following values have been suggested:

40GNP/per. cap/life saved

where a 40 year social average life-time has been considered.

6.4 Comparative Risk, Including Group Risk

In a previous section, 6.2, benefit-cost analysis methods were discussed which reduced all costs and benefits to the same monetary unit and thereby treated them equally. However, there are many reasons why the large single disaster which strongly impacts a particular group of people or a particular location will attract the interest of the policy maker. Preventing the deaths of 100 people in a single building may be easier than preventing the deaths of 100 widely scattered people.

Figure 6-2 suggests a number of distinctions that may be made for various activities. For purposes of earthquake analysis, the expected risk is calculated as a percent of the number of people exposed to the hazard. In order to avoid double counting of people, it may be advisable to develop such data based on large categories of structures. The annual acceptable risk level per exposure for earthquakes has been suggested to be

10^{-7} per individual

This criterion can be used to determine whether or not existing structures are presently at an acceptable risk level.

In the past, research efforts and public safety programs have given more attention to events or accidents having greater single losses (however low the probability of occurrence) than to events or accidents having low average expected deaths. Figure 6-3 illustrates how frequency of occurrence can be contrasted to severity of occurrence. Figure 6-4 illustrates differ-

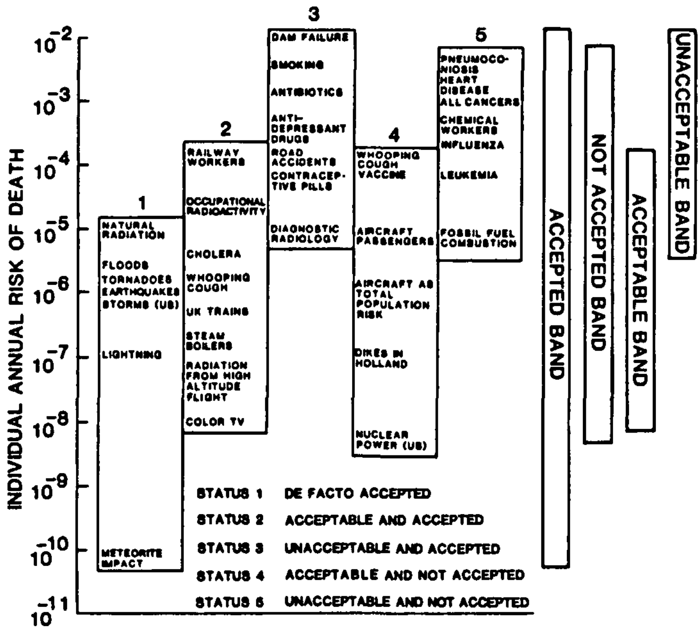
Table 6-1. Cost Per Fatality Averted (1975 Dollars) Implied by Various Societal Activities (Left Column) and Cost Per 20 Years of Added Life Expectancy (Right Column) (Reproduced in Philipson, 1982)

ITEM	$ PER FATALITY AVERTED	$ 20 YR LIFE EXPECTANCY
MEDICAL SCREENING AND CARE		
CERVICAL CANCER	25,000	13,000
BREAST CANCER	80,000	60,000
LUNG CANCER	70,000	70,000
COLORECTAL CANCER:		
FECAL BLOOD TESTS	10,000	10,000
PROCTOSCOPY	30,000	30,000
MULTIPLE SCREENING	26,000	20,000
HYPERTENSION CONTROL	75,000	75,000
KIDNEY DIALYSIS	200,000	440,000
MOBILE INTENSIVE CARE UNITS	30,000	75,000
TRAFFIC SAFETY		
AUTO SAFETY EQUIPMENT - 1966-70	130,000	65,000
STEERING COLUMN IMPROVEMENT	100,000	50,000
AIR BAGS(DRIVER ONLY)	320,000	160,000
TIRE INSPECTION	400,000	200,000
RESCUE HELICOPTERS	65,000	33,000
PASSIVE 3-POINT HARNESS	250,000	125,000
PASSIVE TORSO BELT-KNEE BAR	110,000	55,000
DRIVER EDUCATION	90,000	45,000
HIGHWAYS CONSTRUC.-MAINT. PRACTICE	20,000	10,000
REGULATORY AND WARNING SIGNS	34,000	17,000
GUARDRAIL IMPROVEMENTS	34,000	17,000
SKID RESISTANCE	42,000	21,000
BRIDGE RAILS AND PARAPETS	46,000	23,000
WRONG WAY ENTRY AVOIDANCE	50,000	25,000
IMPACT ABSORBING ROADSIDE DEVICE	108,000	54,000
BREAKAWAY SIGN LIGHTING POSTS	116,000	58,000
MEDIAN BARRIER IMPROVEMENT	228,000	114,000
CLEAR ROADSIDE RECOVERY AREA	284,000	142,000
MISCELLANEOUS NON-RADIATION		
EXPANDED IMMUNIZATION IN INDONESIA	100	50
FOOD FOR OVERSEAS RELIEF	5,300	2,500
SULFUR SCRUBBERS IN POWER PLANTS	500,000	1,000,000
SMOKE ALARMS IN HOMES	250,000	170,000
HIGHER PAY FOR RISKY JOBS	260,000	150,000
COAL MINE SAFETY	22,000,000	13,000,000
OTHER MINE SAFETY	34,000,000	20,000,000
COKE FUME STANDARDS	4,500,000	2,500,000
AIR FORCE PILOT SAFETY	2,000,000	1,000,000
CIVILIAN AIRCRAFT (FRANCE)	1,200,000	600,000
RADIATION RELATED ACTIVITIES		
RADIUM IN DRINKING WATER	2,500,000	2,500,000
MEDICAL X-RAY EQUIPMENT	3,600	3,600
ICRP RECOMMENDATIONS	320,000	320,000
OMB GUIDELINES	7,000,000	7,000,000
RADWASTE PRACTICE - GENERAL	10,000,000	10,000,000
RADWASTE PRACTICE (3)I	100,000,000	100,000,000
DEFENSE HIGH LEVEL WASTE	200,000,000	200,000,000
CIVILIAN HIGH LEVEL WASTE		
NO DISCOUNTING	18,000,000	18,000,000
DISCOUNTING (1% YEAR)	1,000,000,000	1,000,000,000

Table 6-2. Expected Deaths at Given Intensities for a Given Building Category Over Years for a Given Exposed Population P (P is the total number exposed in a nation, or city, or given class of buildings, etc.)

	INTENSITY						
	VI	VII	VIII	IX	X	XI	XII
EXPECTED DEATHS	D_{VI}	D_{VII}	D_{VIII}	D_{IX}	D_X	D_{XI}	D_{XII}

ences in research funding for large single risk versus small single risk events. As a result, use of <u>group risk</u> criteria is an option for assessing earthquake safety programs. These criteria emphasize the aversion of a large-scale narrowly-focused disaster.



*From Nuclear News, September 1980

Figure 6-2. Histogram of Individual Annual Risk of Death for Risk Status 1 to 5 (Reproduced in Philipson, 1982)

Group risk criteria can be introduced into benefit-cost calculations by making adjustments in Table 4-2. Based on that table, for given classes of buildings, the expected number of deaths at given intensities is the exposure times each of the ratios at the intensities. Hence, the analyst can derive a new table, Table 6-2, whose form is found on the previous page.

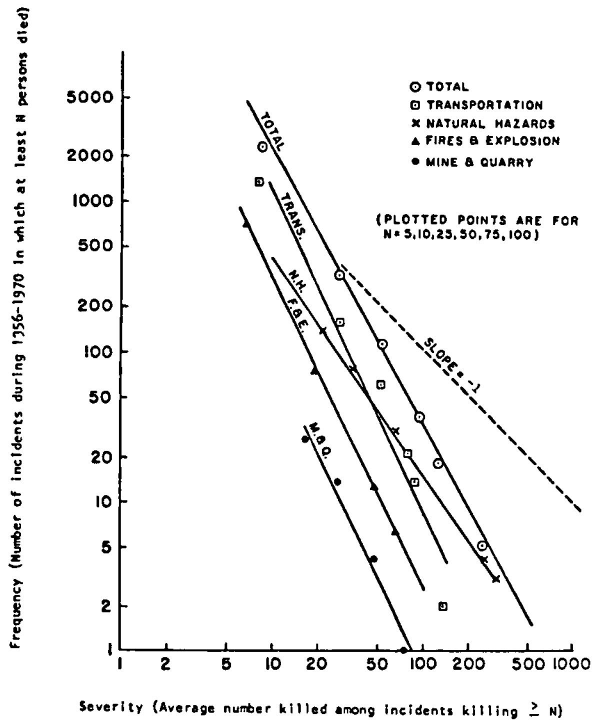
If the analyst were merely using individual risk criteria, then the expected number of annual deaths would be

$$\sum_{j=6}^{12} D_j \; N_j$$

However, to incorporate group risk criteria, the analyst may use an exponent α, between 1 and 3, and arrive at the following <u>policy-adjusted</u> annual expected number of deaths

$$\sum_{j=6}^{12} (D_j)^{\alpha} \; N_j$$

An analogous technique can be used to evaluate economic losses.



Note: Transportation includes commercial and civil aviation, motor vehicle and bus traffic, railroad, and water transport. Military aviation is excluded. Natural hazards include tornadoes, hurricanes, and floods. The only major earthquake during this period occurred in Anchorage, which is outside the study area.

Figure 6-3. Cumulative Accident Frequency Vs. Severity Distribution Continental United States, 1956-1970 (cited in Philipson, 1982)

The reader should note that these group risk criteria are introduced merely as a convenience to simplify assessments in order to incorporate rapidly possible secondary losses, political costs, etc. of larger catastrophes. Choice of α is here left optional. In addition, the outputs of this analysis will depend greatly on the population exposed. Results will be different if one applies this to 1000 people all at once as opposed to 100 people in each of ten different buildings each analyzed separately. Hence, α too may vary with exposures chosen. For these reasons, it is suggested that these simplified methods be examined by users and that choices of α and of exposures appropriate by mode based on such examinations.

6.5 Perceived Risk

The policy-maker working with public safety programs should realize that, for many reasons, perceived risk tends to differ from actual risk. In other words, people tend to think that some risks are greater and other risks are lower than they actually are. For instance, for short periods people may worry greatly about very large earthquakes as opposed to smaller ones that may still be very damaging and also have much higher probabilities of occur-

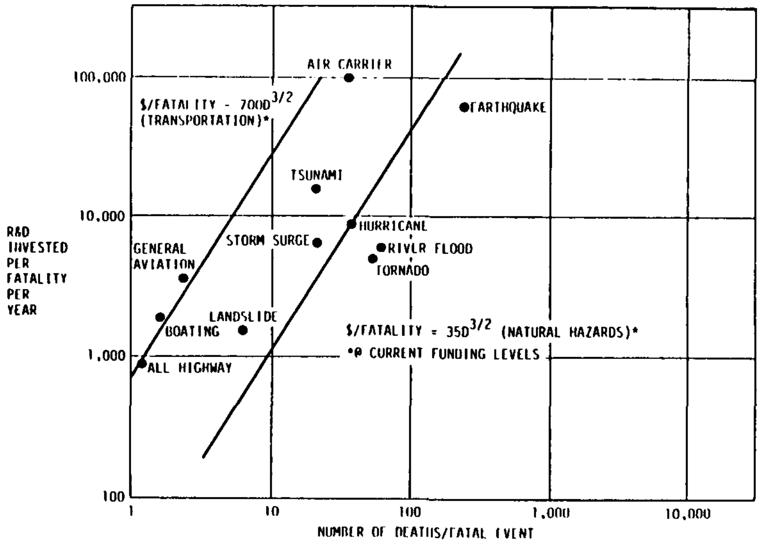


Figure 6-4. Safety Research and Development Funding Levels for U.S. Government Agencies During 1973 Plotted Vs. Fatalities Per Fatal Accident for Various Transportation Modes and Natural Hazards. (Source: J.H. Wiggins, 1978)

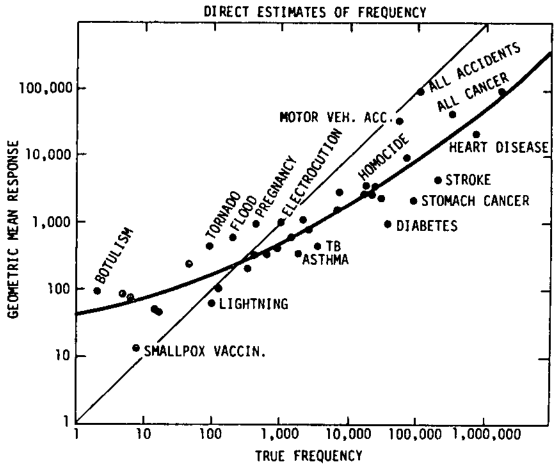
rence. Inasmuch as misperceptions can play a major role in policy formation, it may be necessary to compensate for them.

Figure 6-5 presents one possible pattern of the differences between perceived risk and actual risk: the overestimation of infrequent events in risk perception. Numerous other errors likewise can occur that may tend to favor some programs over others.

6.6 Recent Practices Concerned with Rehabilitation of Existing Structural Buildings

Concerned with the possibility of major losses following an earthquake, the City of Long Beach began to take a hard look at some 900 buildings erected prior to 1933. Uppermost on the inspection list were structures of unreinforced masonry walls with wood floors and wood roofs since these buildings are likely to shake apart first. By 1970, the City of Long Beach had condemned 118 of these buildings. Of these, 46 were demolished and 20 were repaired. The owners of the remaining buildings, particularly of offices and apartments, brought a law suit against the city because they felt that to abandon or greatly strengthen their buildings would be economically disastrous.

At this stage, it was necessary to examine the earthquake-resistant provisions in the city's building code (Wiggins 1970, 1972). It was determined that, at the one extreme, the city could do absolutely nothing to strengthen existing hazardous buildings. Such a course of action would only succeed in averting fire and life-loss if under the unlikely scenario that no intense earthquakes occurred during the remaining life of the buildings in question. At the other extreme, the city could order a wholesale demolition or strengthening of otherwise valuable structures. This would be a prudent course only under the equally improbable situation that an intense earth-



Note - The straight line represents accurate estimation. The curved line fits the subject's mean responses and shows a primary bias of overestimation of infrequent events and underestimation of frequent events. Deviations from the curved line were quite consistent for different groups of subjects and represent secondary biases. These secondary biases are emphasized in the text. Source: Lichtenstein et al. (in press).

Figure 6-5. Perceived Versus True Frequencies of Hazardous Events [H.3] (Reproduced in Philipson, 1982)

quake struck Long Beach in the near future. Both complete inaction, and a sweeping order to immediately demolish or strengthen all old buildings would create economic problems. The goal, therefore, was to devise a reasonable middle course that could be approved by the legislative body and officials of the city.

As a result, in 1971, the City of Long Beach adopted Ordinance C-4950, "Earthquake Hazard Regulations for Rehabilitation of Existing Structures within the "City" (Subdivision 80, Long Beach Municipal Code). This ordinance has several unique features which are important in reducing the risk from existing structural hazards.

The Long Beach ordinance rationally interrelated zoning and building codes. It interrelated the citizens, the elected representatives, and the career civil servants. All were considered part of the policy planning and implementation system necessary to ameliorate structural hazards.

Specifically, the Long Beach ordinance:

(1) coupled land use planning with a building design ordinance by requiring different strength designs in different areas of the city,

(2) required the City Council to make a decision on Policy Risk Level (in this case, death risk),

(3) prescribed performance standards for repairing or rehabilitating existing structures,

(4) provided for owner options on structure life and human exposure,

(5) provided a legal means for demolishing a structure at the end of its life as selected by the owner. The demolition date is attached to the deed,

(6) recognized imperfections by requesting funds to provide for soil dynamics investigation to upgrade the hazard maps,

(7) provided a uniform seismic risk rating system for the building official along with a priority scheme for building examinations,

(8) allowed new materials and designs affecting the earthquake resistance to be reviewed for purposes of adjusting the code values, thus lowering costs with state-of-the-art capabilities, and

(9) provided a means to dynamically analyze structures and site conditions so that individual circumstances can reduce code requirements, if justified.

The Long Beach experience has been examined here because it provides a precedent for the Balanced Risk Methodology which also has been the basis of the analysis for the City of Los Angeles.

The City of Santa Rosa adopted a seismic safety ordinance following the 1969 Santa Rosa earthquake when 85 old unreinforced masonry structures were damaged. After that event, H.J. Degenkolb and Associates prepared a report entitled "Earthquake Safety Investigation Criteria for City of Santa Rosa." Therein they described a time phasing and de facto importance factor criteria using the 1958 Santa Rosa earthquake code for new buildings as a reference. In effect, the Santa Rosa code prescribed that within a few years all buildings built before the first earthquake ordinance comply in some degree with its provisions.

The Santa Rosa ordinance contrasts with the Long Beach ordinance in several respects:

(1) the concept of risk was not treated and thus not decided upon by City Council, and

(2) options for owners regarding dynamic analysis, occupancy changes, lifetime limitations, etc. were not presented.

On January 18, 1973, the City of New York approved Local Law No. 5 entitled "Fire Safety Requirements and Controls." That law requires that existing high rise business buildings provide one-third compartmentation within five years, two-thirds compartmentation within ten years, and the total compartmentation within 15 years. An existing building is defined as "an office building 100 feet or more in height or a building classified as occupancy group E, 100 feet or more in height (which has been or is recently being built)."

All of the provisions pertaining to delays in the total enforcement of the new requirements were included as a means of overcoming certain objections to the new ordinance. A gradual implementation procedure recognizes the impracticality of attempting full-scale and immediate enforcement of retroactive construction requirements of this scope.

In 1974, a seismic safety element study for the City of Los Angeles was completed by J.H. Wiggins Company. However, it was not until after years of deliberation that the Los Angeles City Council passed an ordinance (Ordinance No. 154, 807) on January 7, 1981 that will require rehabilitation of approximately 7,863 unreinforced masonry buildings constructed before 1933. That ordinance is found in Appendix F.

6.7 Hypothetical Examples (illustrative only)

6.7.1 Illustrative Data Developed

Consider an unreinforced masonry structure of 1400 square meters and with a maximum occupancy of 300 people. The building is in category θ, equivalent to 148 points on the Wiggins et al. field inspection and rating technique. For about $160 per square meter the building can be seismically rehabilitated to category θ', or about 50 points on the Wiggins et al. inspection and rating technique. The replacement cost has been determined to be $850 per square meter. From seismic data, the following annual site-specific frequencies of given (MSK or MMI) intensities have been calculated:

INTENSITY	XII	XI	X	IX	VIII	VII	VI
EXPECTED ANNUAL SITE FREQUENCY	$1.4 \cdot 10^{-5}$	$4.5 \cdot 10^{-5}$	$1.4 \cdot 10^{-4}$	$4.6 \cdot 10^{-4}$	$1.48 \cdot 10^{-3}$	$4.72 \cdot 10^{-3}$	$1.51 \cdot 10^{-2}$

Categories θ and θ' have the following expected economic losses (as percentages of replacement cost) at given intensities:

INTENSITY	XII	XI	X	IX	VIII	VII	VI
1. CATEGORY θ	1.0	1.0	.5	.35	.25	.15	.04
2. CATEGORY θ'	.7	.5	.2	.1	.05	.02	0

Expected reductions in economic losses therefore involve subtraction of line 2 from line 1, or

INTENSITY	XII	XI	X	IX	VIII	VII	VI
REDUCTIONS IN ECONOMIC LOSSES	.3	.5	.3	.25	.2	.13	.04

Expected annual economic reductions are thus equal to expected intensity frequencies times expected reductions, or

$$(.3)(1.4 \cdot 10^{-5}) + (.5)(4.5 \cdot 10^{-5}) + (.3)(1.4 \cdot 10^{-4}) + (.25)(4.6 \cdot 10^{-4}) + (.2)(1.48 \cdot 10^{-3}) + (.13)(4.72 \cdot 10^{-3}) + (.04)(1.51 \cdot 10^{-2}) = 1.7 \cdot 10^{-3}$$

(as a percent of replacement cost).

The replacement cost is 1400 square meters times \$850 per square meter, or approximately \$1.2 million dollars. Hence, the annual expected reduction in economic losses is

$$(\$1.2 \cdot 10^{6})\ (1.7 \cdot 10^{-3}) = \$2040$$

Categories θ and θ' have the following expected lives lost at given intensities (per 100 people exposed):

INTENSITY	XII	XI	X	IX	VIII
1. CATEGORY θ	6	4	2	1	.3
2. CATEGORY θ'	.2	1	.7	.3	.05
REDUCTIONS (1.-2.)	4	3	1.3	.7	.25

Again, using the expected annual intensity frequencies, one obtains

Annual deaths per 100 people exposed in existing building

$$(\text{Category } \theta) = 6(1.4 \cdot 10^{-5}) + 4(4.5 \cdot 10^{-5}) + 2(1.4 \cdot 10^{-4}) +$$
$$1(4.6 \cdot 10^{-4}) + 0.3(1.48 \cdot 10^{-3}) = 1.45 \cdot 10^{-3}$$

annual deaths per 100 people exposed

Annual reduction in deaths per 100 people

$$(\text{from Category } \theta \text{ to } \theta') = 4(1.4 \cdot 10^{-5}) + 3(4.5 \cdot 10^{-5}) +$$
$$1.3(1.4 \cdot 10^{-4}) + 0.7(4.6 \cdot 10^{-4}) + 0.25(1.48 \cdot 10^{-3}) = 1.07 \cdot 10^{-3}$$

Since there are a maximum of 300 people exposed, then the present annual death rate per exposure is

$$(3)1.45 \cdot 10^{-3} = 4.35 \cdot 10^{-3}$$

One may use decision methods in Section 6 to determine whether this current level is acceptable, unacceptable, tolerable, intolerable, etc.

A risk analysis can then be done. Suppose that the building is expected to last another 30 years, and that benefits are not discounted. Then costs minus benefits are \$224,000 - 30 (\$2,040), or \$162,800.

Reduction in lives lost is (300 people exposed) ($1.07 \cdot 10^{-3}$ per 100 people exposed), or $3.21 \cdot 10^{-3}$ annually. Overall, such a reduction is 30 years $\cdot 3.21 \cdot 10^{-3}$, or $9.72 \cdot 10^{-2}$.

Hence, cost per life saved is \$163,800/$9.72 \cdot 10^{-2}$ = \$1,670,000. The policy maker may decide at this point what action is appropriate.

6.8 Short Bibliography (See 1.4 as well)

1. Philipson, Lloyd L., "A Review of Risk Evaluation Approaches," prepared for NSF Under Grant No. PRA-8007228, (Redondo Beach, CA: J.H. Wiggins Company, March 1982).

2. Sarin, Rakesh Kumar, "A Social Decision Analysis of Earthquake Safety Problem: The Case of Existing Los Angeles Buildings," (Los Angeles, CA: UCLA Graduate School of Management, 1982).

7. A MODEL CODE FOR THE EARTHQUAKE REHABILITATION OF EXISTING STRUCTURES

In light of the preceding discussion it was deemed desirable by the committee that a model code be presented in this document which could be adopted by the country, state or other legal entity within the country as a legal means and method for implementing the desired level of earthquake rehabilitation. Accordingly, the following code can be adopted in whole or in part or used as a beginning model for the development of yet another geopolitically correct code. The model presented below has been adopted as a starting point by both the cities of Long Beach and Los Angeles, California, USA. Therefore, these principles have been geopolitically implemented to some degree.

It is necessary to acknowledge in the development of any model earthquake rehabilitation code that the physics of the earth and the earthquake itself does not recognize geopolitical boundaries, or various political systems. Damage and loss will surely occur at some future time in highly seismic zones unless effective and timely action is taken, at some acceptable cost, to reduce the effects of earthquake actions.

The following code is written generally for the country/state/city jurisdiction in which the model code is to be applied. Names applying to the jurisdiction are left blank. Further, we refer to building owner, building official, building department, etc. from time to time. This is done recognizing that every country has different political nomenclature and ownership rules. Simply insert your own jurisdiction's political descriptors where necessary.

The code represents a solid implementation act balancing the perspective of individuals about their structures with the physical facts of life and that requires cities which are principally developed respond to the existing earthquake risk problem. It also creates a minimum time period for abatement of the worst risk of about two years from date of code passage.

7.1 Implementation Program - An Ordinance for Mitigating the Earthquake Risk Within ____________________

Section 1 - Purposes:

The purposes of this code are to define:

(1) A table of lateral force resisting capacities determined by structural life and importance factors which shall be used for the design of new structures, as well as the strengthening of existing structures within ____________________. These capacities are a function of those relevant factors which, under the present state of earthquake technology, are believed to substantially influence the acceptable earthquake risk level associated with ____________.

(2) A uniform, systematic and practical procedure for ascertaining the earthquake-generated risk associated with existing structures within ____________________ as a function of those relevant factors which, under the present state of earthquake damage assessment knowledge, are believed to substantially influence said risk.

(3) A flexible procedure for reducing the actual risk presented under current conditions by the affected existing structure to the acceptable risk level. This procedure involves owner options relative to cost-loss relationships involving seismic risk.

(4) Procedures for more completely ascertaining the ground rupture hazards beneath a proposed or existing structure site and developing the design criteria for said site and structure.

(5) The format for an Appeals and Technical Advisory Board for Seismic Design and Planning whose responsibility would be to hear questions of interpretation of this code and to advise appropriate ________ departments of new information regarding seismic safety developments.

It is not the purpose of this ordinance to preclude the assessment of other hazards which may involve fire, exit, plumbing, electrical, etc., or other problems within existing buildings.

Section 2 - Legislative Background:

In enacting this code, the ____________ recognizes the competing, and often conflicting, considerations involved in applying to existing structures (built in good faith in accordance with building codes in force at the time of their construction) more informed, and sometimes more demanding, requirements to promote the public health, safety and well-being. The ___________ enacts this code with the objective of striking a reasonable balance between these various considerations, recognizing that while complete inaction with regard to the earthquake risk for existing structures could, in retrospect, prove disastrous, sweeping drastic action could prove, in retrospect, to have been equally ill-advised. The _____________ recognizes that neither the timing, nor the magnitude, nor the location of an earthquake can be accurately predicted under present technology, but that a scientifically defensible probability may be assigned to the prospect of the occurrence of an earthquake within a given time period which exhibits sufficient intensity and is so located as to produce intolerable stresses on various existing types of structures in the ________________. The ___________________ further recognizes that the demolition or repair of existing structures which, except for earthquake or some other hazard not considered herein, are acceptably safe and useful imposes a substantial hardship on not only the owners of such structures, but also on the tenants and/or users of such structures and on the _______ itself.

Accordingly, between the extreme of doing nothing on the assumption that no damagingly intense earthquake stresses will be experienced during what would otherwise be the normal life of an existing structure, and the opposite extreme of wholesale demolition of otherwise valuable structures upon the similarly improbable assumption that damagingly intense earthquake stresses will occur within the very near future, the ______________ determines to pursue a middle course embodying the selection of an acceptable level of seismic risk and requiring that existing, as well as new structures, accommodate to the level of risk selected.

Section 3 - Scope:

This code shall apply to all artificial structures located within ________.

Section 4 - Prima Facie Hazard Grading:

The building official shall inspect and grade the structures covered by this code to determine the relative prima facie earthquake risk associated with same. The grading shall be conducted using The (Building Inspection Procedure selected from Chapter 4, Appendix C).

Section 5 - Priority of Grading:

In grading the structures subject to this ordinance, the building official shall, in general, grade buildings according to the following priorities. The priorities will be ignored only when, in the opinion of the building official, special hazards exist which warrant an immediate inspection. (Each country, state or other jurisdiction may herein insert its own priority of grading with worst being first and best being last. The examples below refer to buildings defined in the Los Angeles Building Code.)

(1) Type III buildings which utilize unreinforced masonry bearing walls and exhibit poor quality mortar shall be graded first as a group.

(2) Type IV and V buildings with unreinforced masonry veneer, unreinforced non-bearing masonry walls or partitions, poor quality mortar, and poorly anchored bracing systems.

(3) Type III buildings with reinforced concrete and reinforced masonry bearing walls and wall openings with an aggregate area exceeding 50% of the area of one or more of such walls.

(4) Type I and II tall structures with unreinforced masonry curtain and filler walls, and poor quality mortar.

(5) All other types within each of the aforesaid groups of buildings. Individual buildings shall be graded on a priority system corresponding to their occupancy ratings with Importance Factor 2 Buildings being rated first, Importance Factor 3 Buildings being rated second, etc. (see Table II, Section 7).

Section 6 - Calculation of Actual Lateral Force Withstanding Capability, V_a:

In determining the actual lateral force carrying capability, V_a, of a particular structure, the engineer shall calculate the maximum lateral force which the building can be subjected to along any horizontal axis before major structural failure.

Major structural failure shall include, but not be limited to, complete collapse of the structure; the substantial collapse of one or more exterior walls, whether of bearing or non-bearing type; the substantial collapse of one or more heavy interior walls, whether of bearing or non-bearing type; the substantial collapse of any floor or ceiling in the building; the dislodging or shedding of heavy material such as masonry, decorative trim, signs, parapets, fire escapes, air-conditioning equipment, etc.; and any other rapid degradation of structural integrity which may unreasonably endanger the life or health of persons in, on, or about the structure at the time of a seismic event which would exceed V_a in intensity. In determining the lateral force withstanding capability, the engineer shall make appropriate in situ measurements to determine approximate strength of mortar, shear connections, moment connections and materials used in the main body of frames, floors, walls, diaphrams and appendages.

Section 7 - Calculation of Minimum Tolerable Lateral Force Carrying Capacity

(1) The minimum lateral force carrying capacity of a structure shall be computed using the following formula

$$V = Z\ K\ C\ S\ I\ W/\ T^{1/3}$$

Note: (a) This value is to be used in elastic design where safety factors on the materials and connections used yield a conservative combined structural system safety factor of four (4). For local codes whose combined structure system safety factor is more or less than four (4), appropriate adjustments in the resulting value of V must be made.

(b) In no case shall V be less than 80 percent of the value computed in the above formula when dynamic site specific and/or structure specific investigations are used. These are outlined in Section 16.

where Z = 1.5 for Zone 4; 1 for Zone 3; 1/2 for Zone 2; and 1/4 for Zone 1. (Each city/state/country must grade themselves by zone. For purposes of this code a 475 year return period results in the following zone "g" levels: Zone 4 = 60%g; Zone 3 = 40%g; Zone 2 = 20%g; Zone 1 = 10%g.)

C is given in Table 7-1, three possible alternatives are presented for selection by the _______________.

Exception: V shall not be greater than twice (2x) the value computed for structures whose periods, T, are less than 0.125 sec.

K is 1.33 for shear wall structures; 1 for concrete frame structures of any type as well as combination structures and 0.8 for steel momement frame structures.

S = 1 unless actual soil dynamic test data are made available by the owner (see Section 16).

In lieu of using the value of S = 1, the structure owner, at his own expense, may perform a soil dynamics investigation of the actual site of his structure. The S value thus determined, upon validation of same by the building official, may be used. In no case, however, can S be less than 0.8.

I is presented in Table 7-2

W is the weight of the structure

(2) Lateral force on parts or portions of buildings or other structures shall be computed by the following formula:

$$F_p = Z\ C_p\ C/C_{60}\ IW_p$$

where: C and I are given in Tables 7-1 and 7-2, respectively
C_{60} is the value of C at 60 years
C_p is given in Table 7-3
W_p - weight of the part or portion of the structure

(3) Importance factor is defined by Table 7-4.

(4) For all structures having a period, T, greater than 0.6 seconds; being seven or more stories in height; or having an Importance Factor 1 or 2, structural dynamic analyses of the design shall be made to verify the shear, drift and moment carrying resistance capacity of the structure.

Table 7-1. Values for "C" (The City/State/Country is to select this or a modified table dependent upon Policy Risk Level Selected)

LIFE - L (YEARS)	C
2	.0073
5	.012
10	.018
20	.027
40	.040
60 NEW	.050

Policy Risk Level

Approximate Social Risk	Approximate Economic Risk
1 death per 1,000,000 persons exposed per year	1 dollar lost per $10,000 exposed per year

To increase or decrease the Policy Risk Level by a factor of 10 multiply or divide the values of C by 1.28

Table 7-2. Importance Factor, I

STRUCTURE IMPORTANCE*	I
2	1.64
3	1.28
4	1.00
5	0.78

*Importance factor 1 requires special study.

Table 7-3. Horizontal Force Factor "C" for Parts or Portions of Buildings or Other Structures

PART OR PORTION OF BUILDINGS	DIRECTION OF FORCE	VALUE OF C_p
EXTERIOR BEARING AND NONBEARING WALLS, INTERIOR BEARING WALLS AND PARTITIONS, INTERIOR NONBEARING WALLS AND PARTITIONS OVER 10 FEET IN HEIGHT, MASONRY OR CONCRETE FENCES OVER 6 FEET IN HEIGHT.	NORMAL TO FLAT SURFACE	0.20
CANTILEVER PARAPET AND OTHER CANTILEVER WALLS, EXCEPT RETAINING WALLS	NORMAL TO FLAT SURFACE	1.00
EXTERIOR AND INTERIOR ORNAMENTATIONS AND APPENDAGES	ANY DIRECTION	1.00
WHEN CONNECTED TO, PART OF, OR HOUSED WITHIN A BUILDING: TOWERS, TANKS, TOWERS AND TANKS PLUS CONTENTS, STORAGE RACKS OVER 6 FEET IN HEIGHT PLUS CONTENTS, CHIMNEYS, SMOKESTACKS AND PENTHOUSES	ANY DIRECTION	0.20[1,2]
WHEN RESTING ON THE GROUND, TANK PLUS EFFECTIVE MASS OF ITS CONTENTS	ANY DIRECTION	0.10
SUSPENDED CEILING FRAMING SYSTEMS[3] (APPLIES TO SEISMIC ZONES NOS. 2, 3, AND 4 ONLY)	ANY HORIZONTAL DIRECTION	0.20
FLOORS AND ROOFS ACTING AS DIAPHRAGM[4]	ANY DIRECTION	0.10
CONNECTIONS FOR EXTERIOR PANELS	ANY DIRECTION	2.00
CONNECTIONS FOR PREFABRICATED STRUCTURAL ELEMENTS OTHER THAN WALLS, WITH FORCE APPLIED AT CENTER OF ASSEMBLY[5]	ANY HORIZONTAL DIRECTION	0.30

[1]When located in the upper portion of any building where the "h_n/D" ratio is five-to-one or greater the value shall be increased by 50 percent.

[2]"W_p" for storage racks shall be the weight of the racks plus contents. The value of "C_p" for racks over two storage support levels in height shall be .16 for the levels below the top two levels.

[3]For purposes of determining the lateral force, a minimum ceiling weight of 5 pounds per square foot shall be used.

[4]Floors and roofs acting as diaphragms shall be designed for a minimum value of "C_p" of 10 percent applied to loads tributary from that story unless a greater value of "C_p" is required by the basic seismic formula for V.

[5]The "W_p" shall be equal to the total load plus 25 percent of the floor live load in storage and warehouse occupancies.

Table 7-4. Values for Importance Factor* (Level of Acceptable Risk)

IMPORTANCE FACTOR	MAXUMUM HUMAN OCCUPANCY (PERSONS)
5	1 - 9
4	10 - 00
3	100 - 999
2	1000 AND OVER*

*See Table C-28 of report text for further definition.

(5) For all structures having an Importance Factor 1 or 2; having a period, T, greater than 0.1 second; or being seven or more stories in height; soil dynamics investigations shall be conducted in order to derive an appropriate value for S. (These analyses shall be based on data derived under criteria outlined in Section 16.)

(6) An existing structure shall not be considered as presenting an unacceptable earthquake hazard provided V_a (as computed by the owner's engineers on the basis of the structure's actual condition and/or proposed repairs or strengthening measures) equals or exceeds V, (as computed using the appropriate structure lifetime and important factor). The accuracy of any facts upon which V_a or the tabular values selected in computing V are based, shall be substantiated to the satisfaction of the building official or, failing that, to the satisfaction of the Board of Appeals (see Section 19).

(7) In using Table 7-1 and in relating structure lifetime to a date certain by which the demolition or strengthening of a structure must be completed, the parameter, L., representing the lifetime of the structure before its voluntary demolition or further strengthening, shall be considered as beginning at the effective date of adoption of this code.

(8) All new structures shall be designed for a minimum life of sixty (60) years.

(9) All owners of structures selected by the City/State/County of __________ shall provide accessible space for the installation of appropriate earthquake recording instruments. Location of said instruments shall be determined by the ________. The __________ shall make arrangements to provide, maintain and service the instruments. Data shall be the property of the ____________, but copies of the individual records shall be made available to the public upon request and the payment of an appropriate fee. All Importance Factor 1 and 2 structures and new structures seven stories and greater shall contain a minimum of three (3) strong motion earthquake accelerographs to be located within the structure at the direction of the __________. Maintenance and service of the instruments shall be provided by the owner of the building subject to the approval of the building official. Data produced

by the instruments shall be made available to the building official upon his request.

(10) All site investigations and earthquake criteria reports shall be in accordance with the provisions of Sections 15 through 18.

Section 8 - Notice of Inspection:

The building official shall send via certified mail a written Notice of Intent to Inspect together with a blank inspection form to each of the owners of the structure, as the name be shown on the Tax Assessor's rolls. Said notice shall be in substantially the following form:

NOTICE OF INTENT TO INSPECT

Please take notice that a representative of the Department of Building of the ______________________ shall inspect that certain structure located at ____ to determine its potential susceptibility to major damage in the event of an earthquake. Said inspection will commence at the site of said structure at _______ a.m. on _______________, _______________. Please arrange to have someone availabe at the site of said structure at the indicated time who has the authority and means to admit said representative to the structure in order that interior characteristics thereof may be examined. Your failure to provide access to the structure for the representative shall cause said buildings to receive a maximum risk rating in those categories for which the lack of such access prevents inspection. Following completion of said inspection, a copy of the representative's report shall be mailed to you at the address to which this Notice has been mailed. A blank form of the type to be used by the representative in determining the earthquake risk of the aforesaid structure is enclosed with this Notice for your information.

Said Notice shall be mailed at least ten (10) calendar days prior to the date of the proposed inspection.

Section 9 - Notice of Excessive Risk:

Within a reasonable time following the completion of the inspection of a structure, as provided in Section 4, the building official shall compile the data obtained and develop the aggregate grade for the structure inspected. Should a structure be accorded a grade of _____ or more points, the building official shall immediately send via certified mail Notice of Excessive Risk to those persons to whom the Notice of Intent to Inspect was sent, unless prior to this time the building official shall have received a written authorization executed by all of such persons requesting that further correspondence be sent to a particular person only. If such authorization be timely received the building official shall send the Notice of Excessive Risk via certified mail to the person designated in said authorization. The Notice of Excessive Risk shall be in substantially the following form:

PLEASE TAKE NOTICE that an inspection of your structure located at ____ conducted on __________________, 19___, indicated that said structure carries in excessive risk of major damage in the event of earthquake which would endanger the safety of persons and property located in, on, or about said structure at the time of such event. Within sixty (60) days from the date of this notice, you shall present to this office a plan of action for reducing the earthquake risk associated with said structure to an acceptable level. In the event your proposed plan of action does not indicate abandonment and demolition of the structure within one hundred twenty (120) calendar days from the date of this notice, you shall submit to this office information as to the lateral force withstanding capacity of the structure in its present condition, proposed repairs or strengthening measures which you believe will increase the lateral force withstanding capacity of the structure to a level commensurate with the acceptable level of earthquake risk for your prospective occupancy and your projected life for the structure, any proposals you may wish to make for altering the occupancy or prospective life of the structure to reduce the earthquake risk to an acceptable level commensurate with the actual or projected lateral force withstanding capability of the structure, and any other information you believe affects the potential risk of death or serious bodily injury to persons in or near your structure as a result of the occurrence of earthquake forces in the area of your structure. Information as to the magnitude of the lateral force withstanding capability associated with your structure in its present condition, as well as information as to proposed repairs or strengthening measures intended to increase the lateral force withstanding capability, shall be prepared by a structural or civil engineer licensed under the laws of ______________________________ to practice said profession. An extension of the aforesaid sixty (60) day period may be obtained, for good cause shown, by requesting same in writing filed with this office at least seven (7) calendar days prior to the expiration of said sixty (60) day period. Such request shall be accompanied by a written statement of your contemplated action, the accomplishments toward same up to the time of the request, an estimate of the time required to complete the formulation of your proposed plan of action, and the name and address of the engineer, if any, whom you may have engaged.

A copy of the code, by authority of which this notice is sent, may be obtained from ______________________________ upon payment of an appropriate fee.

Section 10 - Structure Owner's Options:

In the event of structure's acceptable lateral force carrying capacity, V, is higher than V_a defined in Section 7 of this code, the structure's owner shall:

(1) Abandon and demolish the building; or

(2) Carry out such repairs or strengthening measures as will raise the level of V_a which the building can withstand to a 60 year life level of V; or

(3) Reduce the projected lifetime to demolition of the structure to the level at which V equals or exceeds V_a; if the owner elects to reduce the projected lifetime to demolition, he shall execute with ____________ an Agreement to Abandon and Demolish, which agreement shall recite a date certain for the completion of demolition and which shall be recorded under the owner's name in the manner of a deed, so as to provide constructive notice to potential vendees that the structure is to be demolished by the date recited; or

(4) Reduce the maximum number of persons exposed per year to death or injury in the event the structure suffered major structural failure during an earthquake, thus producing an acceptable level of

V. This may be accomplished by a formal change in the occupancy rating for the structure; or by owner imposed reductions in the actual occupancy such as would be achieved by closing of the upper floors of a building, or by reducing the seating capacity of a theater, or by reducing the hours of business of a business establishment, etc.; or by any reasonable lawful technique which results in reducing the number of exposed persons; or

(5) Accomplish some combination of (2), (3) and (4) above, which has the aggregate effect of producing an acceptable level of V.

Section 11 - Intermediate Risk Graded Structures:

After substantially all of the structures receiving a prima facie hazard rating of _____ points or more under Section 4 have been the subject of completed administrative action under this code, whether the same have been accomplished on an informal level by conferences between the building official and the structure's owner(s) or on a formal level by hearing and decision, and order, of the Board of Appeals, the building official shall send notices as in Section 4 to the owners of those structures which were assigned a hazard grading under Section 4 of between ____ and _____ points. Administrative action with respect to structures in this group shall proceed in the same manner, and the owners of such structures shall have the same rights and responsibilities, as is hereinbefore provided with respect to the structures which received hazard grading scores of more than _____ points pursuant to Section 4.

Section 12 - Application for Order of Abatement of Nuisance:

In the event the owner of a structure is notified pursuant to Section 9 of this ordinance, and a plan of action satisfactory to the building official is not presented within sixty (60) days after said notice shall have been mailed or within such period of extension of time as may have been granted in writing by the building official, then the building official shall apply in writing to the Board of Appeals for an order declaring the structure to be a nuisance and ordering it to be demolished by a date certain. Said written application shall set forth in the form of factual allegations all facts which, if proven, are necessary to justify an order of condemnation, including but not limited to the following:

(1) The location and legal description of the structure;

(2) Its prima facie hazard rating score with a true copy of the inspection form to be attached to the application;

(3) The structure's present maximum occupancy;

(4) The date upon which the owner of the structure was notified pursuant to Section 9 of this ordinance;

(5) A statement as to whether the structure owner has submitted a plan of action pursuant to Section 9 of this ordinance;

(6) The date certain by which the structure must be demolished, in the building official's opinion, in order to keep the earthquake risk associated with it at or below the applicable acceptable Policy Risk level.

A copy of said written application shall be mailed by certified mail to the persons to whom the notice of Section 8 of this ordinance was mailed.

Section 13 - Hearings on Applications:

Upon receiving a written application pursuant to Section 8 of this ordinance, the Board of Appeals shall set a date and time for a hearing.

Section 14 - Enforcement of the Board Order:

(1) In the event the Board orders a structure demolished immediately upon the effective date of its order, the structure's owner, unless such order is stayed by a court of competent jurisdiction, shall arrange for the vacation and demolition of said structure within sixty (60) days after the Board's order becomes effective. Should the structure owner fail to inform the building official within five (5) days after the Board's order becomes effective that such arrangements have been made or should the owner's scheduled demolition not in fact be completed within the aforesaid sixty-day period, then the building official may arrange for the demolition of the subject structure and impose a lien upon the property for the costs of same.

(2) In the event the Board orders the demolition of the subject structure by a date certain which is three (3) months or more after the effective date of the order, and said order is not stayed by a court of competent jurisdiction, the building official shall prepare a Notice of Pending Order of Demolition and arrange for the recordation of same in the office of the appropriate ____________ recorder. Said notice shall be in substantially the following form:

TO WHOM IT MAY CONCERN:

NOTICE IS HEREBY GIVEN THAT BY order of the Board of Appeals of ________________________ dated _______________, 19___, that certain structure now standing at ____________________________

____

and described generally as ____________________________________

____

must and shall be demolished on or before ___________________, 19____. A certified copy of said order may be obtained from the Department of Building of ___________________________, upon the payment of an appropriate fee. If said structure is not demolished in accordance with the aforesaid order, the same shall be demolished by _______________________________ and the costs therefore assessed as a lien upon the land upon which the structure stood. A lien in the amount of $___________ in favor of the ______________________________ is hereby assessed against said property for the costs of recording this notice.

Said notice shall be recorded under the names of each and every person to whom the notice of Section 8 was mailed. The structure's owner may pay the recording fees for the aforesaid notice and thereby avoid the imposition of a lien for same against the property.

(3) In the event the Board certifies to the validity of any or all of any measures the owner shall have proposed as a means of reducing the earthquake risk, and finds that accomplishment of such measures will reduce the earthquake risk associated with the structure to or below the applicable acceptable Policy Risk level, it shall order the owner to immediately initiate the accomplishment

of such measures and to complete the same within a reasonable time not, however, to exceed ten (10) percent of the projected lifetime to demolition of the structure. The Board shall designate in its order, based on evidence presented to it during the hearing, that date certain which represents a reasonable time in the Board's opinion for the accomplishment of the proposed measures.

(4) The Board shall retain jurisdiction over cases in which it has approved owner-proposed measures for reducing earthquake risk until such measures shall have been timely accomplished. In the event written evidence of the completion of the approved measures is not presented to the Board within five (5) days after the designated date for the completion of such measures shall have passed, the Board shall revise its decision and order to require the immediate vacation and demolition of the structure unless prior to said date the structure's owner shall have applied for an extension of said time. Any application for such an extension shall be in writing setting forth what has actually been accomplished, what remains to be done, and the reasons for the requested extension. Should the Board conclude that good cause has been shown for an extension it may grant such an extension in writing for a period not to exceed fifty percent (50%) of the original period for the first extension, and if an extension has already been granted, for a period not to exceed fifty percent (50%) of the immediately preceding extension.

(5) In the event the building official or any interested, competent person presents written affidavits to the Board indicating the owner is not proceeding in good faith to timely accomplish any measures approved by the Board in its original decision and order, the Board shall on five (5) day's written notice mailed via certified mail to the owner of the structure, schedule and conduct a hearing on the matter. At such hearing, evidence, oral and written, may be presented as in the original hearing, and if the Board is convinced that the owner is not proceeding in good faith to timely carry out the Board's original order, then the Board shall revoke said order and order instead the immediate vacation and demolition of the structure. Written affidavits shall not, however, be received by the Board under this section until at least fifty percent (50%) of the time allowed in the Board's original order shall have expired.

Section 15 - Engineering Geology and Soils Engineering Report Requirements:

Engineering geology and/or soils engineering studies shall be required for any new structure (Importance Factor 1, 2, or 3) and/or development whenever any of the following conditions may exist.

(1) Soils Engineering Reports are required for the site when:

(a) the depth (or heights) of cut or fill at the site is 3 feet or greater.

(b) the fill is to support structural footings.

(c) an engineered cut or fill is required.

(d) the soils are or may be subject to shrinkage and swelling.

(2) Engineering Geology Reports are required for the site when:

(a) finish cut or fill slope faces with vertical heights in excess of 10 feet.

(b) existing slope steeper than 5 horizontal to 1 vertical.

(c) an existing cut slope having a vertical height in excess of 10 feet.

(d) existing sea cliffs, stream bank cliffs, etc. in excess of 10 feet.

(e) existing or suspected earthquake or seismic hazards.

(f) existing or suspected groundwater hazards.

(g) areas are underlain by landslides or soil creep or by rock material susceptible to landslide or creep activity.

(h) areas where alluvial material, slope-wash, rock materials exist that are subject to settlement, subsidence, or hydrocompaction.

(i) areas subject to drifting or loose sand.

Section 16 - Seismic Site Evaluation Requirements for Buildings, Transportation Networks and Public/Community Structures:

When an importance factor 1, 2 or 3 structure (new or existing) is located with a high risk area from any one of the potential ground breakage mechanisms (slope failure, fault break, ground settlement, and liquefaction) as identified on maps prepared to describe these phenomena, an investigation outlined below shall be conducted by the owner's geotechnical consultant. The geotechnical consultant shall be a coordinated team experienced in the fields of soil and foundation engineering, engineering geology, geohysics and earthquake engineering, each licensed in his respective field of expertise, and approved by the building official. Signatures and license numbers are required.

(1) The scope, procedures, responsibilites, and schedules for a Seismic Site Evaluation shall be developed with the geotechnical consultant, owner, architect, and structural engineer prior to the performance of Seismic Site Evaluation options outlined below.

(2) A reconnaissance of the site and a review of published data shall be made by the geotechnical consultant, in the manner outlined in Section 17, to determine if there is the likelihood of one of the following hazards being present.

a. Areas subject to possible ground surface rupture due to faulting shall be evaluated. No new structure shall be located on an active fault (see Section 18).

b. In areas subject to possible ground settlement, liquefaction or slope failure site specific investigations to determine the potential for and possible effect on the proposed or existing structure shall be determined.

c. If the studies determine that any of the above risks equal or exceed the shaking risk, their effects on the proposed development shall be determined prior to construction or development.

The report shall be submitted to the building official and to the owner. It should contain the geotechnical consultant's best judgment in the form of a numerical risk probability relative to shaking risk of any of these hazards occurring during the lifetime of the structure. If any appear to exceed the shaking risk during the lifetime of the new or existing structure, the geotechnical consultant shall submit a proposal to the owner for additional field work that may be necessary to more accurately determine the relative order of these risks. The building official, architect, and design engineer should assess the information and make recommendations to the owner for further work, if required.

(3) If further work is required by the building official, a site specific definition of the ground shaking and building response shall be provided by the geotechnical and engineering consultant as input to design. Existing structure owners can either use the procedure outlined in Section 7 or the following procedure. If the procedure below is employed and V values so computed are higher than those in Section 7, then the new values must be used. However, if these values are lower than those in Section 7, then they also may be used. Recommendations discussing the seismic factors in Section 7 shall be developed. The steps involved in the analysis shall include:

1. Determination of the location of active faults (Section 18) which may affect the site and the definition of potential earthquake magnitudes and epicentral locations for those faults or other source areas.

2. Evaluation of the statistical seismicity of the site in terms of earthquake magnitude, acceleration, velocity, and displacement.

3. Reduction of potential source motions due to geometrical divergence, attenuation and dispersion.

4. Evaluation of the dynamic characteristics of the site with respect to the amplification or attenuation of bedrock or basement rock motions.

5. Determination of the motions meeting the seismic conditions in Section 7 and preparation of a report of findings. The selection of the design earthquakes and ground motions shall reflect the Policy Risk Level and should be made in consultation with the owner, architect and/or design engineer and approved by the building official prior to use in design or rehabilitation. The factors to be considered in the selection shall include:

   a) The probability of earthquake intensity occurrence during the useful life of the structure,

   b) Earthquake magnitudes and locations that appear to be warranted (stated criteria shall be provided),

   c) The cost of construction,

d) The importance of the structure in terms of service to the public and consequences of failure, and

e) The maximum occupancy of the structure.

a. The result of the foregoing analyses shall be presented in one or more of the formats selected by the design engineer and shall be approved by the building official prior to detail design.

1. Constant Ground Acceleration. Design ground acceleration for the site to be used by the structural engineer to design extremely rigid ($T < 0.1$ sec.) or one and two-story structures where the response of this structure because of its rigidity may be assumed to be identical to that of the site.

2. Constant Structural Response Acceleration. Structural acceleration anticipated based on an evaluation of the response of single degree of freedom elastic systems (period range of 0.1 sec. to 0.6 sec.) to the anticipated ground motion for the site. This acceleration should be developed for a level of damping consistent with the type of structure proposed, and should be used in designing structures falling into this category (i.e., more flexible one-story structure to stiffer five-story structures) utilizing special techniques.

3. Elastic Structural Response Spectra. A smoothed, damped response spectrum presenting the response of a single-degree-of-freedom elastic system (period range from 0.1 sec. to 6.0 sec.) to site ground motion is presented, usually on a triparite plot. This response spectrum should be used by the structural engineer in designing more flexible structures in which more than one mode of vibration participates utilizing modal response techniques.

4. Time-History Plot of Probable Ground Motion at the Site. Ground displacement or acceleration at the site is presented as a function of time.

5. Other Data - Standard Seismicity. Previously developed earthquake data deemed appropriate for the site and the proposed structure may be used. This can be presented in the form of any one of the above four formats. Concurrence with the design engineer should be obtained on any limitations of frequencies to be considered.

Section 17 - Guidelines to Geologic and Seismic Reports:

(1) The reconnaissance of the site shall consist of:

a. Regional Review:

A review of the seismic or earthquake history of the regional area should establish the relationship of the site to known faults and epicenters. This would be based primarily on review of existing maps and technical literature and would include:

1. Major earthquakes during historic time and epicenter locations and magnitudes near the site. (Analysis should be made of all site damage sustained due to shaking, or surface faulting in historic times.)

2. Location of any major or regional fault traces affecting the site being investigated, and a discussion of the tectonic mechanics and other relationships of significance to the proposed construction.

3. Evidence of regional fault strain and creep that may influence site design.

4. Examination of time-sequenced aerial photographs to determine the topographic effects of seismic or fault activity at the study location.

5. Review of local ground water data such as water level fluctuation, ground water barriers or other ground water anomalies indicating possible faults.

b. Site Investigation:

A review of the geologic conditions at or near the site that might indicate recent fault or seismic activity. The degree of detail of the study should be compatible with the type of development (priority classification) and geological complexity. The investigation should include the following:

1. Location and chronology of local faults and the amount and type of displacement estimated from historic records and stratigraphic relationships. Features normally related to fault activity such as sag ponds, offset bedding, disrupted drainage systems, alignment of springs, offset ridges, faceted spurs, dissected alluvial fans, scarps, alignment of landslides, and vegetation patterns, to name a few, should be shown on the geologic map and discussed in the report.

2. Locations and chronology of other earthquake induced features caused by ground settlement, liquefaction, etc. Evidence of these features should be accompanied by the following:

   a) Map showing location relative to proposed construction.

   b) Description of the features as to length, width and depth of disturbed zone.

   c) Estimation of the amount of disturbance relative to bedrock and surficial materials.

3. Distribution, depth, thickness and nature of the various unconsolidated earth materials including ground water, which may affect the seismic response and damage potential at the site must be discussed in detail in the report.

(2) If further work is required after completion (2) in Section 16 the detailed evaluation may consist of:

a. Trenching across any known active faults and suspicious zones to determine location and recency of movement, width of disturbance, physical condition of fault zone materials, type of displacement, and geometry.

b. Exploratory Borings to determine depth of unconsolidated materials, ground water and saturation on unconsolidated sediments, and to verify fault plane geometry. In conjunction with the soil engineering studies, obtain samples of soils and bedrock materials for laboratory testing.

c. Geophysical Surveys which may indicate types of materials and their physical properties, ground water conditions, and fault displacements.

d. Other Design Data obtained by the subsurface investigation significant to the engineer in evaluating the design ground motion spectrum.

(3) Conclusions and Recommendations

At the completion of the data accumulating phase of the study, all of the pertinent information is utilized in forming conclusions of potential risk relative to the intended land use or development. Many of the conclusions will be revealed in conjunction with the soil engineering study. This portion of the report must include, but need not be limited to the following:

a. Surface Rupture Along Fault

1. Age, type of surface displacement, and amount of reasonably anticipated future displacements and magnitudes.

2. Definition of any area of high risk (stating the numerical risk probability during structure lifetime).

3. Recommend building restrictions or use-limitations within any designated high risk area.

b. Secondary Ground Effects

1. Ground settlement and shallow ground rupture.
2. Liquefaction of sediments and soils.
3. Potential for earthquake induced landslides.

c. Probable (numerical values must be stated) acceleration which may be assigned to various foundation materials, depend upon the reaction or interaction of the following:

1. Firm bedrock (consolidated).
2. Soft sediments (unconsolidated).
3. Soft and saturated sediments.
4. Artificial fill (existing or proposed).

d. Topographic and proposed grading features which may be affected by anticipated future seismic activity:

1. Thick or deep fill prisms.

2. Effects at fill-bedrock daylight lines relative to foundation conditions.

3. Stability of high cut and natural slopes.

(4) Presentation of Data

Visual aids are desirable in depicting the data and may include:

a. General data

1. Geologic map of regional and/or local faults.
2. Map(s) of earthquake epicenters.
3. Fault strain and/or creep map.

b. Local or site data

1. Geologic map.

2. Geologic across-sections illustrating displacement and/or rupture.

3. Local fault pattern and mechanics relative to existing and proposed ground surface.

4. Geophysical survey data.

5. Logs of exploratory trenches and borings.

(5) Other Essential Data

a. Sources of data

1. Reference material listed in bibliography.
2. Maps and other source data referenced.
3. Compiled data, maps, plates included or referenced.

b. Vital support data

1. Maximum credible earthquake.
2. Maximum probable earthquake.
3. Maximum expected bedrock acceleration.

c. Signature and license number of geologist registered in California.

Section 18 - Definitions of Active, Potentially Active, and Inactive Faults:

A fault is defined as: "A fracture or fracture zone along which there has been displacement of the two sides relative to one another parallel to the fracture. The displacement may be a few inches or many miles." (American Geological Institute, 1957.) With regard to seismic activity, faults can be divided into three major classifications: (A) active faults; (B) potentially active faults; and (C) inactive faults. This grouping is based on the following criteria, listed in order of decreasing hazard:

A. Active faults: These faults are those which have shown historical activity.

B. Potentially active faults: These faults are those, based on available data, along which no known historical ground surface ruptures or earthquakes have occurred. These faults, however, show strong indications of geologically recent activity.

Potentially active faults can be placed in two subgroups that are based on the boldness or sharpness of their topographic features which provide estimates related to recency of activity. These subgroups are as follows.

1. Subgroup 1 - High Potential

   a. Offsets affecting the Holocene deposits (age less than 10,000 - 11,000 years).

   b. A groundwater barrier or anomaly occurring along the fault within the Holocene deposits.

   c. Earthquake epicenters (generally from small earthquakes occurring close to the fault).

   d. Strong geomorphic expression of fault origin features (e.g., faceted spurs, offset ridges or stream valleys or similar features, especially where Holocene topography appears to have been modified).

2. Subgroup 2 - Low Potential

   This subgroup is the same as above, with the exception that the indications of fault movement can be only determined in Pleistocene deposits (between 11,000 ± and 2.5 million years old).

C. Inactive faults: These faults are without recognized Holocene or Pleistocene offset or activity.

Section 19 - Seismic Design and Planning Advisory and Appeal Board:

(1) A Seismic Design and Planning Advisory and Appeal Board shall be established to hear appeals against the Building Departments' rulings.

(2) The Seismic Design and Planning Advisory and Appeal Board shall be composed of five persons appointed by the ______________. These five persons shall consist of a planner, two structural/civil engineers, an engineering geologist, and an engineering seismologist. All shall have experience in land use planning and seismic analysis.

(3) Each member of the Seismic Design and Planning Advisory and Appeal Board shall be paid $__________ per meeting attended but not exceeding $__________ in any one month.

(4) Each member of the Seismic Design and Planning Advisory and Appeal Board shall serve at the pleasure of ______________. Each member of the Seismic Design and Planning Advisory and Appeal Board shall serve for a period of one year unless sooner relieved. A member of the Seismic Design and Planning Advisory and Appeal Board may be relieved at any time by a majority vote of the ______________.

A member of the Seismic Design and Planning Advisory and Appeal Board may be reappointed by ________________.

(5) The Seismic Design and Planning Advisory and Appeal Board shall serve in an advisory capacity and shall provide technical guidance to the ________________________.

(6) The Seismic Design and Planning Advisory and Appeal Board shall consider all appeals and protests which relate to land use designation, requirements for professional analyses, design and rehabilitation criteria as determined from seismic considerations.

(7) The Seismic Design and Planning Advisory and Appeals Board shall submit in writing to ________________________ its findings and recommendations on all matters considered by it.

(8) Each member of the Seismic Design and Planning Advisory and Appeal Board shall have power to administer oaths and to issue subpoenas; failure to appear or testify in response to any subpoena or to produce an item under subpoena "duces tecum" shall be punished as a misdemeanor; the _______________ shall cause such subpoenas to issue under the seal of the ____________, and the Chief of Police shall cause such subpoenas to be served.

The __________ Attorney, or an assistant or deputy designated by him, shall appear at the request of the Seismic Design and Planning Advisory and Appeal Board at any hearing before that Board.

(9) The Seismic Design and Planning Advisory and Appeal Board shall operate in accordance with the rules set forth in this paragraph.

(9-a) Meetings

The regular meeting time of the Board is 2 p.m. on the _________ of each month. The regular meeting place is ________________ of __________________. The Board may revise the time and place of its meetings by filing notice of such change with ___________________.

Regular meetings will be held only when all members of the Board are notified in advance by the Executive Secretary.

Special meetings may be called by the Chairman.

Meetings may be adjourned by the Board should the Board be unable to complete its business during the regular or special meeting, or fail to have a quorum, or find it necessary to delay completion action on an appeal to obtain additional evidence.

(9-b) Attendance and Quorum

A quorum of the Board for the transaction of business shall be three members and the Executive Secretary.

Should any member of the Board be involved in an action before the Board, he shall not sit on the Board during any meeting in which such action is involved. Such member shall not be present at the meeting during the time the appeal in which he is involved is being heard.

Meeting of the Board will be open to the public.

(9-c) Voting

The Chairman may vote.

All actions of the Board shall be decided by a majority vote of those members.

A tie vote on an appeal shall be a vote to deny the appeal and to sustain the decision of the Building Department.

(9-d) Records

A written record of the actions of the Board shall be maintained by the Executive Secretary.

(9-e) Policy Changes

The Board may recommend to the Planning Commission or to the Board of Building and Safety that policies supporting the code be changed but shall not through their decisions attempt to change such policies.

(9-f) Filing Appeals

Appeals shall be filed at least twelve days prior to the regular meeting day of the Board.

All appeals submitted to the Board shall be made in writing together with all supporting evidence.

(9-g) Processing Appeals

The ________ shall file a written statement supporting its position on the appeal together with supporting evidence.

Copies of the appeal and the __________ statement will be made available to all members of the Board prior to the date set for the hearing.

(9-h) Hearing Appeals

The applicant and/or his technical representatives shall be present at the Board Hearing to give oral testimony and to answer questions of the Board.

The Building Department technical representative shall be present at the Board Hearing to give oral testimony and to answer questions of the Board.

The Building Department shall present the evidence supporting its action first, followed by the appellant's presentation of evidence supporting his appeal. The Board may then question both parties before closing the evidence portion of the hearing. The Board may also adjourn to allow the gathering of additional evidence before closing the evidence portion of the hearing. The Board may also adjourn to allow the gathering of additional evidence before closing the hearing.

(9-i) Decision on Appeals

The decision on any appeal shall be furnished in writing to the appellant and to the Building Department within seven working days after the regular meeting date of the Board unless a delay is agreed to by both the appellant and the Building Department.

The reasons for derial of an appeal shall be made known to the appellant in writing if he so requests.

(10) When a matter of appeal is heard by the Seismic Design and Planning Advisory and Appeal Board, the appellant in said matter shall pay a referral fee of $________ and shall also pay a fee as follows:

a. Where no more than two lots are $________ in value of construction are involved in the appeal, $________.

b. Where not less than three nor more than 10 lots, or between $________ and $________ in value of construction are involved in the appeal, $________.

c. Where more than 10 lots or $________ in value of construction are involved in the appeal, $________.

8. CONCLUSIONS

8.1 Existing Building Stock

Past destructive earthquakes and also risk analyses have shown that earthquake risks are highly dependent on the vulnerability of artifacts. The existing building stock plays a major role in determining overall earthquake risks. In order to deal with the existing building stock, it is necessary first to define the expression "old buildings." In spite of differences that may exist between portions of the building stock constructed during different periods of the past, in principle old buildings may be regarded as all buildings not engineered according to codes currently in force. Even buildings engineered a few decades ago in accordance with their prevailing codes may now be regarded as highly hazardous in the light of current knowledge.

The concept of old buildings must, therefore, be dynamic. As codes are periodically updated, additional categories of buildings fall into the stock of old buildings and require appropriate concern. However, old buildings may also be divided into different classes (with respect to age, code in effect, etc.) that reflect significant differences in seismic resistance.

8.2 Post-Earthquake Versus Pre-Earthquake Activities

The concern for the existing building stock must be related both to post-earthquake and pre-earthquake activities. Post-earthquake activities, carried out under emergency conditions, must first reduce the immediate high risks generated by the post-disaster situation. Still, these post-earthquake activities must be carried out so that extremely important information is provided for the longer term activities that take place under subsequent, more relaxed conditions. Then, the concern for a systematic mitigation of earthquake risks must exist. Given the emergency conditions under which post-earthquake activities are carried out, appropriate preparedness measures are necessary and must be considered as a high priority by national bodies.

Systematic attempts to mitigate seismic risks, referred to as pre-earthquake activities, must rely on a synoptic strategy that comprehends all elements of the social fabric. It is necessary to analyze individual buildings in relation to severe but credible earthquake scenarios. It is also necessary to consider systematically those factors involved in a national decision on the intervention in existing buildings and to provide the basic data required for risk mitigation activities.

8.3 Factors to be Considered in the Decision on Intervention

The decision on intervention in existing buildings, intended to limit seismic risk to an acceptable level for the building stock as a whole, must first consider feasible alternatives with regard to the physical intervention as well as with regard to functionality/occupancy of buildings. Risks may be reduced through upgrading structures, reducing occupancy, or removing some parts of an existing building. When intervention is agreed on to be necessary, a deadline for action must be set.

The decision on intervention must be adopted by the owners and on the basis of guidelines or regulation endorsed by national bodies. The comparison of the actual state of a building with current code requirements (possibly introducing some amendments to the provisions of codes in force for the design of new buildings) may represent the rule of procedure for existing buildings. These activities can be carried out by engineers experienced in earthquake-resistant design. However, at a governmental level, it may be necessary to consider full cost-benefit analyses in order to set appropriate guidelines for existing individual buildings or to establish appropriate amendments of current code provisions for new buildings. Cost-benefit analyses shall follow UNDRO risk concepts (seismic hazard, vulnerability, specific risk, elements at risk, risk) in order to estimate and evaluate risks. These analyses should consider, on one hand, predicted losses, the costs of intervention, and maintenance and, on the other hand, the benefits from the use of buildings. A multi-criterial approach may be considered, estimating costs/losses and benefits for various components, expressed in terms of human lives, of money, of cultural values, etc. The outcome of cost-benefit analyses should be expressed at the same time in terms both of actions to be performed and of deadline/priority indices. The solutions and priority indices set for buildings of different categories should be correlated with the general plans of urban, regional, and social development.

8.4 Basic Data Required for the Decision on Intervention

Deciding to intervene in existing buildings requires basic information pertaining to several categories: seismic hazard, vulnerability of buildings, possible consequences of direct seismic effects (damage to buildings) in terms of losses of lives, of economic losses etc., and feasibility and implications of various intervention strategies. If the code approach is used, it is not necessary to construct all these categories of data, since some of them are covered by code provisions. In contrast, when cost-benefit analyses are carried out in an institutional framework, it is advisable to use directly basic data for the categories referred to. Gathering the basic data for these categories requires extensive research, in most cases interdisciplinary in nature, to be organized in each country according to long-term plans.

8.5 Actions to be Undertaken at a National Level

The national ability to mitigate earthquake risks depends heavily on the adoption of appropriate measures by governmental bodies. A systematic concern of these bodies is required in order to make existing possibilities significant and to mitigate risks in the most cost-effective way.

A first concern must be that of providing an appropriate preparedness for post-earthquake activities. This must include adoption of appropriate methodologies, regulations and plans of action, as well as training of teams to be involved in post-earthquake activities. National bodies must be continuously concerned with the development and updating of strategies aimed to mitigate seismic risks to the society as a whole and of the regulatory basis, with the education of various groups involved in earthquake protection, and with the improvement of basic knowledge in terms of concepts, methodologies and basic information.

For each country, the national framework should also be considered from social, economic and cultural perspectives.

8.6 Final Comments on the Manual

This manual represents a pioneering attempt to tackle the difficult and complex issues of the seismic risks related to the existing building stock. Its authors are conscious of its imperfections and of the need to work out, in the near future, an improved edition. However, the general problem raised by this manual, and the guidelines, methodologies and data provided in this framework, are useful. The proper use and application of the manual will have, even at this stage, definitely positive effects.

This manual should be used in conjunction with all other manuals prepared within the framework of this project. The manuals of Working Groups A (on cast-in-place reinforced concrete structures), B (on prefabricated reinforced concrete structures), C (on masonry structures) are intended for use in design of new structures and provide basic knowledge for design and verification. The manuals of Working Group E (on repair and strengthening) and F (on historical monuments) are more directly related to the problems of existing structures.

APPENDIX A

APPENDIX A

PRESENTATION OF CUMULATIVE DAMAGE AND OBSERVED VULNERABILITY IN THE MONTENEGRO EARTHQUAKE OF APRIL 15, 1979 IN YUGOSLAVIA

A.1 Introduction

The earthquake of April 15, 1979, which affected the Montenegro coastal area, is one of the most devastating ever to occur in Yugoslavia. This earthquake affected almost all of Montenegro, part of the Croatian coastal area, and a considerable portion of Northern Albania. This region is known to be very active seismically -- frequently experiencing strong earthquakes, some catastrophic, both locally and regionally. From a tectonic viewpoint, the Montenegro coastal area belongs to the marginal, outer Dinarides and their transition into the Adriatic basin. Neotectonic processes of different direction and intensity are evident in the region as shown by the Dinarides and in the subsidence of the Adriatic basin; these processes are responsible for movement of these first-order structures. Displacements occur along the longitudinal faults, parallel to the coastal belt, in a northwest - southwest direction. The area affected by the April 15, 1979 earthquake represents a contact zone between first-order fault systems. This zone separates the coastal part of the Dinarides from the Adriatic basin and the Pec - Skadar dislocation, which is the southern marginal fault of the Dinarides system.

Seismic activity in this zone started in March 1979 by series of earthquakes of magnitude M = 3-4, followed by series of the strong foreshocks of April 9, 1979 with a maximum magnitude M = 5.4. The main shock occurred on April 15, 1979 at 6:20 a.m. (GMT) with a magnitude M = 7.2. Many strong aftershocks followed, the strongest of magnitude M = 6.1 occurring on May 24, 1979, and with its epicenter close to Budva.

The main shock of April 15, 1979 caused catastrophic consequences to Montenegro coastal area, the area of Scadar and Lesh in SPR Albania, and brought considerable damage to many districts in the continental part of Montenegro, Croatian coast and SR Bosnia and Herzegovina. The earthquake was recorded on an approximate area of 50,000 km^2 in the territory of Yugoslavia by three-componental strong motion accelerographs located in Makarska to the west, in Sarajevo to the north, and in Skopje and Ohrid to the southeast. The earthquake of April 15, 1979 and the strongest aftershocks were recorded by all the strong-motion instruments (three-componental SMA-1 accelerographs and WM-1 seismoscopes) installed in the Montenegro epicentral zone and outside the epicentral zone. As a result, more than 350 high-quality earthquake records have been obtained, the largest number of earthquake records obtained in a single seismic zone. Special attention should be given to 100 earthquake records taken in the epicentral zone, for earthquakes of different magnitudes, at sites of different soil conditions and on bedrock. These records will be further used for investigations of the focal mechanism, seismic wave propagation, influence of soil topography and the local soil conditions on the ground motion modification during strong earthquakes, and dynamic response of structures as well as of other aspects of both fundamental and applicative research on the effects of strong motion earthquakes. The maximum acceleration values are 49.3% g in the area of Ulcinj, for the vertical component and 45.9% g in the area of Petrovac, for the horizontal E-W component. The maximum horizontal acceleration recorded during the strongest aftershock of May 24, 1979 is almost of the same order as that of the main shock recorded in Bar (30.4% g, E-W component), in Petrovac (34.6% g, E-W component) and in the area of Budva (27.7% g, E-W component).

The disastrous earthquake of April 15, 1979 and the following series of very strong aftershocks caused enormous damage to the Montenegro coastal area and to many other districts of SR Montenegro. The amount of damage is very high, owing mainly to the high concentration of material goods in a rather narrow area and the unfavorable foundation conditions of large number of structures. It has been estimated that in Montenegro damage amounts to about 7% of the 1979 gross national income of Yugoslavia. Of 40,000 building structures for which classification of damage and usability has been made in the territory of six coastal communes, Cetinje and Danilovgrad, 54% are either undamaged or experienced only slight damage (marked in green color), 24% suffered serious repairable damage (yellow), and 22% are seriously damaged beyond repair and should be demolished (red).

According to the data presented by the Albanian delegation during their visit to Titograd, damage was also very high in the territory of SPR Albania. About 14,000 public and residential buildings were damaged, 70% of which are in the area of Skadar and 20% of which are in the territory of Lesha. Compared to property losses, human losses are relatively small. According to the information provided by the Republic Committee of Public Health of SR Montenegro, 94 people were killed in Montenegro coastal area during this earthquake, while 1,172 people were slightly or seriously injured, most of whom were in the communes of Bar, Ulcinj and Kotor. In the territory of SPR Albania, 35 people were killed and 374 injured, most of whom were in the territory of Skadar.

For estimation of earthquake effects and usefulness of numerous acceleration records, the Institute of Earthquake Engineering and Engineering Seismology in Skopje accomplished preliminary investigations to correlate the distribution of damage in the territory of SR Montenegro and the recorded maximum ground accelerations. Some of these correlations are presented in this report. The earthquake effects have been analyzed for building structures only, according to the damage and usability classification made by the Republic Commission of the Executive Council of the Republic Assembly of Montenegro in cooperation with the District Commissions. A uniform methodology developed by the Institute of Earthquake Engineering and Engineering Seismology, University of "Kiril and Metodij", Skopje has been followed. For distribution analysis of maximum ground acceleration, only records of the main shock of April 15, 1979 with a magnitude $M = 7.2$ were used (Figure A-1). The comparison made between the distribution of damaged structures and the distribution of maximum ground accelerations shows that a relatively good correlation exists and that a rather uniform application was made of the classification methodology for damage and usability of structures (Figures A-2 and A-3).

A high percentage of damage resulted mainly from the vibrational effects of the earthquake. However, the amount of damage was also significantly influenced by the instability of soil surface layers, as evidenced by liquefaction, landslides, intensive settlements, and rock falls. These occurrences affected buildings having shallow foundations, especially in coastal districts. As a result of liquefaction and extensive ground settlements, many significant industrial and hotel buildings constructed according to seismic design principles failed because inadequate site investigations had been made and insufficient knowledge had existed of the dynamic behavior of unstable soils under earthquake conditions. Instability of soil surface layers is evident even for lower ground accelerations and it played dominant role in the damage level for coastal districts, especially those of Bar, Budva, Tivat, Kotor and Herceg Novi.

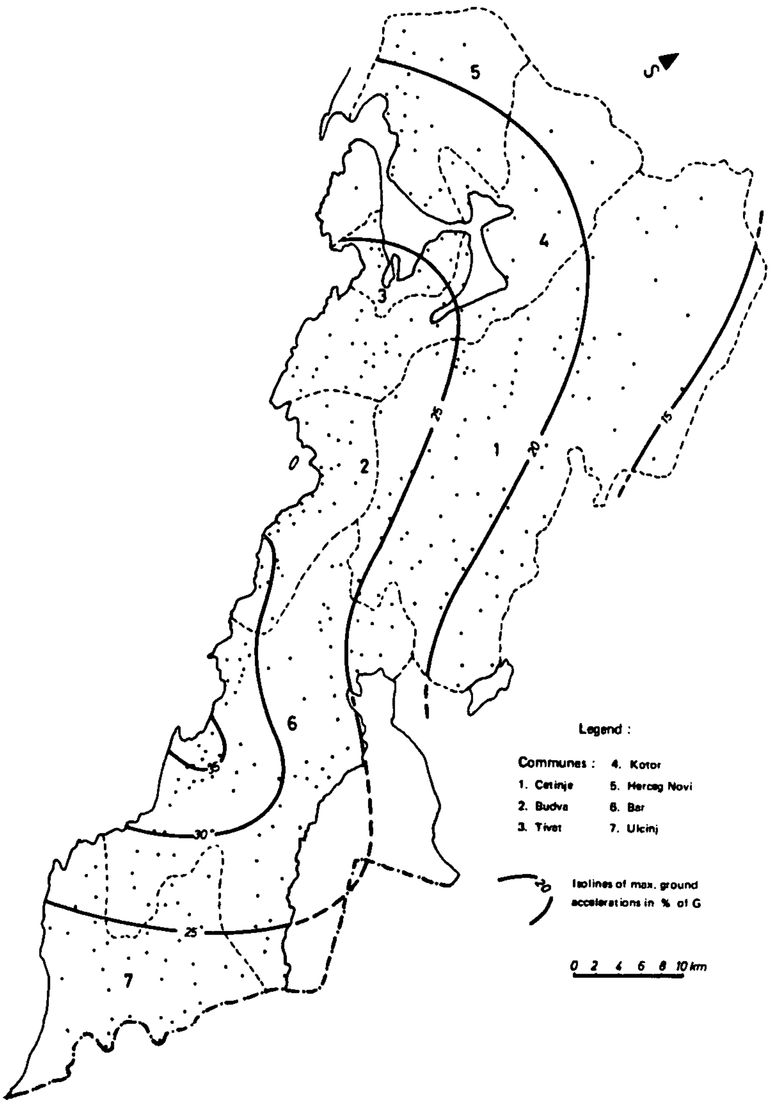


Figure A-1. Distribution of Maximum Ground Accelerations (W-E Component) During April 15, 1979 Earthquake (Source: Petrovski, J. et al., 1983)

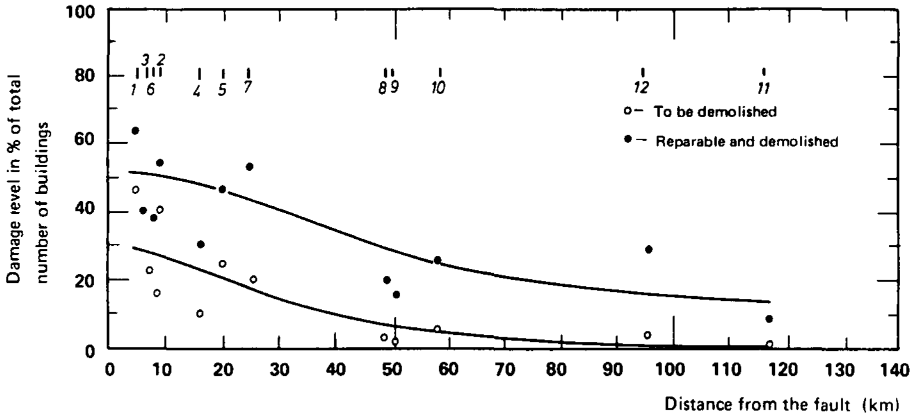


Figure A-2. Montenegro Earthquake 1979. Average Relationships of Damage Distribution Versus Fault Distance (Source: Petrovski, J. et al., 1981)

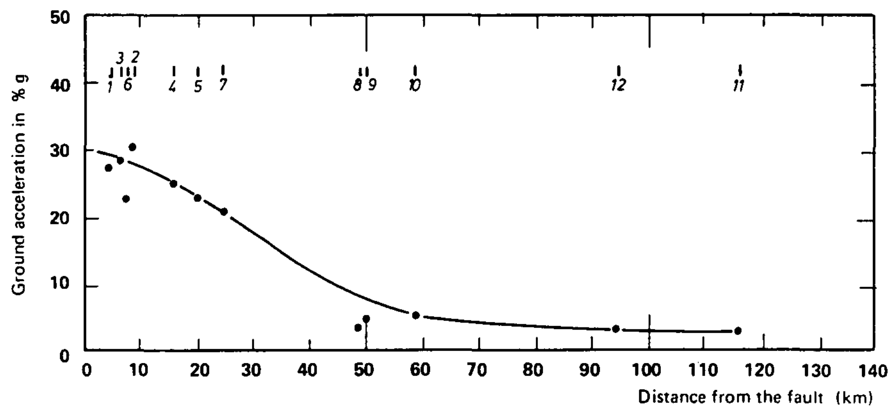


Figure A-3. Montenegro Earthquake 1979. Maximum Ground Acceleration Distribution Based on E-W Component Records. (Source: Petrovski, J. et al., 1981)

A.2 Classification of Damage and Usability Analysis of Building Structures

Classification of the damage and usability of structures in the territory of SR Montenegro was carried out based on the uniform methodology developed by the Institute of Earthquake Engineering and Engineering Seismology, Skopje, in cooperation with the Republic Institute of Town Planning and Design, Titograd. This methodology is based on the experience of other earthquakes in Yugoslavia, and is designed to meet the requirements of the Republic Commission for Classification of the Damage and Usability of Buildings affected by the April 15, 1979 earthquake. According to this methodology, buildings are classified in eight categories, the first three (marked in green color) being allocated to usable buildings with smaller nonstructural damage and negligible structural damage, the second two categories (marked in yellow) covering temporarily unusable buildings with seriously damaged structural systems which can be repaired, and the last three categories (marked in red) having been allocated to severely damaged, nonusable buildings which should be demolished since no technical and economically justified solutions can be adopted for their repair and strengthening. Besides the main classification, the adopted methodology requires that certain general physical data be gathered on the given inspection forms. Such data include size of the building, structural system, nonstructural members, foundation conditions, type of damage, owner of the building, and its function. These follow the Uniform Methodology for estimation of damages caused by natural disasters, enacted by the Federal Bureau of Statistics of Yugoslavia.

The classification of damage and usability of structures was carried out on about 57,000 buildings, directed by the Republic and District Commissions for Classification of Buildings Damaged by Earthquakes on the Territory of SR Montenegro. 500 engineers and technicians from the whole country were organized into about 100 working groups and 20 teams of sepcialists for classification of damage of industrial structures, port and harbor facilities, regional and local infrastructure network, and other structures of vital importance.

Tabular presentations of data for building structures classified in six coastal and six continental communes have been provided in Tables A-1 and A-2, both totalled, and also broken down for each commune separately. A summary presentation of overall earthquake effects has been given fr the three basic categories of damaged buildings, usable (green), temporary nonusable (yellow), which should be repaired and strengthened, and nonusable (red) buildings which should typically be demolished. From the summary presentation of data of the classification made on damage degree and usability, it has been found that 54% of buildings are usable, 24% can be repaired and strengthened, and 22% should be demolished since their repair is either technically or economically unacceptable. This classification with all detailed data about the structural systems, size of the building, function and owners, contains the overall data that illustrate the scale of the problem, and the specific measures with priorities to be undertaken in reduction of earthquake consequences. This classification provides data which together with an adequate economic analysis of selected samples of building categories, can be used as a reliable basis for evaluation of economic losses and planning of measures for first meeting housing requirements and then revitalizing both economic and noneconomic activities.

A.3 Maximum Ground Acceleration Distribution

On the basis of the preliminary analysis of acceleration records obtained by three-componental accelerographs SMA-1 installed in the coastal and continental communes of SR Montenegro, maximum acceleration amplitudes have been obtained for each component separately, and have been used to estimate of the spatial distribution of maximum acceleration. Maps of acceleration distribution have been elaborated for N-S, W-E and also vertical components, respectively. The W-E component is presented in Figure A-1. This presentation of spatial ground acceleration distribution gives a synopsis of the influence of focal mechanism, topography, local geology and local soil conditions on the recorded maximum ground accelerations. The elongated shape of acceleration isolines which are either parallel or slightly inclined with respect to the coastal line indicates the predominant influence of the focal mechanism of the April 15, 1979 earthquake upon the maximum ground acceleration distribution, with a tendency of sharp decrease toward the continent.

Table A-1. Classification of Buildings Damaged in the Montenegro Earthquake of April 15, 1979 Presentation of Total Number of the Buildings (Source: Petrovski, J. et al., 1983) (Page 1 of 2)

Damage level of buildings	Classification of damage buildings in the community											
	ULCINJ		BAR		BUDVA		TIVAT		KOTOR		H. NOVI	
	No. Bldg's	%	No. Bldg's	%	No. Bldg's	%	No. Bldg's	%	No. Bldg's	%	No. Bldg's	%
Non damaged or slightly damaged	1.963	35	4.493	44	1.454	58	2.020	70	2.943	51	3.622	62
To be repaired	964	17	2.067	20	437	19	496	17	1.406	24	1.247	21
To be demolished	2.708	48	3.712	36	577	23	386	13	1.417	25	953	17
Total	5.655	100	10.272	100	2.518	100	2.902	100	5.766	100	5.822	100

Table A-1. Classification of Buildings Damaged in the Montenegro Earthquake of April 15, 1979 Presentation of Total Number of the Buildings (Source: Petrovski, J. et al., 1983) (Page 2 of 2)

Damage level of buildings	Classification of damage buildings in the community													
	CETINJE		NIKŠIĆ		TITOGRAD		DANILOVGRAD		IVANGRAD		KOLAŠIN		TOTAL	
	No. Bldg's	%	No. Bldg's	%	No. Bldg's	%	No. Bldg's	%	No. Bldg's	%	No. Bldg's	%	No. Bldg's	%
Non damaged or slightly damaged	3.160	45	2.490	74	4.971	83	4.840	79	649	88	498	69	33.123	58
To be repaired	2.431	34	620	18	891	15	1.081	17	84	12	184	26	11.958	21
To be demolished	1.507	21	276	8	116	2	222	4	2	0	35	5	11.911	21
Total	7.098	100	3.386	100	5.978	100	6.143	100	735	100	717	100	56.992	100

Table A-2. Classification of Buildings Damaged in the Montenegro Earthquake of April 15, 1979 Presentation of Total Area of the Buildings (Source: Petrovski, J. et al., 1983)

Damage level of buildings	Classification of damage buildings in the community																	
	ULCINJ		BAR		BUDVA		TIVAT		KOTOR		H. NOVI		CETINJE		DANILOVGRAD		TOTAL	
	Area (m^2)	%	Area (m^2)	%	Area (m^2)	%	Area (m^2)	%	Area (m^2)	%	Area (m^2)	%	Area (m^2)	%	Area (m^2)	%	Area (m^2)	%
Non damaged or slightly damaged	266.429	32	595.601	40	348.496	55	362.316	67	465.463	50	938.057	69	480.631	51	483.708	80	3.940.701	54
To be repaired	176.355	21	427.118	29	141.519	22	101.812	19	259.127	28	241.429	18	297.457	32	100.702	17	1.745.518	24
To be demolished	381.987	47	473.583	31	147.149	23	80.156	14	198.418	22	170.416	13	159.955	17	18.120	3	1.629.784	22
Total	824.771	100	1.496.302	100	637.164	100	544.284	100	923.008	100	1.349.901	100	938.043	100	602.530	100	7.316.003	100

The maximum acceleration distribution map of the W-E component presented in Figure A-1 represents a linear interpolation of the recorded maximum acceleration. It is further used to determine maximum accelerations and equivalent ground accelerations in about 360 subcommunes and settlements of six coastal communes and Centinje. Equivalent ground acceleration is a parameter similar to effective ground acceleration and is directly connected with structural response in that the measure considers frequency and amplitude content of the recorded earthquake time histories, local soil conditions, duration of earthquakes, number of dominant response pulses, dynamic characteristics of structures, and nonlinear structural response. This measure is one of the most effective earthquake parameters for presentation of structural damage and vulnerability, and has been used for the development of empirical vulnerability functions presented in Chapter 2.

A.4 Damage Distribution and Observed Vulnerability

Based on the established data bank on damage and usability of the buildings as well as on the distribution of maximum and equivalent ground accelerations, different analyses may be performed for presentation of earthquake effects in the form of tables, graphs or maps. One can summarize the relationships between damage and each of the following categories: structural types (stone masonry, brick masonry, strengthened masonry, reinforced concrete frame and wall structures, timber, steel and others), usage (residential, health and social welfare, public services, educational, tourism and catering, economical, industrial and others), local soil conditions, type of foundation structure, number of stories, and other relevant structural and environmental parameters. Several of these summary relationships are found both here and in Chapter 2. The most general of them, as shown in Figure A-2, are the cumulative presentation of damaged buildings to be demolished and total damaged buildings with respect to the distance of the considered urban area from the causative fault. The high level damaging effects are evident for distances up to 50 kilometers from the causative fault, a finding which is in rather good correlation with the maximum ground acceleration distribution as presented in Figure A-3. The second general presentation is given in Figure A-4, where, on the basis of the presented data in Tables A-3 and A-4, relationships of cumulative damage as well as percentage of total existing area of the buildings are presented for buildings to be demolished, repairable buildings, and also total unusable buildings and with respect to maximum ground acceleration. These general relationships presented in Figures A-2, A-3 and A-4 may be used for pre-earthquake or post-earthquake immediate assessment of damage level in other countries of the Mediterrenean region with similar seismic conditions and similar structural types of buildings. More detailed presentation of empirically developed vulnerability functions for different types of structures and usage are given in Chapter 2. The geographic distribution of observed vulnerability of stone masonry, brick masonry, and reinforced concrete frame buildings are given in Figures A-5 through A-7 for the affected region of Montenegro due to the earthquake of April 15, 1979. This type of data and similar ones elaborated for the specific urban areas may be used for physical and urban planning as a basis of the rehabilitation programs in the earthquake affected regions and urban areas and in conjunction with detailed consideration of the existing stock of buildings. With appropriate consideration of economic parameters for repair and strengthening of earthquake damaged buildings and with construction of new buildings having more favorable structural types to resist earthquakes during rehabilitation programs, much more favorable conditions will be created for reducing seismic risk.

Only a few general and detail tables, diagrams, and maps are presented in Chapter 2 and this Appendix, from the analysis of the effects of Montenegro Earthquake of April 15, 1979 in Yugoslavia. However, the intention here is

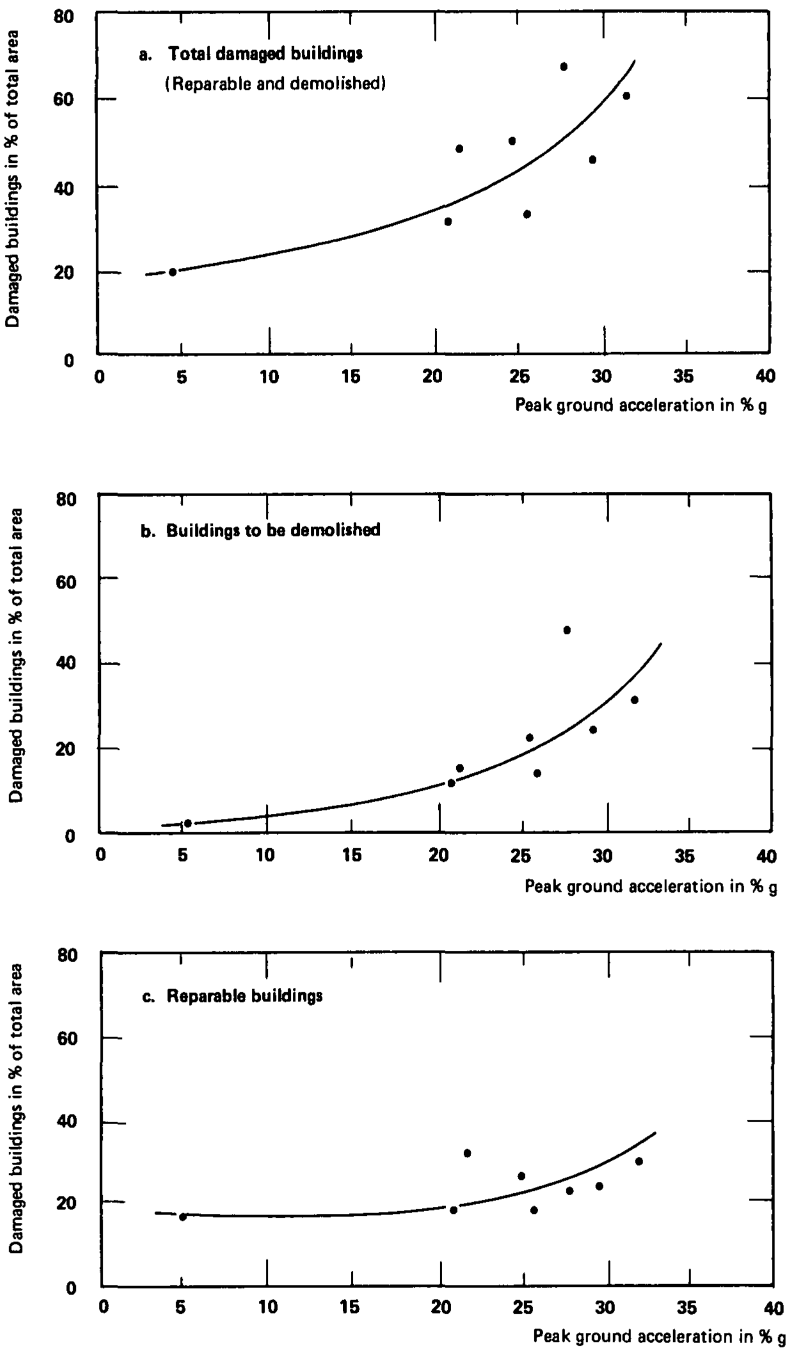


Figure A-4. Montenegro Earthquake 1979. Damaged Buildings as a Percent of Total Existing Area of Constructed Buildings (Source: Petrovski, J. et al., 1981)

Table A-3. Montenegro Earthquake of April 14, 1979: Number of Buildings Damaged Relative to Levels of Ground Shaking (Source: Petrovski, J. et al., 1983)

Number	Communities	Distance from the fault (km)	No. of Bldgs	Damage Analysis		Ground Acc. in % of g	Intensity MCS
				Green %	Red + orange %		
1	ULCINJ	5.0	5.655	35	65	27.4	9+
2	BAR	9.5	10.272	44	56	32.1	9+
3	BUDVA	6.5	2.518	58	42	29.2	9+
4	TIVAT	16.5	2,902	70	30	24.8	9+
5	KOTOR	19.5	5.766	51	49	24.3	9
6	H. NOVI	8.0	5.822	62	38	21.5	8+
7	CETINJE	24.0	7.098	45	55	22.2	8
8	DANILOVGRAD	49,0	6.143	79	21	4.6	7+
9	TITOGRAD	50.0	5.978	83	17	4.9	7+
10	NIKŠIĆ	58.5	3.386	74	26	7.5	7
11	IVANGRAD	117.5	735	88	12	(2.0)	6+
12	KOLAŠIN	95.0	717	69	31	(2.2)	6+
TOTAL :			56.992	58	42	-	-

Table A-4. Montenegro Earthquake of April 15, 1979: Area of Buildings Damaged Relative to Levels of Ground Shaking (Source: Petrovski, J. et al., 1983)

Number	Communities	Distance from the fault (km)	Area of Bldgs (m^2)	Damage Analysis		Ground Acc. in % of g	Intensity MCS
				Green %	Red + orange %		
1	ULCINJ	5.0	824.771	32	68	27.4	9+
2	BAR	9.5	1.496.302	40	60	32.1	9+
3	BUDVA	6.5	637.164	55	45	29.2	9+
4	TIVAT	16.5	544.284	67	33	24.8	9+
5	KOTOR	19.5	923.008	50	50	24.3	9
6	H. NOVI	8.0	1.349.901	69	31	21.5	8+
7	CETINJE	24.0	938.043	51	49	22.2	8
8	DANILOVGRAD	49.0	602.530	80	20	4.6	7+
TOTAL :			7.316.003	54	46		

to illustrate how efficient and rational use of these data may be made in post-earthquake damage evaluation, planning and performance of rehabilitation programs, pre-earthquake assessment of the expected vulnerability and seismic risk, and planning of future measures and policy for mitigation of seismic risk on the level of the communes, regions, and the entire country.

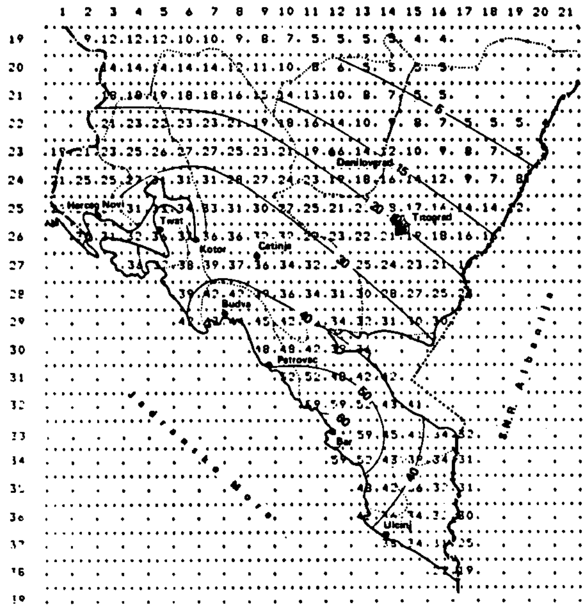


Figure A-5. Montenegro Earthquake, 1979. Distribution of Observed Vulnerability on Stone Masonary Buildings (Source: Petrovski, J. et al., 1983)

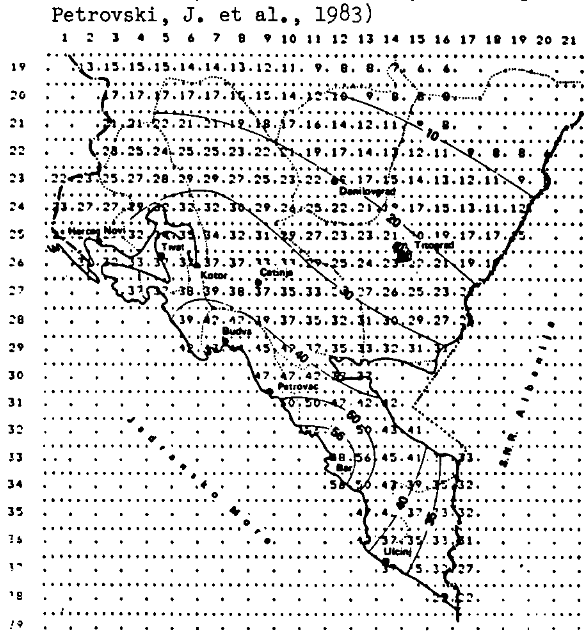


Figure A-6. Montenegro Earthquake, 1979. Distribution of Observed Vulnerability for Brick Masonary Buildings (Source: Petrovski, J. et al., 1983)

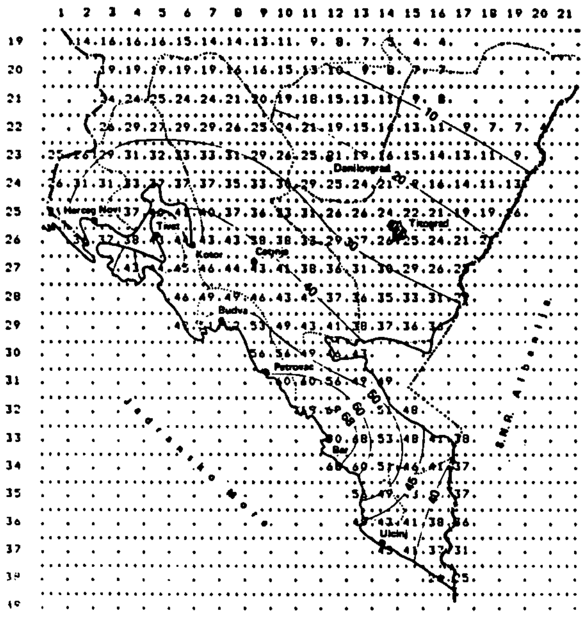


Figure A-7. Montenegro Earthquake, 1979. Distribution of Observed Vulnerability on Reinforced Concrete Frame Buildings (Source: Petrovski, J. et al., 1983)

A P P E N D I X B

APPENDIX B

SIMPLIFIED PROCEDURES FOR THE DECISION ON POST-EARTHQUAKE INTERVENTION

In this appendix, supplementary information is given concerning some of the issues discussed in Chapter 3 and related to decision procedures for post-earthquake rehabilitation of existing structures.

The main topics covered by this appendix are

1. Practical guidelines for the estimation of the actual base shear force capacity of frame, shear wall, and masonry structures.

2. Background on the simplified methodology for examination of reinforced concrete frame structures and decision on rehabilitation.

3. Illustrated applications of the simplified methodology.

B.1 Estimation of the Required and the Actual Base Shear Force Capacity of an Existing Structure

This estimation is needed in order to determine the value of the strength factor, namely,

$$R = \frac{S_{cap}}{S_{req}}$$

as part of the methodology explained in Chapter 3.

A complete examination of the seismic behavior and resistance of an existing structure could be made by dynamic analysis both in the elastic and in the post-elastic range. This examination requires definitions of both the damage and also the condemnation thresholds in the form of design accelerograms, as well as complete and exact information about the structure. For a reinforced concrete structure, this information would consist of the exact location and size of all structural and nonstructural members, the quantity and location of the reinforcement - both longitudinal and transverse - in all structural members, the actual quality of built-in materials for structural as well as for nonstructural elements, and the state of the connections between the structure and the infill masonry. This information is generally either not available, or incomplete; it is generally not obtainable in an acceptable period of time and at an acceptable cost.

So it is not reasonable to resort to lengthy and sophisticated analysis methods where only incomplete input data are available. More direct methods are quite acceptable for the needs of decision making.

B.1.1 Estimation of the Actual Base Shear Force Capacity of the Existing Structure (S_{cap})

S_{cap} is a function of the probable state of stress in the structure under seismic loads, the actual quality of the materials, and the bearing capacity of the most important structural members for the expected state of stress.

According to the methodology presented here, S_{cap} is to be assessed in accordance with the basic assumptions admitted by the codes for the demonstration of the lateral force resistance of structures, i.e., by linear analysis and without taking in account the contribution of nonstructural building components.

So, in terms of strength, S_{cap} may be defined as the lateral force for which a first significant section at the ground floor reaches its limit strength.

S_{cap} may be determined as follows:

1. make a linear analysis for a conventional base shear force (e.g., $S_{conv} = 0.10\ W$) and obtain a M_{conv} diagram;

2. based on the respective moment diagram, identify the sections where the first yield is likely to appear;

3. for the respective columns or walls, assess the axial load and then with the aid of an interaction diagram, assess the M_{cap} of the respective sections;

4. compare the bending moments M_{conv} with the M_{cap} found, and determine

$$\gamma = \frac{M_{cap}}{M_{conv}}$$

and then determine

$$S_{cap} = \min \gamma\ M_{conv}$$

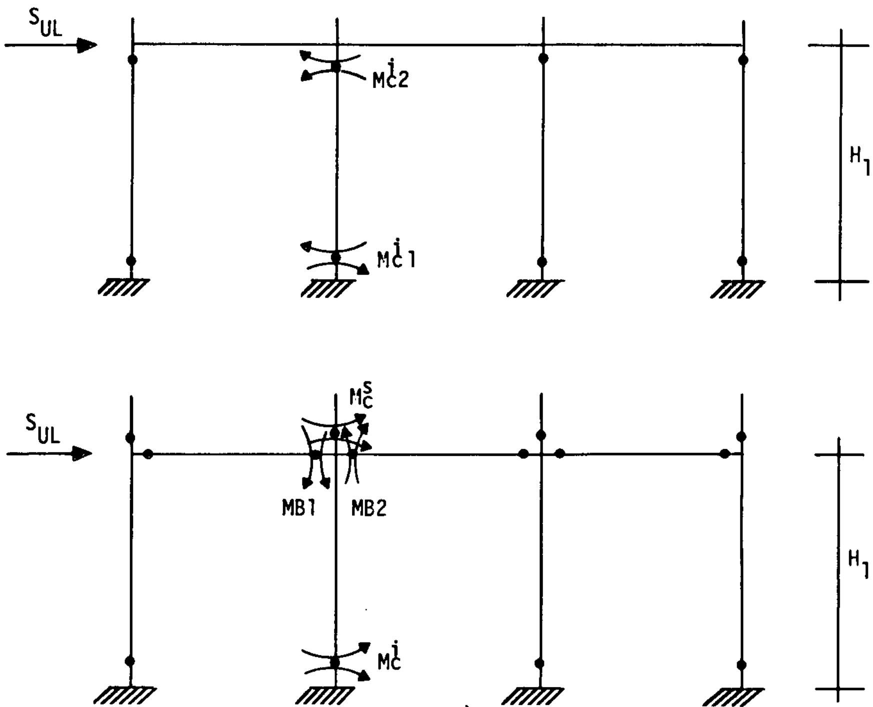
The elastic analysis for S_{conv} can be performed by any available method. If adequate computation facilities and programs are available, then the most direct method is to analyze the entire structure. If not, it can be assumed that the critical level is the ground floor, and the analysis, based on approximate methods, may be restricted to this level.

B.1.2 Estimation of the Equivalent Base Shear Force at Condemnation Threshold

For a complete representation of the structural performance capacity, an estimation of the equivalent base shear force at condemnation threshold Sc_{cap} will also be needed. Generally, it is not defined by the codes. Therefore, a conventional value of $Sc_{cap} = 0.8\ S_{UL}$ is suggested here. Consequently S_{UL}, the ultimate base shear force capacity is also to be assessed.

The ultimate strength is defined as the state in which the structure at a given level, tends to become a mechanism. Hence, the ultimate base shear force is defined here as the force which leads to the appearance of plastic hinges in all critical sections at ground level. The location of these critical sections specifically depends on the given structure. Therefore, the only examples that could be given in Figure B-1 are for a classical type frame structure and for a classical shear wall structure.

As in all such approaches, the order of appearance of the yielding is indifferent and therefore need not be investigated. The limitations of this approach are evident. The approach is valid if:

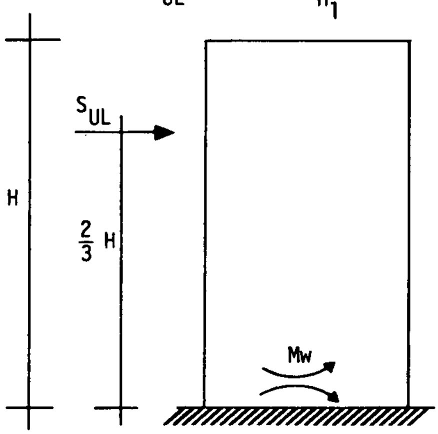


a)

S_{UL} cap = min of: $$S_{UL} = \sum \frac{Mc1^{i} + Mc2^{i}}{H_1}$$

or

$$S_{UL} = \sum \frac{MB1 + MB2 + Mc^{s} + Mc^{i}}{H_1}$$



$$S_{UL} = 1.5 \sum \frac{Mw}{H}$$

b)

Figure B-1. Estimation for S_{UL} for Reinforced Concrete Frame and Shearwall Structures

- the ductile behavior of all significant structural members is secured;
- the dispersion of the stiffness of the significant members is low, and yield occurs at short enough intervals in all members, so that in no critical section could the concrete have reached its ultimate strain before steel yielding has appeared in all the other critical sections.

For frame structures, a previous elastic analysis - even by approximate methods - is useful in identifying the critical sections. For shear wall structures, with rare exceptions, critical sections lie at ground level if there is a stiff infrastructure, or at foundation level if such an infrastructure is not available.

To illustrate this approach, take the frame structure in Figure B-1-a.

$$S_{ULcap} = \sum \frac{M_{B1} + M_{B2} + M_c^s + M_c^i}{H}$$

where H = story height, M_B, M_c = ultimate bending moments in the end sections of beams and columns (see Figure B-1(a)), S_c(cap.) = 0.8, and S_{UL} = conventional base shear force at condemnation threshold.

For the reinforced concrete bearing wall of constant height (Figure B-1(b)), it may be assumed that:

$$S_{UL} = \sum 1.5 \frac{M_w}{H}$$

where M_w = the ultimate bending moment of the lateral load bearing shear walls at their base section, H = the height of the shear walls, and $S_{c\ cap}$ = 0.8 S_{UL}.

Of course, the existence of vertical gravitation loads has to be kept in mind for columns and walls as well as for the beams. Therefore, interaction diagrams are to be utilized in determining M_{UL} and then S_{UL}.

For combined frame and shear wall structures, a previous elastic analysis is unavoidable in order to determine the participation of the columns in taking up a significant part of the base shear. Note that in combined frame and bearing wall structures, in which vertical loads are carried jointly by the columns and the walls, while the lateral loads are carried mostly by the walls, the fixity of the latter at basement or foundation level is also to be examined. If this fixity is not ensured, then the resulting problem is to be incorporated into any rehabilitation concept.

B.1.3 Estimation of the Required Average Base Shear Force Capacity According to the Aseismic Codes for a Similar Building

The assessment of the required base shear force capacity (S_{req}) according to the codes needs no elaborate comment.

$S_{req.}$ is dependent on the specific structure under examination with respect only to the outline dimensions (base, height, number of stories) and the structural type (RC frames with or without significant masonry infill, RC bearing walls, masonry bearing walls), as well as the importance or occupancy degree of the building.

The probable fundamental period is to be deduced by accepted approximation formulas. Through use of the design spectra and code coefficients, S_{req} is calculated from the equation:

$$S_{req} = C_s W$$

where C_s = the base shear force coefficient and W = the total vertical load.

B.1.4 Remarks on Preceding Methods

As regards the two distinct levels of base shear force capacity--the level of damage and the condemnation threshold, it should be pointed out that:

a) Figure 3-6 provides that a more comprehensive approach, if needed, to determine them.

b) for frame structures, the variation in the axial load in columns due to the overturning moment is not computed using these approximate analysis methods. This could make some difference when entering with the axial load in the interaction diagram. But usually the variation of the axial load due to the overturning moment does not exceed ± 20%, so that the error is not significant for the decision.

c) the ground story was considered to be the most critical. This is generally true for regular structures classified in Chapter 3 as good. For structures classified as "acceptable," the weak zone is probably the most critical.

Two special problems arise with respect to using these methods for skeleton structures.

The first is the structural role of the infill masonry. Most codes explicitly disallow its consideration in calculations. Nevertheless, past earthquakes have produced numerous examples of skeleton structures which, when examined by code conventions, should have collapsed or have been heavily damaged, but which nonetheless survived as a result of the infill masonry and have withstood the earthquake even when moderately damaged. For such cases, these methods only relate such structures to code provisions and do not estimate the actual lateral force resistance. In all cases where the density, dimensions, quality of materials, and location of the infill walls justifies the assumption that infill masonry plays an important structural role, this can be taken into consideration, with due regard for the reliability of the input analysis assumptions. In order to estimate the actual lateral force resistance, the analysis is then to be carried out by nonconventional methods of appropriate sophistication whose description is beyond the aims of this manual. But apart from these qualifications, the same decision methodology on intervention is valid.

The second problem concerns irregular and disorderly skeleton structures which can hardly be handled in the conventional way because of the unclarity of the state of stress for lateral force action. For these structures, it is advisable to begin by assimilating them to a more classical structural type and then to examine them as such. If this examination suggests maintaining them as they are (which is improbable), then a more sophisticated analysis would be needed to decide on the needed intervention.

B.1.5 Estimation of $S_{cap.}$ for Masonry Bearing Wall Structures

The foregoing methods have been explained in terms specific to reinforced concrete structures. For masonry structures, the problem is in some ways simpler because only the strength is to be examined when actual expected stresses are compared to some permissible values. In other respects, a standard detailed decision methodology is more difficult to construct owing to the larger variety of structural layouts and especially the large dispersion of the quality and characteristics of the material called masonry. Therefore, at present, only the simple method explained below is available.

The base shear force capacity can be expressed as

$$S_{capX.} = \tau_{ox} \; Ax = \sim 0.3 \; \sigma_N \; \sum Ax$$

in which $\tau_{ox} = 0.3\ \sigma_N$ is a mean value of the ultimate shear stress in a horizontal section (and is highly dependent on the friction, and consequently on the effective normal stress due to gravitational loads) and Σ Ax = the sum of the horizontal areas of the walls parallel to the X-direction.

B.2 The Basis of the Simplified Methods Applied to Examination Existing Frame Structures (explained in Section 3.4.1.5 and in the flow chart of Figure 3-6)

The approach explained in Sections 3.4.1.1 to 3.4.1.4 and the flow chart in Figure 3-6 (page 2) are somewhat too general. In order to be comprehensive, all theoretically possible combinations of layout quality, strength, stiffness and ductility have been envisaged. Some of them are highly improbable. Low strength is not likely to be with associated considerable stiffness and ductility; a poorly engineered structure having a disorderly layout is not likely to have good strength or ductility.

At the same time, the examined characteristics are interrelated. The gravitational load, the base shear force, and the resulting banding moments and deformations can be deduced from each other, so that the state of the structure could be described by a single one of these characteristics.

B.2.1.1 Use of State of Stress as a Descriptor

In the following simplified methodology this basic criterion has been selected to be the state of stress under gravitational loads, in form of a mean compression factor.

$$n = \frac{\sigma_o}{f_c} = \frac{\sum N}{f_c \quad A_c}$$

where

σ_o = mean compression stress in the columns under axial gravitational loads;

ΣA_c = total area of the significant columns cross section;

fc = concrete strength (prismatic or cylindrical)

It can easily be deduced that if

$$S = Cs\ N$$

then

$$\frac{e}{h} = \frac{m}{n} = Cs\ \alpha\ \frac{H}{h} \tag{B-1}$$

where

Cs = base shear force factor

α = factor locating the inflection point on the story height as a function of the restraint degree at the top and at the bottom of the column

$\frac{H}{h}$ = column slenderness (ratio of the story height to the column width)

For the demonstration and notations, see Figure 3-6. Clearly,

$$R = \frac{S_{cap}}{S_{req}} = \frac{Cs\ cap}{Cs\ req}$$

So, for R = 0.8, Cs cap = 0.8 Cs req. Consequently,

$$\frac{e}{h} = (0.8\ Cs\ req)\ (\frac{H}{h})$$

$$(R = 0.8)$$

$$\frac{e}{h} = (0.5\ Cs\ req)\ (\frac{H}{h})$$

$$(R = 0.5)$$

Starting with these data in a dimensionless m-n interaction diagram, the situation looks as in Figure 3-10. It follows that the values of R could be derived directly from computing n act. and comparing it to the limit values $n_{1.0}$, $n_{0.8}$, and $n_{0.5}$. These limit values depend on the specific interaction diagram, which is a function of the steel ratio and of the ratio between steel - and concrete strength, $\frac{fy}{fc}$. They depend also on the specific values of $\frac{e}{h} = f\ (Cs\ req., \frac{H}{h})$. As such they seem at first sight have a large dispersion. But, after examining existing structures as well as the provisions of the technical regulations governing the design of reinforced concrete structures in some given periods, we have concluded that the dispersion is rather low. Usually,

$$\rho_t = 0.01 - 0.02 \quad \text{and} \quad \mu = \frac{fy}{fc} = 20$$

so $\rho_t\ \mu$ varies between 0.20 and 0.40.

At the same time, for frame structures the usual values are Cs req = 0.05 - 0.10, α = 0.5 - 0.7, and $\frac{H}{h}$ = 6 - 10. So usually,

$\frac{e}{h}$ = 0.150 - 0.700 when R = 1.0

$\frac{e}{h}$ = 0.120 - 0.560 when R = 0.8

and $\frac{e}{h}$ = 0.075 - 0.350 when R = 0.5

Putting all this on a design chart, the area of possible variation results is shown in Figure 3-12. The extreme limits of variation are n = 0.60 and n = 0.20. Also,

- If n > 0.60, then the structure has no chance to achieve R > 0.5 and consequently should be strengthened.
- If n < 0.20, then R > 0.8, and no intervention is needed.
- Between these extreme values the structure may achieve any strength factors depending essentially of the value of Cs req and H/h. The influence of α and ϕ_t μ is less significant.

Assuming for ρ_t, ϕ, and μ, the constant mean values of ϕ_t N = 0.2 and α = 0.6, diagrams could be drawn to determine directly the possible values of R as a function of n = α/fc and Cs req for a given value of H/h. Such a diagram is shown in Figure 3-7. Its use is explained in the same figure:

- for a known n and Cs req, the maximum estimated R could be estimated (point A), or
- for a known n and a desired R, Cs cap could be estimated.

B.2.1.2 Extension to Entire Structure

The deductions made on a single column in B.2.1.1 may be extended to the entire structure. This is more valid if there is a proportionality between the distribution of the axial load on the columns and their stiffness. In such case n = σ_o/f'c may be assumed to be average value of

$$\text{med } n = \frac{W}{\sum A_c \text{ fc}}$$

where

W = the total vertical load

and $\sum A_c$ = the sum of the column cross sections

If there is an important dispersion of the individual n values and of the stiffness of the columns, then the most significant columns for lateral force resistance are to be selected, the average n value is to be determined for this group, and the respective deductions are to be made starting from this value.

B.2.2 Special Comments on Frame and Skeleton Structures

Some comments are worth making on the state of stress and ductility of the frame and skeleton structures - as a function of the period of the period of their design and completion.

Before 1940 in Europe, concrete strength for common buildings was usually about 12 N/mm^2 cubic strength and about 10 N/mm^2 for prismatic strength. The allowable stress for axial loads was about 4 N/mm^2.

After 1940, according to the widely used DIN code, the columns were to be dimensioned by an ultimate strength method with a safety factor of three. Codes based on the ultimate strength method, valid in some East-European countries between about 1950 and 1960, required a safety factor of only two for centrically loaded columns. The required minimal values for the steel ratio were generally of 0.8%. To economize on steel, it was recommended never to go beyond about 1.5%. The importance of ductility was acknowledged only in the last decades. Before 1960, the stirrups in the columns where spaced at 12 to 15 times the diameter of the longitudinal reinforcement and to a maximum limit of 35 cm.

According to the above considerations, a correctly engineered structure of this period would probably have:

- med. n = 0.30 - 0.75
- ρ_t = 1 - 1.2%
- a very low transverse reinforcement ratio

Consequently, a structure of this period could be coefficiently strong, but in no case adequately ductile. First, the balance point lies usually at around n = 0.25 - 0.30, and above this value even failure in axial force and bending is brittle if, as is highly probable, the concrete is not well confined. Second, for values of $n > 0.5$, shear failure before bending failure is possible (even for columns with a normal slenderness), and there is no transverse reinforcement to prevent it.

B.3 First Example: An Apartment Building

B.3.1 Description

Dimensions	21.20 x 9.80 m
Number of stories	Basement + 4
Usage category	dwelling
Structure:	Two way reinforced concrete frames (see Figure B-2). The basement is stiffened by r.c. contour walls. Thin hollow brick infill walls.

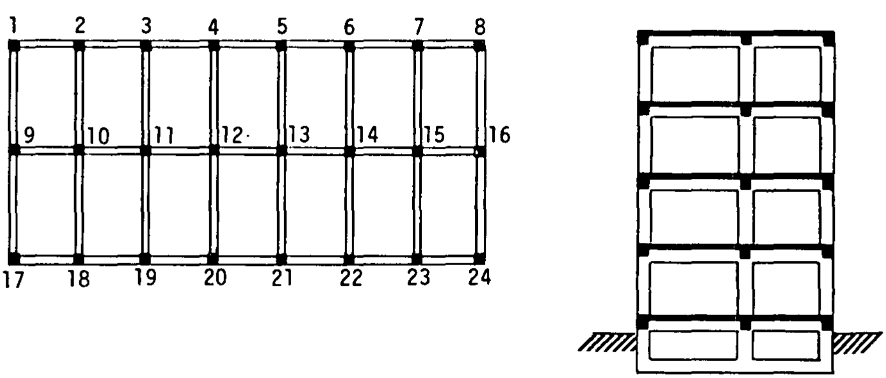


Figure B-2. Layout of the Structure of Example 1

B.3.2 Examination of the Existing Structure

B.3.2.1 Determining S req.

$$W = (21.2 \times 9.80)m^2 \times 4 \text{ stories} \times 12 \frac{kN}{m^2} = 10000 \text{ kN}$$

Assuming that the fundamental period for such a building should be about 0.4 - 0.5 sec (T = 0.10 m), then according to the respective seismic code:

$$Cs \text{ req} = 0.32 \times 2 \times 0.20 \times 0.8 = 0.1024 \approx 0.10$$

and $$S \text{ req} = Cs \text{ req } W = 0.10 \times 10000 = 1000 \text{ kN}$$

B.3.2.2 Overall Examination of the Structural Layout

There are frames with a large number of correctly shaped and symmetrically distributed bays in both principal directions. The beams are sufficiently stiff (the height to span ratio $\frac{h}{l} = \frac{1}{10}$ in both directions). The infrastructure is very stiff so that the structure may clearly be considered as fixed at ground floor level.

So the stress pattern under lateral loads is well-defined and can be analyzed by the usual methods.

Consequently, the structure as a whole may be considered to be "good".

However, since the dimensions of the column section are 50 x 20 cm there is a significant difference of stiffness in the two principal directions. In the longitudinal direction, the columns are very slender (the ratio $\frac{h}{H} = \frac{1}{15}$). Due attention should be given to problems arising from these facts.

B.3.2.3 Determining the Strength Factor R

The simplified procedure illustrated in Figure 3-6, page 3 is used.

The total area of the columns cross section,

$$\Sigma A = 24 \times 50 \times 20 = 24{,}000 \text{ cm}^2 = 2{,}400{,}000 \text{ mm}^2$$

$$\sigma_o = \frac{W}{\Sigma A} = \frac{10000000}{2{,}400{,}000} = 4.2 \frac{N}{mm^2}$$

$$n = \frac{\sigma_o}{f'c} = \frac{4.2}{10} = 0.42$$

Since 0.3 < n < 0.5, proceed to further examination.

The transverse and the longitudinal directions are to be examined separately because of the significant difference of the columns' stiffness in the two directions.

B.3.2.4. Further Examination from the Transverse Direction

$$\frac{H}{h} = \frac{300}{50} = 6$$

Estimating also $\alpha = 0.6$, we obtain from the respective diagram illustrated in Figure 3-7.

$R = 0.60$ $(0.5 < R < 0.8)$

As a result, the category of the structure is B (good layout + R = 0.5 - 0.8)).

Displacements

The estimated drift at ground story, under a lateral seismic load equal to S req = 100 tons is $\Delta = 0.2 - 0.3\%$. So,

$$\Delta < \bar{\Delta}_D$$

Ductility

The execution drawings are available. The columns have a longitudinal reinforcement 6 Ø 14 bars, ($\rho_t = 0.9\%$) and stirrups Ø 6 spaced at 20 cm.

There is no concern for the confinement of the concrete in the vicinity of the joints (narrow spacing of the stirrups) either in the columns or in the beams. Reinforcement is torsioned steel, that is, steel with comparatively brittle behavior.

So, as a whole, ductility conditions are to be considered as being not fulfilled.

Setting up the alternative rehabilitated concepts (according to the flow chart in Figure 3-6)

- strengthening type I (improving the ductility, e.g., by adding transverse reinforcement in form of stirrups or steel encasings covered by a thin layer of concrete protection)

- strengthening type III (improving strength and ductility, e.g., by jacketing the columns)

- strengthening type IV (adding strength and ductility by new structural members).

B.3.2.5 Further Examination from the Longitudinal Direction

For $\frac{H}{h} = 15$ and an estimated $\alpha = 0.6$, from the corresponding n - Cs req diagram, it follows that

$R \cong 0.8$ $(R << 0.5)$

Consequently, the category of the structure is C (good layout + R < 0.5).

Displacements

The estimated drift is $\bar{\Delta} = \frac{1}{50} = 2\%$, exceeding even the condemnation threshold ($\bar{\Delta}_c = 1.5\%$). This leads directly to the conclusion that strengthening type III or type IV is needed).

The ductility conditions are not fulfilled as shown under B.3.2.4.

Alternative strengthening concepts are therefore

type III or
. type IV

B.3.2.6 Estimation of the Probable Costs of the Strengthening

(Damage degree of nonstructural members: middle to extensive); Probable efficiency index EF = 45% (from table in Figure 3-13).

B.4 Second Example: Another Apartment Building

B.4.1 Outline Description of the Building

Usage: apartments
Number of stories: Basement + 7 stories
Dimensions: inscribed in a rectangle 31 x 19m. (see Figure B-3)
Built area at ground level: 450 sq.m.
Total developed area: 3200 sq.m.
Type of structure: reinforced concrete skeleton (columns and beams with irregular layout) and plain brick infill walls

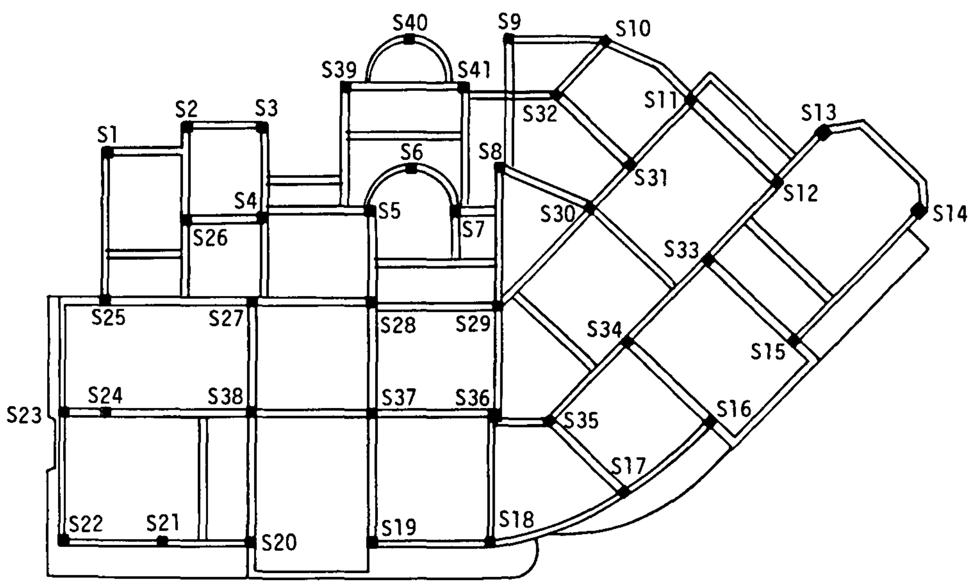


Figure B-3. Layout of the Structure of Example 2

B.4.2 Examination of the Existing Structure

B.4.2.1 Determining $S_{req.}$

Total weight W = 38300 kN

Assuming that the fundamental period of such a building should be about 0.7 sec (T = 0.1 x 7),

$c_{s\ req}$ = 0.20 x 2 x 0.20 x 0.8 = 0.064 (according to the Romanian code)

and S_{req} = 0.064 x 3830 = 2451 kN

B.4.2.2 Examination of the Structural Layout

The position of the columns is very irregular. There are some rows of columns and beams susceptible of forming frames, but their position is chaotic and their conformation is faulty because there are many non-colinearities of the respective beams. At the same time the position of a large number of beams has no correspondence with the layout of the columns (beams resting on beams).

In principle, this type of structure could be designed to have a satisfactory seismic resistance, so that the structure cannot be labelled as being necessarily faulty. But the state of stress can be determined only by extensive and sophisticated analysis, so that at first examination no clear image of this state results. As a whole, then, the structure could be classified as "unclear," possibly faulty because of some obvious faults (for instance weak and non-colinear beams).

It is to be noted that the dimensions of the columns at ground floor vary from 28 x 28 cm to 50 x 40 cm, and those of the beams from 15/30 to 260/60 cm.

The workmanship is average to poor.

B.4.2.3 Determining the Strength Factor R

Based on the simplified procedure illustrated in Figure 3-9, page 3, the total area of the columns' cross section

$$\Sigma A_c = 58772 \text{ sq. cm.}$$

and

$$\sigma_o = \frac{W}{\Sigma A_c} = \frac{3830000}{58772} = 6.51 \frac{N}{mm^2}$$

The concrete has an estimated strength $f_c = 12 \frac{N}{mm^2}$ so that

$$n = \frac{\sigma_o}{f'c} = 0.54$$

This value is beyond the limit of n = 0.5 so that strengthening is needed. Nevertheless, in order to illustrate the procedure, we proceed to further examination.

Assuming an average $\frac{H}{h} = 7$ and an average steel ratio of $\rho_t = 1\%$, from the corresponding $n - c_{s\ req}$ diagram, it follows that $R = 0.4$. From the superposition of an unclear layout and $R < 0.5$, it follows that the structure is to be classified in category E.

B.4.3 Constructing Alternative Rehabilitation Concepts

From the flow chart in Figure 3-6, page 3, the advisable rehabilitation is a strengthening of type IV.

B.4.4 Estimation of the Efficiency Index

The damage to nonstructural members is "moderate" (about 1/3 of the partitions and infill walls completely destroyed).

Extrapolating from the table in Figure 3-13, a probable efficiency index is

EF = 55 - 60%

B.4.5 Comments

In this case the decision is very rapid because of the unsatisfactory layout and the low strength factor. If we nevertheless had decided to proceed to further examination, it would have resulted that:

a) the drift at ground floor level (due to a lateral force equal to $S_{req} = 2450$ kN) is probably high but does not exceed the condemnation level;

b) with a high probability, the ductility conditions are far from being fulfilled. The structure was designed and completed in 1935-36. Additionally, it was not quite satisfactorily engineered even compared to valid regulations then.

The estimation of the strength factor R was made based on average values of n. For the needs of this case, this procedure was acceptable because the decision is evident from the beginning.

A P P E N D I X C

APPENDIX C

EXISTING METHODS FOR EVALUATION OF DAMAGE POTENTIAL

C.1 EXISTING CATEGORIZATION TECHNIQUES DISCUSSED

C.1.1 Proxy Categorization Methods: Uses and Limits

In the absence of sufficient data from Balkan region damage evaluation forms, substitute categories must be used to assess the vulnerability of existing buildings and their proposed replacements (through seismic rehabilitation or else demolition and replacement). Since categorization techniques are the least sophisticated method of risk assessment, they cannot be expected to encompass all pertinent structural factors. Categorization relies on judgmental rating of structures based primarily on the framing system, accompanied by factors such as the seismic code in effect (as indicated by construction date) and general structural parameters.

Categorization techniques are the basic means by which expected economic losses to the structures are calculated. More refined techniques often merely modify economic estimates derived initially from categorization techniques but based on a more exacting analysis of structural response.

Numerous categorization techniques exist in the literature but those listed below may be most appropriate for Balkan structures:

(1) summary curves developed by Haresh C. Shah and others,
(2) curves developed by Robert Whitman and others,
(3) curves developed by K.V. Steinbrugge and others, and
(4) curves developed by J.H. Wiggins and others.

In addition, it is useful to look at observed vulnerability relationships developed for Romanian structures. These observed vulnerability relationships strongly indicate that one may grossly underestimate the seismic vulnerability of some building types if only summary curves are used, as suggested above. In addition, a categorization of Turkish buildings is included, although associated vulnerability data are very limited to date for such structures.

Before outlining the categorization techniques listed above, it is necessary to mention some of their limitations. The statistical unreliability of such techniques can be partially indicated by the diverse loss estimates derived using the different techniques as applied to a structure (so that identical shaking hazard estimates are employed). For an unreinforced masonry structure, the following diverse loss estimates might be derived for two different hazard macrozones (hazard zones are rated from 0 to 5, with 5 representing portions of Japan, 4 representing Los Angeles, etc.).

In a hazard zone 3, the following annual expected losses (as a percent of replacement cost) might be calculated using diverse techniques:

$15.5 \cdot 10^{-4}$ based on Steinbrugge techniques

$11.9 \cdot 10^{-4}$ based on Whitman techniques

$4.4 \cdot 10^{-4}$ based on Wiggins techniques

For a hazard zone 4, the following results may be calculated:

$61.6 \cdot 10^{-4}$ based on Steinbrugge techniques

$51.6 \cdot 10^{-4}$ based on Whitman techniques

$36.4 \cdot 10^{-4}$ based on Wiggins techniques

Other measures of statistical uncertainty tend to be absent, or comparatively insignificant, except for Romanian data. Those data suggest that loss estimates might be an order of magnitude higher for some Romanian structures.

Other limitations of existing techniques include the following:

- as already suggested, modifications are needed for local building materials, inspection practices, etc.
- the distinction between structural and non-structural damage tends to be blurred
- there is an absence of good data on loss distribution,
- hazard data for the damaged and undamaged buildings could be improved with more strong-motion instrument data and also with regard to various ground failure hazards
- intensity scales tend to be used which involve use of discrete intervals and so give rise to numerous interpretive problems
- existing vulnerability models need to be related more completely to structural analysis
- loss estimates may depend on who makes repairs, who funds repairs, who makes the estimate, etc.
- loss estimates depend on an accurate assessment of buildings with little or no damage at given levels of groundshaking (and those comparatively undamaged buildings are generally of less interest)

Some of these problems may be overcome, in whole or in part, through eventual damage survey evaluations.

C.1.2 Proxy Categorization Methods: Summary

C.1.2.1 Sauter and Shah Techniques

Based on an examination of a broad range of previous vulnerability curves (including those referred to in other techniques discussed here), Sauter and H. Shah have developed a family of vulnerability curves, as illustrated in Figure C-1. To use such curves, the analyst must read off the expected loss at a given intensity. For instance, at intensity VI, a mean value of 8% of replacement cost is indicated for adobe structures. It shall be assumed that the Modified Mercalli Intensity (MMI) Scale has values identical with those in the Medvedev-Sponheuer-Karnik (MSK) Intensity Scale. (See Medvedev and Sponheuer, 1969.)

C.1.2.2 Whitman et al. Techniques

Based largely on 1971 San Fernando Valley earthquake data, with special emphasis on buildings with five or more stories, R.V. Whitman et al. damageability estimates are categorized in terms of Uniform Building Code (UBC)

Standards (U.S.), which require little or no seismic resistance in zones 0 and 1 (UBC 0-1) and considerably more resistance in zone 4. Use of these estimates would thus tend to be confined to highrise structures, and judgment would be required to compare broadly U.S. construction practices and those in Balkan countries. Some assistance on this matter is found in J.H. Wiggins et al. on L.A. City. Table C-1 summarizes Whitman et al. findings.

Table C-1. Mean Damage Ratios (as percents of replacement costs) of Various Classes of UBC Buildings at Given Modified Mercalli Intensities (Source: Whitman, 1975)

MM INTENSITY	CONSTRUCTION CLASS			
	UBC 0-1	UBC 2	UBC 3	ZONE S (UBC 4)
VI	0.22%	0.16%	0.13%	0.10%
VII	3.0%	1.9%	1.4%	1.2%
VIII	52%	18%	10%	5%
IX	100%	100%	45%	21%
X	100%	100%	100%	100%

In addition, Whitman and Cornell have also developed provisional probability distributions for selected "damage states." (See Table C-2) Such damage states are qualitative assessments of the degree of damage to the structure, and are only indirectly related to percent losses. However, probability distribution functions are useful in earthquake insurance studies and also can prove to be useful in estimating expected deaths, if, for instance, collapse is used as a major criterion.

C.1.2.3 Steinbrugge et al. Techniques

Based on examinations of many earthquakes, K.V. Steinbrugge et al. have used Insurance Services Offices (U.S.A.) building classifications to develop vulnerability curves. Those classifications are found in Table C-3. As with all other classification schemes referred to here, engineering judgment must often be used to determine degree of seismic resistance expected of a given structure.

Figure C-2 illustrates vulnerability curves for most of the classes mentioned and Table C-4 puts those curves into a tabular form. It should be noted that single family dwellings are not included in these classifications. For single family dwellings with wood frames, Table C-5 summarizes data by Steinbrugge et al. Also included in Table C-5 are mobile home vulnerability estimates.

Table C-2. Illustration of Building Damage by Distributions Whitman and Cornell, 1976, p. 352

DAMAGE PROBABILITIES (%) FOR PILOT APPLICATION OF SEISMIC DESIGN DECISION ANALYSIS

DESIGN STRATEGY	DAMAGE STATE	MODIFIED MERCALLI INTENSITY							DAMAGE RATIO (%)	CENTRAL VALUE
		V	VI	VII	VII.5	VIII	IX	X		
UBC 0-1	O - NONE	100	27	15	0	0	0	0	0 - 0.05	0
	L - LIGHT	0	73	48	21	0	0	0	0.05 - 1.25	0.3
	M - MODERATE	0	0	33	45	20	0	0	1.25 - 20	5
	H - HEAVY	0	0	4	29	41	0	0	20 - 65	30
	T - TOTAL	0	0	0	5	34	75	25	65 - 100	100
	C - COLLAPSE	0	0	0	0	5	25	75	100	100
UBC 2	O - NONE	100	47	20	0	0	0	0		
	L - LIGHT	0	53	50	36	10	0	0		
	M - MODERATE	0	0	29	52	53	0	0		
	H - HIGH	0	0	1	11	31	0	0		
	T - TOTAL	0	0	0	1	5	80	60		
	C - COLLAPSE	0	0	0	0	1	20	40		
UBC 3	O - NONE	100	57	25	5	0	0	0		
	L - LIGHT	0	43	50	48	25	0	0		
	M - MODERATE	0	0	25	41	53	20	0		
	H - HIGH	0	0	0	6	21	52	0		
	T - TOTAL	0	0	0	0	1	23	80		
	C - COLLAPSE	0	0	0	0	0	5	20		
ZONE S(4)	O - NONE	100	67	30	10	0	0	0		
	L - LIGHT	0	33	49	58	40	10	0		
	M - MODERATE	0	0	21	29	52	30	0		
	H - HIGH	0	0	0	3	8	58	0		
	T - TOTAL	0	0	0	0	0	2	90		
	C - COLLAPSE	0	0	0	0	0	0	10		

Table C-3. Building Classifications for Estimating Earthquake Losses (As Suggested by K.V. Steinbrugge et al.) (Page 1 of 2)

CLASS IV-B:

BUILDINGS HAVING A STRUCTURAL SYSTEM AS DEFINED BY THE NOTE (ABOVE) WITH EXTERIOR AND INTERIOR NON-BEARING WALLS OF ANY MATERIAL.

CLASS IV-C:

BUILDINGS HAVING SOME OF THE FAVORABLE CHARACTERISTICS OF CLASS IV-A BUT OTHERWISE FALLING INTO CLASS IV-B.

CLASS IV-D:

BUILDINGS HAVING (a) A PARTIAL OR COMPLETE LOAD CARRYING SYSTEM OF PRECAST CONCRETE, AND/OR (b) REINFORCED CONCRETE LIFT SLAB FLOORS AND/OR ROOFS, AND (c) OTHERWISE QUALIFYING FOR CLASSES IV-A, B, OR C.

CLASS IV-E:

BUILDINGS HAVING A COMPLETE REINFORCED CONCRETE FRAME, OR A COMPLETE FRAME OF COMBINED REINFORCED CONCRETE AND STRUCTURAL STEEL. FLOORS AND ROOFS MAY BE ANY MATERIAL WHILE WALLS MAY BE OF ANY NON-LOAD BEARING MATERIAL.

CLASS V: MIXED CONSTRUCTION

CLASS V-A:

1. DWELLINGS, NOT OVER TWO STORIES IN HEIGHT, CONSTRUCTED OF POURED-IN-PLACE REINFORCED CONCRETE, WITH ROOFS AND SECOND FLOORS OF WOOD FRAME.

2. DWELLINGS, NOT OVER TWO STORIES IN HEIGHT, CONSTRUCTED OF ADEQUATELY REINFORCED BRICK OR HOLLOW CONCRETE BLOCK MASONRY, WITH ROOFS AND FLOORS OF WOOD.

CLASS V-B:

ONE STORY BUILDINGS HAVING SUPERIOR EARTHQUAKE DAMAGE CONTROL FEATURES INCLUDING EXTERIOR WALLS OF (a) POURED-IN-PLACE REINFORCED CONCRETE, AND/OR (b) PRECAST REINFORCED CONCRETE, AND/OR (c) REINFORCED BRICK MASONRY OR REINFORCED BLOCK MASONRY. ROOFS AND SUPPORTED FLOORS SHALL BE OF WOOD OR METAL DIAPHRAGM ASSEMBLIES. INTERIOR BEARING WALLS SHALL BE OF WOOD FRAME OR ANY ONE OR A COMBINATION OF THE AFOREMENTIONED WALL MATERIALS.

CLASS V-C:

ONE STORY BUILDINGS HAVING CONSTRUCTION MATERIALS LISTED FOR CLASS V-B, BUT WITH ORDINARY EARTHQUAKE DAMAGE CONTROL FEATURES.

CLASS V-D:

1. BUILDINGS HAVING REINFORCED CONCRETE LOAD BEARING WALLS WITH FLOORS AND ROOFS OF WOOD AND NOT QUALIFYING FOR CLASS IV-E.

2. BUILDINGS OF ANY HEIGHT HAVING CLASS V-B MATERIALS OF CONSTRUCTION, INCLUDING WALL REINFORCEMENT; ALSO INCLUDED ARE BUILDINGS WITH ROOFS AND SUPPORTED FLOORS OF REINFORCED CONCRETE (PRECAST OR OTHERWISE) NOT QUALIFYING FOR CLASS IV.

CLASS V-E:

BUILDINGS HAVING UNREINFORCED SOLID UNIT MASONRY OF UNREINFORCED BRICK, UNREINFORCED CONCRETE BRICK, UNREINFORCED STONE, OR UNREINFORCED CONCRETE, WHERE THE LOADS ARE CARRIED IN WHOLE OR IN PART BY THE WALLS AND PARTITIONS. INTERIOR PARTITIONS MAY BE WOOD FRAME OR OF THE AFOREMENTIONED MATERIALS. ROOFS AND FLOORS MAY BE OF ANY MATERIAL. NOT QUALIFYING ARE BUILDINGS WITH NON-REINFORCED LOAD CARRYING WALLS OF HOLLOW TILE OR OTHER HOLLOW UNIT MASONRY, ADOBE, OR CAVITY CONSTRUCTION.

CLASS V-F:

1. BUILDINGS HAVING LOAD CARRYING WALLS OF HOLLOW TILE OR OTHER HOLLOW UNIT MASONRY CONSTRUCTION, ADOBE, AND CAVITY WALL CONSTRUCTION.

2. ANY BUILDING NOT COVERED BY ANY OTHER CLASS.

CLASSES VI-A, B, C, D, AND E: EARTHQUAKE RESISTIVE CONSTRUCTION

ANY BUILDING OR STRUCTURE WITH ANY COMBINATION OF MATERIALS AND WITH EARTHQUAKE DAMAGE CONTROL FEATURES EQUIVALENT TO THOSE FOUND IN CLASSES I THROUGH V BUILDINGS. ALTERNATIVELY, A QUALIFYING BUILDING OR STRUCTURE MAY BE CLASSED AS ANY CLASS FROM I THROUGH V (INSTEAD OF VI-A, B, C, D, OR E) IF THE CONSTRUCTION RESEMBLES THAT DESCRIBED FOR ONE OF THESE CLASSES AND IF THE QUALIFYING BUILDING OR STRUCTURE HAS AN EQUIVALENT DAMAGEABILITY.

Table C-3. Building Classifications for Estimating Earthquake Losses (As Suggested by K.V. Steinbrugge et al.) (Page 2 of 2)

CLASS I: WOOD FRAME
<u>CLASS I-A:</u> 1. WOOD FRAME AND FRAME STUCCO DWELLINGS REGARDLESS OF AREAS AND HEIGHT. 2. WOOD FRAME AND FRAME STUCCO BUILDINGS, OTHER THAN DWELLINGS, WHICH DO NOT EXCEED 3 STORIES IN HEIGHT AND DO NOT EXCEED 3,000 SQ. FT. IN GROUND FLOOR AREA. 3. WOOD FRAME AND FRAME STUCCO HABITATIONAL STRUCTURES WHICH DO NOT EXCEED 3 STORIES IN HEIGHT REGARDLESS OF AREA. <u>CLASS I-B:</u> WOOD FRAME AND FRAME STUCCO BUILDINGS NOT QUALIFYING UNDER CLASS I-A.
CLASS II: ALL-METAL BUILDINGS
<u>CLASS II-A:</u> ONE STORY ALL-METAL BUILDINGS WHICH HAVE A FLOOR AREA NOT EXCEEDING 20,000 SQ. FT. <u>CLASS II-B:</u> ALL-METAL BUILDINGS NOT QUALIFYING UNDER CLASS II-A.
CLASS III: STEEL FRAME BUILDINGS
<u>CLASS III-A:</u> BUILDINGS HAVING A COMPLETE STEEL FRAME WITH ALL LOADS CARRIED BY THE STEEL FRAME. FLOORS AND ROOFS SHALL BE OF POURED-IN-PLACE REINFORCED CONCRETE, OR OF CONCRETE FILL ON METAL DECKING WELDED TO THE STEEL FRAME (OPEN WEB STEEL JOISTS EXCLUDED). EXTERIOR WALLS SHALL BE OF POURED-IN-PLACE REINFORCED CONCRETE OR OF REINFORCED UNIT MASONRY PLACED WITHIN THE FRAME. BUILDINGS SHALL HAVE A LEAST WIDTH TO HEIGHT ABOUT GROUND (OR ABOVE ANY SETBACK) RATIO OF NOT EXCEEDING ONE TO FOUR. NOT QUALIFYING ARE BUILDINGS HAVING COLUMN-FREE AREAS GREATER THAN 2,500 SQ. FT. (SUCH AS AUDITORIUMS, THEATERS, PUBLIC HALLS, ETC.). <u>CLASS III-B:</u> BUILDINGS HAVING A COMPLETE STEEL FRAME WITH ALL LOADS CARRIED BY THE STEEL FRAME. FLOORS AND ROOFS SHALL BE OF POURED-IN-PLACE REINFORCED CONCRETE OR METAL, OR ANY COMBINATION THEREOF, EXCEPT THAT ROOFS ON BUILDINGS OVER THREE STORIES MAY BE OF ANY MATERIAL. EXTERIOR AND INTERIOR WALLS MAY BE OF ANY NON-LOAD CARRYING MATERIAL. <u>CLASS III-C:</u> BUILDINGS HAVING SOME OF THE FAVORABLE CHARACTERISTICS OF CLASS III-A BUT OTHERWISE FALLING INTO CLASS III-B. <u>CLASS III-D:</u> BUILDINGS HAVING A COMPLETE STEEL FRAME WITH FLOORS AND ROOFS OF ANY MATERIAL AND WITH WALLS OF ANY NON-LOAD BEARING MATERIALS.
CLASS IV: REINFORCED CONCRETE, COMBINED REINFORCED CONCRETE AND STRUCTURAL STEEL FRAME
<u>NOTE:</u> CLASS IV-A, B, AND C BUILDINGS SHALL HAVE ALL VERTICAL LOADS CARRIED BY A STRUCTURAL SYSTEM CONSISTING OF ONE OR A COMBINATION OF THE FOLLOWING: (a) POURED-IN-PLACE REINFORCED CONCRETE FRAME, (b) POURED-IN-PLACE REINFORCED CONCRETE BEARING WALLS, (c) PARTIAL STRUCTURAL STEEL FRAME WITH (a) AND/OR (b). FLOORS AND ROOF SHALL BE OF POURED-IN-PLACE REINFORCED CONCRETE, EXCEPT THAT MATERIALS OTHER THAN REINFORCED CONCRETE MAY BE USED FOR THE ROOFS ON BUILDINGS OVER 3 STORIES. <u>CLASS IV-A:</u> BUILDING HAVING A STRUCTURAL SYSTEM AS DEFINED BY THE NOTE (ABOVE) WITH POURED-IN-PLACE REINFORCED CONCRETE EXTERIOR WALLS OR REINFORCED UNIT MASONRY EXTERIOR WALLS PLACED WITHIN THE FRAME. BUILDINGS SHALL HAVE A LEAST WIDTH TO HEIGHT ABOVE GROUND (OR ABOVE ANY SETBACK) RATIO OF NOT EXCEEDING ONE TO THREE. NOT QUALIFYING ARE BUILDINGS HAVING COLUMN-FREE AREAS GREATER THAN 2,500 SQ. FT. (SUCH AS AUDITORIUMS, THEATERS, PUBLIC HALLS, ETC.).

Table C-4. Percent Losses at Given Modified Mercalli Intensities for Various Construction Classes (Based Upon Algermissen, Steinbrugge and Lagorio, 1978, p. 66)

INTENSITY	CONSTRUCTION CLASS									
	V-E	IV-D	IV-E	IV-B	V-D	III-B,III-D IV-C,V-C	III-C,IV-E V-B	III-A	II-B	II-A
X	50%	42%	37%	33%	30%	23%	18%	15%	12%	8%
IX	35%	30%	27.5%	25%	22.5%	17.5%	13%	11%	8%	7%
VIII	25%	22%	19%	18%	16%	12.5%	7.5%	6%	4.5%	4%
VII	14.5%	12.5%	11%	10%	9%	7%	2%	1.5%	1%	2.5%
VI	4%	3%	2.5%	2.5%	2.5%	2%	0	0	0	0

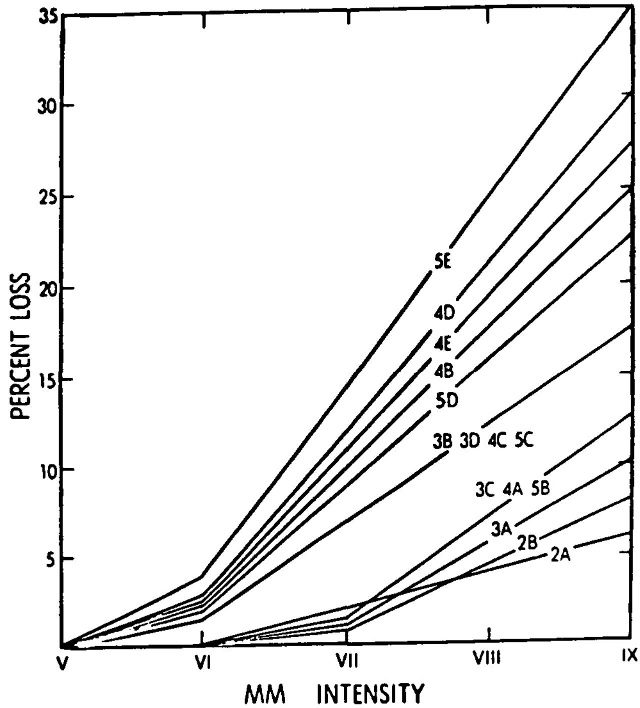


Figure C-2. Modified Mercalli Intensity - Loss Relationship (by Class of Construction). Descriptions of the Various Classes May be Found in Appendix Table C-3 (Source: Algermissen, Steinbrugge, Lagorio, p. 66. 1978.)

Table C-5. Percent Losses at Given Modified Mercalli Intensities for Mobile Homes and Wood Frame Dwellings (Source: Steinbrugge and Schader, 1979, p. 16)

MM INTENSITY	CONSTRUCTION CLASS		
	MODERN WOOD MOBILE HOMES	OLDER WOOD FRAME DWELLINGS	FRAME DWELLINGS
IX	18%	12%	9%
VIII	12%	8%	6%
VII	5%	2.5%	2.5%
VI	2.5%	0.8%	0.8%
V	1%	0%	0%

C.1.2.4 Wiggins et al. Techniques

Distinguishing between commercial-industrial and residential structures, Wiggins et al. rate buildings from 1 to 4, with Q=1 structures having comparatively the least resistance. Roughly speaking, Q=1 is equivalent to a UBC 0-1 structure (see previous discussion of Whitman), Q=2 is equivalent to a UBC 2 structure, Q=3 is equivalent to a UBC seismic zone 3 structure, and Q=4 is equivalent to a UBC 4 structure. In addition, a quality factor 5 might be used for carpentered one- and two-story wooden structures with good wood siding. Carpentered structures are built to specifications and not formulas and have been found to be about four times as strong as typical seismic zone 3 (U.S.) design loads would allow. (J.H. Wiggins, Effects of Sonic Boom.) Further, wood that is well-nailed, bolted or otherwise jointed has excellent ductility and damping characteristics.

Quality ratings also can be assigned based on five structural parameters:

- strength
- physical condition
- integrity
- workmanship
- drift-to-yield and ductility to failure

These five factors will be discussed in greater detail in Appendix CII.

Loss estimates for these various categories are indicated in Figures C-3a and C-3b, and are tabulated in Table C-6.

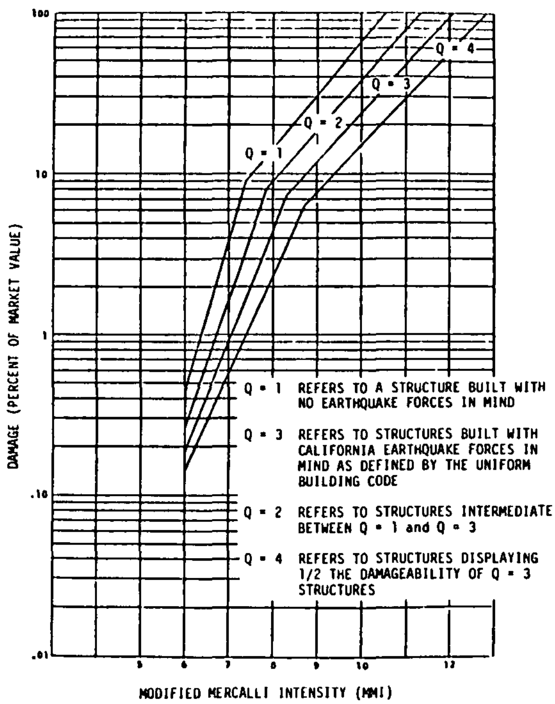


Figure C-3a. Damage Algorithms for Residential Commercial Construction

Table C-6. Mean Damage Ratios (As Percents of Replacement Costs) by Wiggins' Categorization Techniques

(MSK OR) MM INTENSITY	COMMERCIAL-INDUSTRIAL CATEGORIES				RESIDENTIAL CATEGORIES			
	Q=1	Q=2	Q=3	Q=4	Q=1	Q=2	Q=3	Q=4
VI	.13%	.06%	.06%	.05%	.44%	.27%	.19%	.14%
VII	4.5%	.89%	.55%	.33%	3.9%	1.7%	.95%	.58%
VIII	12%	5.6%	3.1%	2.0%	14%	9.0%	4.6%	2.4%
IX	37%	14%	6.7%	4.0%	30%	18%	12%	7.8%
X	100%	33%	14%	7.8%	65%	38%	23%	15%
XI	100%	81%	31%	15%	100%	78%	47%	29%
XII	100%	100%	67%	30%	100%	100%	93%	57%

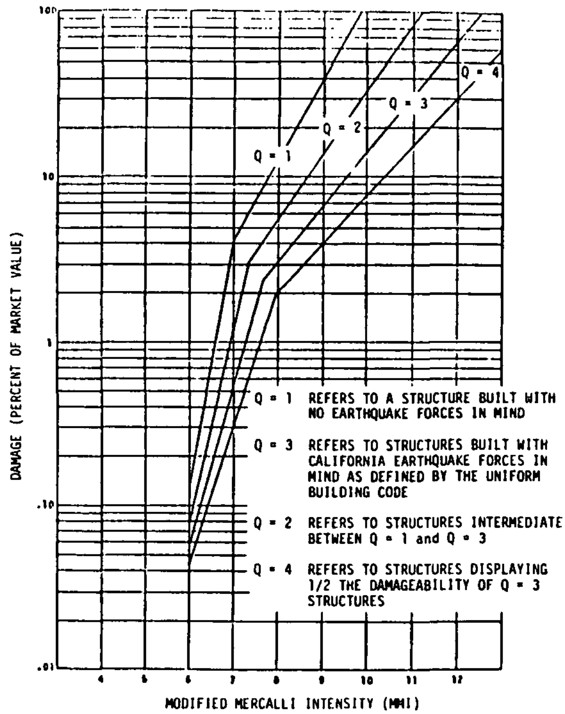


Figure C-3b. Damage Algorithms for Industrial Construction

C.1.2.5 Techniques for Romanian Structures

Data have been developed from a recent Romanian earthquake (4 March 1977) that strongly suggest that use of the foregoing techniques may grossly underestimate expected losses for selected classes of structures in some regions.

Categories developed for damage evaluation and vulnerability assessment are found in Table C-7. "A" categories pertain to surveys made in Bucharest; "B" categories pertain to surveys made in Jassy. Figures C-4(a), -4(b), and -4(c) indicate mean damage for such categories of structures at given intensities. Values are tabulated in Table C-8. In addition, one-sigma differences were calculated, and those are found in Table C-9.

On the assumption that the MSK scale is equivalent to the MM Intensity Scale, these findings indicate that at lower intensities various classes of Romanian structures are much more susceptible to damage than would be indicated through use of the techniques developed by Shah et al., Whitman et al., Steinbrugge et al., and Wiggins et al. Romanian structures may be more vulnerable at lower intensities.

Those differences are especially important for loss estimation purposes. For overall loss estimates, differences in vulnerability estimates at lower intensities are much more significant than differences at higher intensities for the simple reason that the expected frequency of lower intensities is far greater than that of higher intensities. As a rule of thumb, in a high-

Table C-7. Classification of Romanian Structures
(Source: H. Sandi and D. Vasilescu) (Page 1 of 2)

A1. BUILDINGS WITH POOR QUALITY MATERIAL
RELATIVELY OLD (MOST BEFORE 1900). POOR QUALITY MATERIALS (ADOBE, WOOD WITH EARTH INFILL, ETC.). TYPICALLY ONE-STORY, WITH TIMBER FLOORS. NOT ENGINEERED.

A2. OLD BUILDINGS WITH MASONRY BEARING WALLS AND FLEXIBLE FLOORS (UP TO 1940)
TYPICALLY ONE- TO TWO-STORIES, BUT UP TO FIVE-STORIES IN RARE CASES. TYPICALLY IRREGULAR IN CONFIGURATION. VARIABLE QUALITY OF MASONRY. OFTEN POOR FOUNDATIONS WHEN BASEMENT PRESENT.

A3. NEW BUILDINGS WITH MASONRY BEARING WALLS AND FLEXIBLE FLOORS (POST 1940)

A4. OLD BUILDINGS WITH MASONRY BEARING WALLS AND RIGID FLOORS (UP TO 1940)
SAME AS A2, A3, EXCEPT FOR FLOORS. PRESENCE OF REINFORCED CONCRETE SLABS, MOSTLY GIRDERS, AND REINFORCED CONCRETE LINTELS, SOMETIMES CAST TOGETHER WITH FLOORS.

A5. NEW BUILDINGS WITH MASONRY BEARING WALLS AND RIGID FLOORS (POST 1940)
WITH SOME SEISMIC RESISTANCE. OFTEN PREFABRICATED FLOORS (AS A RULE, STRIPES, BUT EARLIER, BEAMS WITH INFILL PIECES). AFTER 1960, REINFORCED CONCRETE MEMBERS WERE USED TO IMPROVE PERFORMANCE OF BEARING MASONRY. PREFABRICATED FLOORS SOMETIMES WERE NOT TRULY RIGID DIAPHRAGMS.

A6. BUILDINGS WITH REINFORCED CONCRETE FRAME
HEIGHT BETWEEN SIX AND TWELVE STORIES. UP TO 1940, AND PRACTICALLY UP TO 1950, NO LATERAL RESISTANCE IN DESIGN. THEN, VERTICAL BEARING MEMBERS WERE OFTEN NOT CONTINUOUS OVER THE ENTIRE HEIGHT OF THE BUILDING. NO CONCERN FOR MOMENT-RESISTING NODES. REINFORCED CONCRETE STRUCTURE TRANSMITTED PART OF LATERAL LOAD TO THE GROUND AND ALSO CONFINED INFILL MASONRY, AN ESSENTIAL ELEMENT FOR LATERAL LOAD RESISTING CAPACITY. EARTHQUAKE-RESISTANCE WAS DESIGNED AFTER 1950. THOSE SO DESIGNED PERFORMED BETTER IN THE 1977 EARTHQUAKE.

A7. HIGHRISE BUILDINGS WITH REINFORCED CONCRETE BEARING WALLS AND SMALL INTERVALS
IN THE EARLIER STAGE, SLIDING FORMS WERE USED. ALSO STEEL OR TIMBER DISMOUNTABLE FORMS HAVE BEEN USED. ONE OR TWO LONGITUDINAL INTERNAL BEARING WALLS. TRANSVERSE WALLS WERE SPACED 3-4M APART.

A8. HIGHRISE BUILDINGS WITH REINFORCED CONCRETE BEARING WALLS AT LARGER INTERVALS
LARGER INTERVALS ALLOWED MORE ARCHITECTURAL FREEDOM IN SHAPING APARTMENTS.

Table C-7. Classification of Romanian Structures
(Source: H. Sandi and D. Vasilescu) (Page 2 of 2)

B1. OLD BUILDINGS
TYPICALLY WITHOUT ENGINEERING DESIGN. FOUNDATION ON STONE (OLDER) OR IN CONCRETE (MORE RECENT). FLOORS OF STEEL BEAMS AND MASONRY ELEMENTS, OR WOODEN ELEMENTS. UP TO FOUR STORIES. MOST PREVIOUSLY AFFECTED BY EARTHQUAKES, FLOOD, FIRE, ETC. TYPICALLY SURVEYED WERE ONE- OR TWO-STORY RESIDENTIAL BUILDINGS OF BRICK MASONRY. TYPICALLY WOOD FLOORS AND STONE FOUNDATIONS.

B2. NEW MASONRY BUILDINGS WITH CAST-IN-PLACE REINFORCED CONCRETE FLOORS
CONSTRUCTED 1950-1970. HEIGHT UP TO FIVE STORIES (TYPICAL). LONGITUDINAL AND TRANSVERSE WALLS ARE COMPARATIVELY STIFF. REINFORCED GIRDERS BUILT AT ALL FLOORS, INCLUDING FOUNDATION LEVEL.

B3. NEW MASONRY BUILDINGS WITH PRECAST FLOORS
TRANSVERSE WALLS AND UNIQUE MEDIAN LONGITUDINAL WALLS INTERRUPTED BY THE STAIRCASE. FLOORS OF PRECAST STRIPES SUPPORTED BY TRANSVERSE WALLS. CONCRETE FOUNDATIONS WITH REINFORCED CONCRETE GIRDERS. FIVE STORIES HIGH.

B4. LARGE PANEL STRUCTURES
FIVE STORIES HIGH. TRANSVERSE WALLS CONFINING ROOMS AND STAIRCASES. COUPLES OF LONGITUDINAL MEDIAN WALLS WITHOUT OPENINGS, CONFINING THE STAIRCASE.

B5. BUILDINGS WITH FRAMED STRUCTURES AND WITH COMPOSITE STRUCTURE (FRAMES AND BEARING WALLS)
SEVERAL TYPES: CAST-IN-PLACE COMMON FRAMED STRUCTURES WITH FLOORS CAST-IN-PLACE OR OF PRECAST PANELS; LAMELLAR FRAMED STRUCTURES WITH CAST-IN-PLACE OR PRECAST BEAMS AND WITH FLOORS OF PRECAST PANELS; LAMELLAR FRAMED STRUCTURES COMPOSITE WITH SOME SHEAR WALLS. INFILL AND SEPARATION WALLS OF BRICK MASONRY OR LIGHT WEIGHT CONCRETE. RIGID BASEMENT A FOUNDATIONMENT. SYMMETRIC LAYOUT.

B6. REINFORCED CONCRETE SHEAR WALL BUILDINGS
EIGHT TO ELEVEN STORIES. FLOORS CAST-IN-PLACE EXCEPT FOR USE OF STEEL FORMS, WHERE FLOORS ARE PRECAST. FOUNDATIONS DESIGNED AS GENERAL MATS; HENCE A RIGID BASEMENT BOX. LAYOUT WEAKNESSES IN GEOMETRY, STIFFNESS DISTRIBUTION, OR CONNECTIONS BETWEEN STRUCTURAL MEMBERS.

Table C-8. Mean Values of Damage Degree at Given MSK Intensities for Romanian Structures (Source: Sandi and Vasilescu)

BUILDING CATEGORY	MSK INTENSITY						
	VI	VI.5	VII	VII.5	VIII	VIII.5	IX
A1	-	25%	29.9%	37.6%	41.2%	-	-
A2	-	21.7%	33.4%	36.4%	42.4%	60.6%	-
A3	-	20.7%	24.2%	28.1%	38.6%	-	-
A4	-	22.1%	25.6%	26.3%	34.9%	-	-
A5	-	22.7%	25.7%	29.3%	37.4%	-	-
A6	-	-	25.3%	26.1%	30.3%	42.8%	-
A7	-	32.6%	35.8%	42.6%	57.2%	-	-
A8	-	35.9%	40.6%	42.9%	49.3%	-	-
B1	-	28.0%	-	55.0%	-	-	-
B2	-	8.8%	-	17.6%	-	-	-
B3	-	34.7%	-	43.6%	-	-	-
B4	-	49.4%	-	52.6%	-	-	-
B5	-	30.0%	-	37.1%	-	-	-
B6	-	34.7%	-	40.0%	-	-	-

Table C-9. One-Sigma Differences (As Percents of Replacement Costs) for Table C-8*

BUILDING CATEGORY	MSK INTENSITY						
	VI	VI.5	VII	VII.5	VIII	VIII.5	IX
A1	-	14.3%	16.9%	18.4%	16.4%	-	-
A2	-	8.5%	16.4%	12.5%	12.3%	19.6%	-
A3	-	7.5%	10.0%	14.1%	21.4%	-	-
A4	-	13.8%	11.3%	12.4%	11.9%	-	-
A5	-	12.8%	19.2%	15.2%	20.4%	-	-
A6	-	-	16.1%	14.0%	17.2%	24.2%	-
A7	-	13.4%	15.8%	16.3%	16.4%	-	-
A8	-	15.8%	14.9%	15.6%	16.8%	-	-
B1	-	21.0%	-	25.3%	-	-	-
B2	-	13.7%	-	16.6%	-	-	-
B3	-	25.0%	-	24.5%	-	-	-
B4	-	18.3%	-	37.1%	-	-	-
B5	-	22.4%	-	21.4%	-	-	-
B6	-	22.8%	-	24.5%	-	-	-

*Add (or substract) these from Table C-8 in order to get one -sigma$^{+(\text{or }-)}$ of mean.

Table C-10. Classification of Conventional Turkish Buildings (Source: Akkas and Erdik, 1982, p.8)

CATEGORY	TYPE	LOCATION	DESCRIPTION	SEISMIC CHARACTERISTICS
TIMBER-FRAMED STRUCTURES	RURAL "HIMIS"	MAINLY WESTERN TURKEY	POORLY JOINTED TIMBER FRAMES FILLED FOR INSULATION WITH STONE AND ADOBE, HEAVY ROOFS.	POOR DAMPING QUALITY AND WEAK LATERAL RESISTANCE, SPECIALLY WITH HEAVY ROOFS.
	URBAN "BAGDADI"	MAINLY WESTERN TURKEY	CLOSELY SPACED LATHS ARE NAILED TO THE VERTICAL MEMBERS. THE INFILL IS LIGHT, AND THE SURFACE PLASTERED.	THE DUCTILITY AND DAMPING CAPACITY ARE INCREASED, AND THE LATHS PREVENT SPALLING OF THE INFILL.
MASONRY STRUCTURES	ADOBE "KERPIC"	MIDDLE ANATOLIAN PLATEAU	REGULAR SHAPED WALLS ARE MADE WITH HEAVY FLAT ROOFS. CONTINUOUS TIMBER LINTELS ADDED FOR STRENGTH.	THE EARTHQUAKE RESISTANCE IS VERY POOR.
	STONE	EASTERN ANATOLIO	A WIDE VARIETY OF STONE HOUSES EXIST, FROM RUBBLE MASONRY TO REFINED URBAN MORTARED MASONRY.	PRIMITIVE STONE MASONRY IS WEAKEST IN LATERAL RESISTANCE. MORTAR ADDS STRENGTH.
	BRICK	ALL URBAN AREAS	A POPULAR TYPE OF CONSTRUCTION BECAUSE OF LIGHTNESS AND ECONOMY, AND PLEASANT APPEARANCE.	LESS DAMAGE IS CAUSED WITH STONE BUILDINGS. FAILURE IS NON-DUCTILE.
	HOLLOW BRICK	URBAN ZONES	USED IN THE SAME WAY AS BRICK, CONTINUOUS CONCRETE LINTELS ARE OFTEN ADDED FOR STRENGTH.	HIGHER LATERAL RESISTANCE THAN BRICK, FAILURE IS NON-DUCTILE.
	MIXED	ALL URBAN	UNDER-REINFORCED CONCRETE COLUMNS AT CORNERS WITH BRICK WALLS AND ALSO CONTINUOUS LINTELS.	THE OVERALL LATERAL RESISTANCE MAY PARTLY INCREASE, BUT THE DIFFERENT MATERIALS ARE EASILY DISUNITED, CAUSING HAZARD.
REINFORCED CONCRETE		ALL AREAS	CONVENTIONAL REINFORCED CONCRETE FRAMED STRUCTURES ARE MADE.	DEPENDING ON THE DESIGN, WORKMANSHIP EARTHQUAKE AND SOIL CONDITIONS, THE RESULTS RANGE FROM POOR TO VERY GOOD.

Table C-11. Estimated Deaths as a Percent of Occupants Within Given Buildings Affected at Given Modified Mercalli Intensities (Source: Background Data for U.S.G.S., 1976)

INTENSITY	CONSTRUCTION TYPE					
	UBC* 3, 1976 AND AFTER	UBC 2 1 STORY	UBC 0 OR UBC 1 1 STORY	UBC 2 2 OR MORE STORIES	UBC 0 OR UBC 1 2 OR MORE STORIES	STRUCTURES WITHIN FAULT ZONE OF DEFORMATION
VIII	.002	.005	.0067	.008	.01	.013
IX	.006	.015	.02	.025	.03	.04
X	.009	.023	.03	.038	.045	.06

*UBC = Uniform Building Code (U.S.).

ly seismic region, expected frequencies increase by a factor of three for each decreasing intensity increment. In less seismic regions, this factor can be five or more. For single scenario earthquake events, even large ones, the area covered at lower intensities is expected to be much more extensive than the area covered at high events. Magnitude and proximity of buildings to the epicenter will make a considerable difference in how use of diverse loss estimates affect outcomes for single scenarios.

Thus, the engineer, planner, or policy maker who is using any of the foregoing categorization techniques must be aware of underestimating losses for structures with little seismic resistance.

C.1.2.6 Taxonomy for Turkish Buildings

Local types of structures may need to be described in some detail so that loss estimation data can be developed. For Turkish structures, that classification is found in Table C-10. At present, no vulnerability estimates are associated with those various classes.

C.1.3 Two Life-Loss Estimation Techniques as a Function of Building Type

Both sets of estimates here pertain directly to the structures classified by U.S. Uniform Building Code seismic standards. Hence, the engineer must make translations for building types in other countries. The first estimates are derived from U.S.G.S. (United States Geological Survey) background data and are summarized in Table C-11. The second come from work by Whitman et al. and are summarized in Table C-12. Estimates can be adjusted (upwards) for hospital patients and for others who may not be so capable of coping with building damage.

Many more casualties might be expected than deaths, but the existing procedures for estimating casualties are not described here.

Table C-12. Mean Life Loss as a Percent of Occupants by UBC Seismic Design and Modified Mercalli Intensity (Whitman & Cornell, 1976, p. 354)

	CONSTRUCTION CLASS			
INTENSITY	UBC 0-1	UBC 2	UBC 3	ZONE S (UBC 4)
VII	0.0001	0.000025	0	0
VIII	0.0144	0.0033	0.0006	0.0002
IX	0.058	0.048	0.014	0.0016
X	0.153	0.086	0.048	0.029

C.1.3.1 Life Loss as a Function of Collapses

Nejat Bayulke and Nuri Akkas argue for using death rate as a function of collapses. They have used such rates as 0.38 deaths per collapse in Bolu (p. 10) and 0.60 deaths per collapse for Gediz (p. 13).

A more generalizable method is developed by S.A. Anagnostopoulos and R.V. Whitman, who estimate deaths as a function of percents of occupants and degree of damage.

As indicated earlier, probability distributions for damage can still become much better defined, and there are several possible probability distribution functions which could be applied to building collapse and life loss. Exploring several possible probability distribution functions, Anagnostopoulous and Whitman develop a table (Table C-13) of expected values for various degrees of damage to non-reinforced masonry structures affected at various intensities. They argue that, where the probability of collapse is not zero, then the collapse state outweighs all other states in terms of estimating potential deaths. Clearly, though, until existing techniques are refined through better data, the risk analyst must use caution in estimating potential life loss from seismic structural damage.

Table C-13. Expected Number of Deaths Per 100,000 People in Non-reinforced Masonry Structures at Various Intensities (Example Study - Anagnostopoulos and Whitman)

DAMAGE STATE	INTENSITY (MSK OR MMI)				
	VI	VII	VIII	IX	X
MODERATE	0	9	6	1	0
HIGH	0	8	100	80	10
TOTAL	0	18	72	900	810
COLLAPSE	0	0	200	1000	10,000
SUM	0	35	378	1981	10,820

C.1.4 Content Losses

For U.S. buildings, content losses may on the average be estimated at present at 32% of building losses. However, content value may vary as a percent of building value, and that percent may be used to estimate content losses.

C.2 DISCUSSION OF VISUAL INSPECTION AND RATING TECHNIQUES

C.2.1 The Field Evaluation Method (Culver et al.)

This method involves deriving an overall capacity ratio rating for the structure based on the following features:

- Seismicity (intensity rating)
- Quantity of resisting elements
- Symmetry of resisting elements
- Present condition of resisting elements
- General rating of structure based on frame system
- Rigidity features
- Anchorage and connections
- Chords
- Other life hazards (partitions, glass, ceiling, light, exterior and interior appendages and wall cladding).

The general formula for calculation of the capacity ratio rating is as follows:

$$\text{Capacity ratio rating (CR)} = \frac{1}{3}\,(GR+2SR_m)/IR \qquad \text{(C-1)}$$

wherein

$$SR_m = \text{max.}\ \frac{S+Q+4PC}{6},\ A, R, C$$

and

GR = the general structure rating,
S = the symmetry rating,
Q = the rating of the quantity of resisting elements,
PC = the rating of the present condition of resisting elements,
A = the anchorage rating,
C = the chords rating,
R = the rigidity rating, and
IR = the intensity rating.

When CR < 1, the building is rated "good," when 1. < CR < 1.4, the building is rated "fair," when 1.4 < CR < 2.0, the building is rated "poor," and when 2.0 < CR, the building is rated very poor.

Tables C-14 through C-22 provide general descriptions whereby structures can be evaluated in accordance with formula (C-1). Forms for making field notes and assessments are found in Figure C-5. Such forms also include qualitative ratings to assess "other life hazards."

Table C-14. Seismic Ratings of Various Elements (Field Evaluation Method)

TYPE OF RESISTING ELEMENT	GENERAL SEISMIC RATING (GR)
A. STEEL MOMENT RESISTANT FRAMES	1
B. STEEL FRAMES - MOMENT RESISTANCE CAPABILITY UNKNOWN	2
C. REINFORCED CONCRETE MOMENT RESISTANT FRAMES	1
D. REINFORCED CONCRETE FRAMES - MOMENT RESISTANCE CAPABILITY UNKNOWN	2
E. MASONRY SHEAR WALLS - UNREINFORCED	4
F. MASONRY OR CONCRETE SHEAR WALLS - REINFORCED	1
G. COMBINATION - UNREINFORCED SHEAR WALLS AND MOMENT RESISTANT FRAMES	2
H. COMBINATION - REINFORCED SHEAR WALLS AND MOMENT RESISTANT FRAMES	1
J. BRACED FRAMES	1
K. WOOD FRAME BUILDINGS, WALLS SHEATHED OR PLASTERED	1 OR 2
L. WOOD FRAME BUILDINGS, WALLS WITHOUT WOOD SHEATHING OR PLASTER	4

Table C-15. Seismic Rating of Symmetry of Elements (Field Evaluation Method)

SYMMETRY OF RESISTING ELEMENTS	SEISMIC RATING (S)
SYMMETRICAL	1
FAIRLY SYMMETRICAL	2
SYMMETRY POOR	2 OR 3
VERY UNSYMMETRICAL	3 OR 4

NOTE: - Add 1 (but rating not to exceed 4) to each rating if a high degree of vertical non-uniformity in stiffness occurs.

Table C-16. Seismic Rating of Quantity of Elements (Field Evaluation Method)

QUANTITY OF RESISTING ELEMENTS	SEISMIC RATING (Q)
MANY RESISTING ELEMENTS	1
MEDIUM AMOUNT OF RESISTING ELEMENTS	2
FEW RESISTING ELEMENTS	3
VERY FEW RESISTING ELEMENTS	4

NOTE: - If exterior shear walls are at least 75% of building length, this rating will be 1.

Table C-17. Seismic Rating for Present Condition of Elements (Field Evaluation Method)

PRESENT CONDITION OF RESISTING ELEMENTS	SEISMIC RATING (PC)
NO CRACKS, NO DAMAGE	1
FEW MINOR CRACKS	2
MANY MINOR CRACKS OR DAMAGE	3
MAJOR CRACKS OR DAMAGE	4

NOTE: - In masonry walls, note quality of mortar - good or poor. If lime mortar is poor, use next higher rating.

Table C-18. Seismic Rating for Various Resisting Elements (Field Evaluation Method)

RIGIDITY	SEISMIC RATING (R)
RIGID	1
SEMI-RIGID	1.5
SEMI-FLEXIBLE	2.0
FLEXIBLE	2.5

Table C-19. Seismic Rating for Anchorage and Connections (Field Evaluation Method)

ANCHORAGE AND CONNECTIONS	SEISMIC RATING (A)
ANCHORAGE CONFIRMED - CAPACITY NOT COMPUTED, BUT PROBABLY ADEQUATE	1
ANCHORAGE CONFIRMED - CAPACITY NOT COMPUTED, BUT PROBABLY INADEQUATE	2
ANCHORAGE UNKNOWN	3
ANCHORAGE ABSENT	4

Table C-20. Seismic Rating for Chords or Ties (Field Evaluation Method)

CHORDS OR TIES	SEISMIC RATING (C)
CHORDS CONFIRMED, BUT CAPACITY NOT COMPUTED	1
CHORDS UNKNOWN, BUT PROBABLY PRESENT	2
CHORDS UNKNOWN, BUT PROBABLY NOT PRESENT	3
CHORDS ABSENT	4

Table C-21. Seismicity Ratings of MM Intensities (Field Evaluation Method)

MM INTENSITY	SEISMICITY RATING (IR)
VIII OR HIGHER	1
VII	2
VI	3
V OR LESS	4

Table C-22. Risk Rating of Buildings (Field Evaluation Method)

RATIOS OF (SR) TO (IR)	RISK RATING
LESS THAN 1.0	GOOD
1.0 TO 1.4	FAIR
1.5 TO 2.0	POOR
MORE THAN 2.0	VERY POOR

C.2.2 Wiggins et al. Rating Method

The Wiggins et al. rating method depends on ratings for the following items:

- framing system and/or walls (vertical load carrying system)
- diaphragm and/or horizontal bracing system
- partitions
- special hazards, including building layout, design, and soil conditions
- physical layout
- importance factor for structure
- severity of seismic environment.

Put into a summary form, the risk safety factor for a given structure is evaluated in accordance with the following equation:[1]

$$\text{Risk Safety Factor } (R_s) = \frac{352\ (DAF)_s \cdot A_r}{180 - (F+D+P+PC+SH)} \qquad \text{(C-2)}$$

wherein

F = rating of framing system and/or walls

D = rating of diaphragms and/or bracing system

P = rating of partitions

PC = rating of present condition

SH = rating of special hazards

$(DAF)_s$ = dynamic amplification factor based on general soils wherein

$$(DAF)_s = \begin{cases} 5 \text{ for "A" earth materials} \\ 4 \text{ for "B" earth materials} \\ 3 \text{ for "C" earth materials} \end{cases}$$

[1] In the equation, the factor 352 is derived from 180/0.44, wherein 0.44 is a function of expected increase in lives lost as A_r increases. If A_r doubles, for instance, the expected risk factor would multiply by 1/0.44, or 2.3. As better data are available, this factor can be determined more accurately.

and A_r = maximum bedrock acceleration (as percent of g) expected during lifetime of the structure.

(1.) Framing System and/or Walls:

The framing system and walls refer to the vertical load carrying system of a structure, whether it consists of independent framing of columns and beams or bearing walls, or a combination of bearing walls and framing. Frames may also be lateral force resisting; walls may also be shear walls.

(2.) Diaphragm and/or Bracing System:

The diaphragm and/or bracing system refers to the horizontal bracing systems or diaphragms, if any, which operate to distribute lateral forces applied against the structure to the vertical resisting elements of the structure.

(3.) Partitions:

Partitions refer to interior walls which may, depending on their number, design, and condition, resist lateral forces applied to the structure and furnish support to the floors and roofs despite the loss of the exterior bearing walls of the structure.

(4.) Special Hazards:

Special hazards refer to deficiencies in a structure's layout, design, soil conditions and/or location with respect to other structures or areas where people might be present during an earthquake. The grading score shall be directly proportional to the life hazard presented by the special characteristics of the building including, but not limited to, building dimensional plan; asymmetrical bracing or shear wall systems; building height, presence of inadequately secured parapets, fire escapes, gargoyles, light fixtures, heavy equipment, and other things which might be dislodged during an earthquake; location of the structure on poor soil and/or unstable soil, adverse geologic conditions, and the presence of lower-height structures, pedestrian mallways or walks, and other areas of human concentration in such relation to the structure as to expose same to the danger of falling debris from the structure.

NOTE: The Building Department shall have the authority to initiate action for the abatement of special hazards when the grading points are 35 or more in this particular category, even though the structure as a whole receives a grade of less than 100.

(5.) Physical Condition:

Physical condition refers to present structural integrity.

The following definitions are used for various categories of earth materials:

Table C-23. Framing Systems and/or Walls

FRAMING SYSTEMS AND/OR WALLS - THE FRAMING SYSTEM AND WALLS REFER TO THE VERTICAL LOAD CARRYING SYSTEM; EITHER INDEPENDENT FRAMING OF COLUMNS AND BEAMS OR BEARING WALLS, OR COMBINATIONS THEREOF. FRAMES MAY BE LATERAL FORCE RESISTING; WALLS MAY BE SHEAR WALLS.

LOW (0-POINTS)	INTERMEDIATE (20-POINTS)	HIGH (40-POINTS)
• REINFORCED CONCRETE AND STEEL VERTICAL AND LATERAL RESISTING FRAMES ADEQUATELY[1] DESIGNED, WITH REINFORCED CONCRETE AND MASONRY FILLER WALLS • REINFORCED CONCRETE[2] AND MASONRY BEARING WALLS[2], THREE STORIES AND LESS • STEEL RIGID FRAMES, ADEQUATELY BRACED • WOOD WITH PLYWOOD AND DIAGONAL SHEATHED WALLS 3 STORIES AND LESS	• REINFORCED CONCRETE[2] AND STEEL VERTICAL LOAD FRAMES • REINFORCED CONCRETE AND MASONRY BEARING WALLS OVER THREE STORIES • WOOD FRAME OVER THREE STORIES • POORLY BRACED STEEL BUILDINGS • UNREINFORCED CONCRETE AND MASONRY FILLER WALLS, GOOD[3] MORTAR	• UNREINFORCED MASONRY FILLER AND BEARING WALLS WITH POOR[4] QUALITY MORTAR

1"Adequately" means conforming to current code.

2Reinforcing of concrete and masonry must comply with current code to be placed in "low" or "intermediate" categories.

3Core test of masonry must be 750 psi or greater to qualify.

4Core test of masonry is below 750 psi.

GRADE________POINTS

Table C-24. Diaphragm and/or Bracing System

DEGREE OF POTENTIAL LIFE RISK

DIAPHRAGM AND/OR BRACING SYSTEM - THIS REFERS TO HORIZONTAL BRACING SYSTEMS OR DIAPHRAGMS WHICH SHOULD DISTRIBUTE THE LATERAL FORCES TO VERTICAL RESISTING ELEMENTS

LOW (0-POINTS)	INTERMEDIATE (10-POINTS)	HIGH (20-POINTS)
• WELL ANCHORED, REINFORCED CONCRETE SLABS AND FILLS, ADEQUATELY AS DIAPHRAGMS • WELL ANCHORED, CONTINUOUS DOUBLE SHEET METAL DECKING • WELL ANCHORED, BLOCKED PLYWOOD • WELL ANCHORED, STEEL BRACING SYSTEMS OTHER THAN RODS	• SINGLE SHEET METAL DECKING LIGHTWEIGHT FILLS, UNBLOCKED PLYWOOD, DIAGONAL SHEATHING AND GYPSUM, ALL ANCHORED AND ADEQUATE AS DIAPHRAGMS • PRECAST CONCRETE UNITS WITHOUT FILL, ANCHORED AND ADEQUATE AS DIAPHRAGMS • DIAGONAL ROD BRACING SYSTEM ANCHORED AND ADEQUATE DESIGN	• STRAIGHT OR DIAGONAL WOOD SHEATHING WITHOUT ADEQUATE CONNECTIONS TO WALLS AND NO SPECIAL NAILING • INCOMPLETE OR INADEQUATE BRACING SYSTEMS

GRADE________POINTS

Table C-25. Partitions

PARTITIONS - REFER TO INTERMEDIATE, NON LOAD BEARING WALLS, WHICH MAY, DEPENDING ON THEIR NUMBER, DESIGN, AND CONDITION, RESIST LATERAL FORCES APPLIED TO THE STRUCTURE AND FURNISH SUPPORT TO THE FLOORS AND ROOF DESPITE THE LOSS OF EXTERIOR BEARING WALLS OF THE STRUCTURE.

LOW (0-POINTS)	INTERMEDIATE (10-POINTS)	HIGH (20-POINTS)
• MANY[5] WOOD OR METAL STUD BEARING • MANY[5] REINFORCED MASONRY BEARING • MOVEABLE METAL OR GYPSUM BOARD • FEW[5] REINFORCED MASONRY, NON-BEARING • FEW[5] UNREINFORCED MASONRY, GOOD[3] QUALITY MORTAR AND WELL ANCHORED	• FEW[5] UNREINFORCED MASONRY, POOR MORTAR • MODERATE AMOUNT OF WOOD AND STEEL, BEARING TYPE	• MANY[5] UNREINFORCED MASONRY, POOR[4] MORTAR, AND FEW STEEL OR WOOD STUD: BEARING OR NON-BEARING

[5]"Many" and "few" are relative terms. Typical, small offices and apartments would be considered to have "many"; warehouses and garages to ordinarily have "few".

GRADE__________POINTS

Table C-26. Physical Condition

PHYSICAL CONDITION - PHYSICAL CONDITION IS AN INDICATION OF QUALITY OF DESIGN AND/OR CONSTRUCTION, AS WELL AS PAST DAMAGE HISTORY. POOR PHYSICAL CONDITION INDICATES THAT MEMBERS ARE HIGHLY STRESSED, AND THAT ADDITIONAL STRESSES IMPOSED BY AN EARTHQUAKE WILL CAUSE GREATER DAMAGE.

LOW (0-5-10 POINTS)	INTERMEDIATE (10-15-20 POINTS)	HIGH (20-35-50 POINTS)
• MINOR[8] CRACKS	• MODERATE[9] CRACKING AND MINOR SETTLEMENT CRACKS • MINOR UNREPAIRED EARTHQUAKE DAMAGE	• SERIOUS[10] CRACKING, BOWING OR LEANING OF WALLS • SERIOUS[10] SETTLEMENT OR CRACKING • SIGNS OF INCIPIENT OR ACTUAL STRUCTURAL FAILURE • SERIOUS[10] DETERIORATION OF STRUCTURAL MATERIALS • SERIOUS[10] UNREPAIRED EARTHQUAKE DAMAGE

[8]Minor means few small cracks, generally less than 1/16" across.

[9]Moderate means many cracks 1/16" to 1/8" across.

[10]Serious means many cracks over 1/8" across, settlements more than normal design limits. Serious bowing or leaning more than 1/2 wall thickness.

GRADE_________POINTS

Table C-27. Special Hazards

SPECIAL HAZARDS - THESE INCLUDE DEFICIENCIES IN BUILDING LAYOUT, DESIGN AND SOIL CONDITIONS WHICH CAN CONTRIBUTE SIGNIFICANTLY TO THE LIFE HAZARD. LOWER RANGECHARGES ARE INDICATED WHERE DESIGNS ARE WELL EXECUTED, DETAILED AND CONSTRUCTED. THE GRADING SCORE SHALL BE DIRECTLY PROPORTIONAL TO THE LIFE HAZARD PRESENTED BY THE SPECIAL CHARACTERISTICS OF THE BUILDING INCLUDING, BUT NOT LIMITED TO BUILDING DIMENSIONAL PLAN, ASYMMETRICAL BRACING OR SHEAR WALL SYSTEMS, BUILDING HEIGHT, PRESENCE OF INADEQUATELY SECURED PARAPETS, FIRE ESCAPES, GARGOYLES, LIGHT FIXTURES, HEAVY EQUIPMENT, LOCATION OF STRUCTURE ON POOR SOIL OR UNSTABLE SOIL, ADVERSE GEOLOGIC CONDITIONS, AND EXPOSURE OF OTHER STRUCTURED OR PEOPLE TO POSSIBLE FALLING DEBRIS FROM THE STRUCTURE.

LOW (0-5-10 POINTS)	INTERMEDIATE (10-15-20 POINTS)	HIGH (20-35-50 POINTS)
• NONE, EXCEPT VERY MINOR AMOUNTS	• POOR DIMENSIONAL PLAN, "L" AND "T" SHAPES, (NON-RECTANGULAR) • LACK OF SYMMETRICAL BRACING OR SHEAR WALLS • EXCESSIVE LENGTH TO WIDTH AND HEIGHT RATIOS (GREATER THAN 4:1) • QUESTIONABLE[6] SOIL CONDITIONS WHICH COULD RESULT IN SETTLEMENT OR AMPLIFIED GROUND MOTION IN AN EARTHQUAKE • POORLY ANCHORED CHANDELIERS AND LIGHT FIXTURES	• EXCESSIVE[7] WALL OPENING AND WALL HEIGHTS WITHOUT ADEQUATE[1] DESIGN • OVERHANGING ADJACENT UNREINFORCED MASONRY FILLER OR BEARING WALLS, POOR[4] QUALITY MORTAR • POOR SOIL CONDITIONS, UNCOMPACTED & SATURATED FILLS. UNSTABLE SIDEHILL CONDITIONS • INADEQUATELY[1] ANCHORED ROOF TANKS OR SIGNS • UNREINFORCED MASONRY CHIMNEYS, POOR[4] QUALITY MORTAR • INADEQUATELY[1] ANCHORED ORNAMENTATION AND VENEER ABOVE 1ST STORY • UNBRACED WOOD CRIPPLE STUDY WALLS IN TYPE V • NON-BEARING, UNREINFORCED MASONRY WALLS, PARAPET WALLS OR APPENDAGES

[6] Generally, expansive or relatively loose, silty or clayish soils. Refer to soils factor in Section 2904 of 1970 UBC code.

[7] Excessive wall openings include percentages of 50 and over for any one wall in any story. Excessive wall heights include height to thickness ratios of 30 or more.

GRADE________POINTS

Table C-28. A Scale of Acceptable Risks* (Page 1 of 3)

LEVEL OF ACCEPTABLE RISK (STRUCTURE IMPORTANCE)	KINDS OF STRUCTURES
EXTREMELY LOW (FAILURE OF A SINGLE STRUCTURE MAY AFFECT SUBSTANTIAL POPULATIONS) STRUCTURE IMPORTANCE = 1	STRUCTURES WHOSE CONTINUED FUNCTIONING IS CRITICAL, OR WHOSE FAILURE MIGHT BE CATASTROPHIC: NUCLEAR REACTORS LARGE DAMS POWER INERTIE SYSTEMS PLANTS MANUFACTURING OR STORING EXPLOSIVE OR TOXIC MATERIALS
SLIGHTLY HIGHER THAN UNDER LEVEL 1 (FAILURE OF A SINGLE STRUCTURE MAY AFFECT SUBSTANTIAL POPULATIONS) STRUCTURE IMPORTANCE = 2	STRUCTURES WHOSE USE IS CRITICALLY NEEDED AFTER A DISASTER: IMPORTANT UTILITY CENTERS HOSPITALS WITH EMERGENCY FACILITIES FIRE, POLICE, AND EMERGENCY COMMUNICATIONS FACILITIES FIRE STATIONS CRITICAL TRANSPORTATION ELEMENTS SUCH AS BRIDGES AND OVERPASSES SMALLER DAMS AND RESERVOIRS LARGE OIL AND GAS STORAGE FACILITIES LARGE OIL AND GAS PIPELINES EMERGENCY POWER SYSTEMS MORE THAN 1000 PEOPLE EXPOSED

*Much of this table has been taken from Meeting the Earthquake Challenge, the final report to the legislature, State of California, by the Joint Committee on Seismic Safety, January, 1974.

Table C-28. A Scale of Acceptable Risks* (Page 2 of 3)

LEVEL OF ACCEPTABLE RISK (STRUCTURE IMPORTANCE)	KINDS OF STRUCTURES
LOWEST POSSIBLE RISK TO THE OCCUPANTS OF THE STRUCTURE (FAILURE OF A SINGLE STRUCTURE WOULD AFFECT PRIMARILY ONLY THE OCCUPANTS) STRUCTURE IMPORTANCE = 3	STRUCTURES OF HIGH OCCUPANCY, OR WHOSE USE AFTER A DISASTER WOULD BE PARTICULARLY CONVENIENT: SCHOOLS CHURCHES THEATERS, AUDITORIUMS MULTI-UNIT RESIDENTIAL (>3 STORIES) RAIL TERMINAL FACILITIES CIVIC BUILDINGS DORMATORIES RETAIL BUILDINGS (>3 STORIES) OFFICE BUILDINGS (>3 STORIES) HOTELS, MOTELS (>3 STORIES) LIGHT MANUFACTURING (MULTISTORY) HEAVY MANUFACTURING JAILS, DETENTION HALLS, REFORMATORIES CHEMICAL PROCESSING FACILITIES SEWAGE PROCESSING FACILITIES CLINICS, CONVALESCENT HOSPITALS HOSPITALS WITHOUT EMERGENCY FACILITIES CLUB BUILDINGS, LODGE HALLS AIRPORT CONTROL TOWERS ALTERNATIVE OR NON-CRITICAL BRIDGES AND OVERPASSES SECONDARY UTILITY STRUCTURES A MAXIMUM OF 100 TO 1000 PEOPLE EXPOSED

*Much of this table has been taken from <u>Meeting the Earthquake Challenge</u>, the final report to the legislature, State of California, by the Joint Committee on Seismic Safety, January, 1974.

Table C-28. A Scale of Acceptable Risks* (Page 3 of 3)

LEVEL OF ACCEPTABLE RISK (STRUCTURE IMPORTANCE)	KINDS OF STRUCTURES
AN "ORDINARY" LEVEL OF RISK TO OCCUPANTS OF THE STRUCTURE (FAILURE OF A SINGLE STRUCTURE WOULD AFFECT PRIMARILY ONLY THE OCCUPANTS) STRUCTURE IMPORTANCE = 4	THE VAST MAJORITY OF STRUCTURES: SINGLE FAMILY RESIDENCES MULTI-UNIT RESIDENTIAL (1-3 STORIES) RETAIL BUILDINGS (1-3 STORIES) OFFICE BUILDINGS (1-3 STORIES) HOTELS, MOTELS (1-3 STORIES) LIGHT MANUFACTURING (SINGLE STORY) LIBRARIES WAREHOUSES PARKING FACILITIES (MULTI-STORY) LESS THAN 100 PEOPLE EXPOSED
A RISK WHERE PERSONS ARE PRIMARILY NOT PRESENT STRUCTURE IMPORTANCE = 5	STRUCTURES WHERE THE SOCIAL AND ECONOMIC RISK IS SMALL: PRIVATE GARAGES SHEDS AND AGRICULTURAL STRUCTURES NOT EXCEEDING 1000 SQUARE FEET IN AREA

*Much of this table has been taken from Meeting the Earthquake Challenge, the final report to the legislature, State of California, by the Joint Committee on Seismic Safety, January, 1974.

(1) "A" Earth Materials

Type "A" earth materials consist of unconsolidated to poorly consolidated sediments deposited within the past 11,000 ± years (Holocene). These sediments were primarily deposited by streams (that is, they are alluvial deposits). However, small areas of lake, marine, and eolian (wind laid) deposits are also present. These deposits consist of interlayered and interfingered clay, silt, sand, gravel, cobbles, and various combinations of these materials. Minor amounts of organic material are probably present in some areas. Portions of these materials may be subject to seismic or hydro consolidation or consolidation due to loading.

(2) "B" Earth Materials

Earth materials designated as type "B" consist of semi-consolidated to moderately consolidated sediments. Most of these materials were deposited during the Pleistocene (11,000 ± to 2,500,000 ± years ago) although some may be slightly older. The nature of deposits and composition of these materials is similar to the type "A" earth materials except that a larger portion of the type "B" earth materials are marine in origin. The main difference between the type "A" and "B" earth materials is thus the greater degree of consolidation of the "B" earth materials. Landslides sometimes occur in these materials where oversteepened slopes exist.

(3) "C" Earth Materials

Earth materials designated as type "C" consist primarily of well consolidated sedimentary rocks of Tertiary age of older (older than 2.5 ± million years). Type "C" earth materials within the City include conglomerate, sandstone, siltstone, and shale. Minor amounts of volcanic rock are also included within the type "C" earth materials. These "C" earth materials range from massive to very well bedded. Slope failure can occur in this material. Failure in these materials is commonly along bedding planes or weaker units (uncemented clay-shale or clayey silt shale) within a bedded sequence. Such planar features are natural zones of weakness.

(4) "D" Earth Materials

Earth materials designated as type "D" consist of dense igneous and metamorphic rocks. In addition, some very well consolidated and lithified, pre-Tertiary (more than 63 ± million years old) metasedimentary rocks are included in this classification.

These rocks contain joint planes, rock cleavage, or foliation planes. These planes of weakness along which landslides may occur.

Tables C-23 through C-27 explicate F,D,P,PC, and SH factors, respectively. A total of 180 points is possible for F+D+P+PC+SH. For general purposes, life hazard potential is rated low if a building receives 5 - 50 points, intermediate if a building receives 50 - 100 points, and high if a building receives 100 - 180 points. Since Q=4 or Q=5 structures should not be inspected, these values generally correspond to Q=3 (low), Q=2 (intermediate) and Q=1 (high) vulnerability categories. For uniformity of application of this method, however, the risk safety factor is included since it also incorporates variations in seismic shaking hazard.[2] In general, buildings may be categorized as follows based on risk safety factors:

Highest Risk	$R_s > 2.0$
Next Highest Risk	$2.0 > R_s > 1.5$
Third Highest Risk	$1.5 > R_s > 1.0$
Lowest Risk	$1.0 > R_s$

For example, if a structure is rated at 100 (F+D+P+PC+ SH), and $A_r = 0.20g$, and it is located on "B" earth materials, then for the structure $R_s = 3.52$. In terms of Balkan region structures to be surveyed, it is expected that A_r (based on hazard studies plus assumptions on the life-span of the structure) will be higher in many regions, and that structures will often exceed a rating above 100. Hence, the R_s factor can be used to calibrate comparative risk of structures in the highest risk category.

In application of the foregoing risk factors, one may also wish to incorporate strength multipliers for varying degrees of structural importance. Table C-28 provides one means of identifying structural importance. As indicated earlier, such importance factors may also be modified with respect to density of usage. Relative to Table C-28, one may multiply R_s by the following factors (in order to obtain R_s'):

STRUCTURE CLASS	STRENGTH MULTIPLIER
1	2.10
2	1.64
3	1.28
4	1.00
5	0.78

The grading of special hazards themselves may lead to requirements by inspectors that those special hazards be abated. In addition, R_s factors can be used to estimate expected deaths.

[2]If one constructs a worst average vulnerability curve from U.S. sources, one finds that expected losses are multiplied by 4.34 for one site as opposed to another if A_r is doubled. If one uses Q=1 and Q=3 vulnerability curves, this factor becomes 7.78 and 7.31, respectively.

A P P E N D I X D

APPENDIX D

ANALYTICAL TREATMENT OF SEISMIC RISK

D.1 INTRODUCTORY CONSIDERATIONS

This appendix is addressed basically to technical bodies of government agencies which set practical guidelines to provide for a satisfactory degree of safety to the existing building stock. Qualified practitioners may also use this appendix as a guide for improved understanding of the seismic risk affecting the existing building stock.

The conceptual framework presented here agrees with the general concepts adopted by UNDRO, [11], and also adapts these seismic risk concepts to existing structures. This adaptation uses the work of Working Group B "Vulnerability and Seismic Hazard" of the UNDP/UNESCO Project RER/79/014, "Earthquake Risk Reduction on the Balkan Region." [12]

The basic concepts, as ordered and defined in [11], are:

1. Natural Hazard -- the probability of occurrence within a specified period of time, in a given area, of a potentially damaging natural phenomenon;

2. Vulnerability -- the degree of loss to a given element of risk (see #4 following) or set of such elements resulting from the occurrence of a natural phenomenon of a given magnitude and as expressed on a scale from 0 (no damage) to 1 (total loss).

3. Specific Risk -- the expected degree of loss due to a particular natural phenomenon and a function of both natural hazard and vulnerability.

4. Elements At Risk -- the population, properties, and economic activities including public services at risk in a given area.

5. Risk -- the expected number of lives lost, persons injured, damage to property, disruption of economic activity, resulting from a particular natural phenomenon; risk is therefore the product of specific risk and elements at risk.

These definitions rely on a consistent philosophy. Yet their generality makes them insufficiently accurate for the needs of more detailed seismic risk assessments of existing buildings. In a broad sense, seismic risk is a product of a system of random factors. Major sources of randomness are represented by the basic data of the problem: the natural hazard (in this case, seismic hazard), the vulnerability of structures (in this case, the seismic vulnerability) and the elements at risk. Since seismic risk evaluation relies essentially on forecasting events (natural events and effects upon man-made works), quantification techniques must rely on probabilistic concepts. These concepts will be applied to characterize basic data as well as to derive conclusions on specific risk and also risk.

A first adaptation of the UNDRO definitions of basic concepts is made here (and elaborated mathematically in later sections of this appendix):

1. Seismic Hazard -- the sequence of seismic actions of various intensities and other characteristics (like spectral characteristics, duration, etc.) expected (in a probabilistic sense) to affect a given site during a specified period of time.

2. Seismic Vulnerability -- the distribution or damage and/or loss resulting from seismic action, considered as a function of an appropriate measure of the seismic action affecting a structure dealt with.

3. Specific Seismic Risk -- damage and/or losses expected (in a probabilistic sense) to affect a structure and its occupants and contents during a specified period of time, given that the elements at risk are permanently exposed.

4. Elements At Risk -- the people, property, cultural values, etc. that can be affected by the seismic action and that may or may not be exposed in connection with a given structure.

5. Seismic Risk -- the damage and/or losses expected (in a probabilistic sense) to affect a structure and the other elements at risk related to that structure during a specified period of time and as a result of seismic action.

These adapted definitions are related to site-specific structures. Their revision would be obviously needed were one to consider geographically dispersed building stocks, lifelines, etc.

It may be noted that:

a) the specific risk must be evaluated by means of appropriate convolutions of the basic data related to seismic hazard and seismic vulnerability (see in this sense Section D.4);

b) the risk must be evaluated by means of appropriate convolutions of the basic data related to elements at risk and previously obtained specified risk evaluation results or products (to which convolutions may be reduced under definite conditions).

Figures D-1, and D-2, which present flow-charts corresponding to steps a and b, respectively, give also a general idea about the outline of Appendix D.

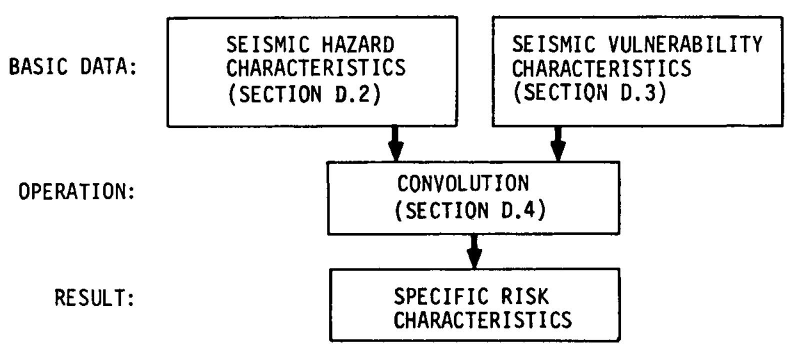


Figure D-1. Evaluation of Specific Risk

Engineering codes at present employ basically deterministic formats to characterize hazard, structural strength, ductility, etc. In later sections, code parameters will be related to those parameters proper to a probabilistic approach. These relations allow one to investigate how varying code parameters influences safety, and to exhibit how code parameters are affected by expected subsequent service duration, and also building importance, as well as elements at risk.

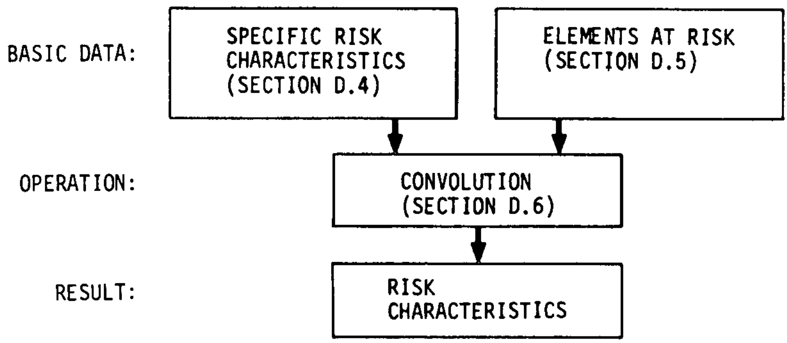


Figure D-2. Evaluation of Risk

D.2 HAZARD CHARACTERIZATION

D.2.1 General

The earthquake hazard at a site represents basically a sequence of seismic actions believed to affect that site in the future. Current knowledge does not permit a deterministic prediction of seismic events. Expectation of future seismic events is therefore considered in terms of numbers of events exceeding certain appropriately defined sizes, and as functions of the future time interval considered. The results of these probabilistic analyses lead to the specification of various design parameters, like those dealt with in Section D.2.

The earthquake action may be present in two basic forms:

a) catastrophic forms, like earthquake-induced landslide, rockfall, subsidence, or ground rupture;

b) normal forms, like motion of ground-structure interface for elevated structures, imposed deformations for embedded structures, etc.

This distinction is of major importance for subsequent activities connected with the mitigation of risk in existing structures. In case the form (a) is present, it is not possible in practice to adopt measures of strengthening existing structures, such as to reduce the earthquake risk. The basic question is whether and to what extent to tolerate further exposure (which may include the acceptable period of subsequent service as well as the acceptable level of exposure of elements at risk). In cases where only form (b) occurs, the various possible measures of strengthening of structures may be efficient in mitigating the risk. Decision makers can consider a wider spectrum of possible ways to confer a suitable degree of safety for the future.

The characterization of earthquake hazard at a given site involves two basic aspects:

a) characterization of an individual occurrence of seismic action;

b) characterization of a sequence of occurrences of seismic action (expected to affect the site considered).

The appropriate characterization of earthquake hazard as well as the various methodologies used for the quantification required basically respresent concerns of engineering seismology.

The parameters used for ground motion characterization in this manual are compatible with code formats. Since no single code format is accepted for all countries where this manual is to be used, the users must themselves correlate the format adopted in this manual with the formats of national codes. Since this manual is concerned with protection of structures against expected future events, some kinds of envelopes of future events believed not to be exceeded with some reasonable probability should be basically considered.

This section covers first characterization of an individual ground motion for use in engineering activities, and second a corresponding characterization of sequences of ground motions occurring at a specific site. Finally, relations are developed between code formats and more analytical characterizations of seismic hazard required for appropriate seismic risk analyses.

D.2.2 Characterization of an Individual Ground Motion

D.2.2.1 Actual Ground Motion

An actual ground motion is defined basically by means of its (corrected, free-field) accelerogram at a designated site. If the time history of ground acceleration in a given direction is $w_g(t)$, its effects upon linear SDOF (single-degree-of-freedom) systems are described by the response spectra for:

a) absolute acceleration,

$$S_a(T,n) \approx \max_t \left| \int_0^t e^{-n\,\omega(t-t')} \sin \omega' \,(t-t')\; w_g(t')\, dt' \right| \tag{D.2.1a}$$

b) relative pseudo-velocity,

$$S_p(T,n) \approx \max_t \left| \int_0^t e^{-n\,\omega(t-t')} \sin \omega'(t-t')\; w_g(t')\, dt' \right| \tag{D.2.1b}$$

c) relative displacement,

$$S_d(T,n) = \max_t \left| \int_0^t e^{-n\,\omega(t-t')} \sin \omega' \,(t-t')\; w_g(t')\, dt' \right| \tag{D.2.1c}$$

The system parameters used are:

a) the (undamped) natural circular frequency, ω;

b) the fraction of critical damping, n;

c) the (undamped) natural frequency, $f = \frac{\omega}{2\pi}$ (D.2.2a)

d) the natural (undamped) period, $T = \frac{2\pi}{\omega} = \frac{1}{f}$ (D.2.2b)

e) the reduced (damped) natural circular frequency,

$$\omega' = \omega \quad 1-n^2 \qquad (D.2.2c)$$

The approximate relations

$$S_d(T,n) \approx \frac{1}{\omega'} S_p(T,n) \approx \frac{1}{\omega'^2} S_a(T,n) \qquad (D.2.3)$$

are valid.

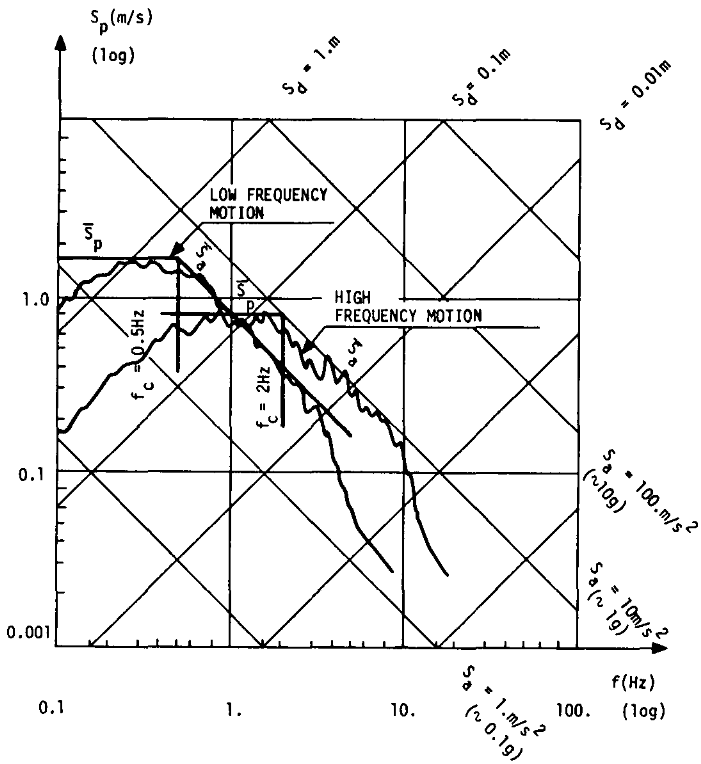


Figure D-3. Spectral Accelerations, Spectral Pseudo-Velocities and Corner Frequencies for Two (High and Low Frequency) Response Spectra (n=0.05)

The three response spectra defined by relations (D.2.1) may be represented by a unique plot (tri-partite diagram), as in Figure D-3, where illustrative examples are given for the spectra of a low (predominant) frequency motion and for a high (predominant) frequency motion, as determined for n = 0.05. An actual motion can be characterized in a simple manner by a spectral acceleration $\bar{S}_a$ and by a spectral pseudo-velocity $\bar{S}_p$, as drawn in that figure. The straight lines corresponding to the spectral characteristics $\bar{S}_a$ and $\bar{S}_p$ play the role of envelopes and may be exceeded only locally by one to two narrow peaks of the response spectrum. The crossing of the straight lines corresponding to the spectral acceleration and to the spectral pseudo-velocity defines the corner frequency f_c:

$$f_c = \frac{1}{2\pi} \frac{\bar{S}_a}{\bar{S}_p} \qquad (D.2.4)$$

(where consistent units are used for measuring $\bar{S}_a$ and $\bar{S}_p$).

The corner frequency may lie in the range from 0.4 to 0.8 Hz. for low frequency motions and from 1.5 to 4.0 Hz. for high frequency motions (the latter being the most common under conditions of crustal sources and harder soil).

It may be useful to characterize a ground motion by means of the peak ground acceleration,

$$PGA = \max_t \int w_g\ (t)| \qquad (D.2.5)$$

and, correspondingly, of a non-dimensional dynamic factor, the normalized response spectrum (of absolute acceleration),

$$S_a^o\ (T,n) = \frac{S_a(T,n)}{PGA} \quad \text{(as a result, } n = 0.05\text{)} \qquad (D.2.6)$$

It may be useful also to characterize a ground motion by means of the effective peak acceleration (EPA) and effective peak velocity (EPV), defined by means of the relations

$$EPA = \frac{1}{2.5}\ \bar{S}_a \qquad (D.2.7a)$$

and

$$EPV = \frac{1}{2.5}\ \bar{S}_p \qquad (D.2.7b)$$

A ground motion may also be characterized by means of the macro-seismic (MSK or MM) intensity, I. In case there exists a record of ground motion, the macro-seismic (MSK or MM) intensity be estimated by means of the relation

$$\begin{aligned} I &= \log_4 EPA(m/s^2) + \log_4 EPV(m/s) + 8 \qquad (D.2.8) \\ &= \log_4 [EPA(m/s^2) \times EPV(m/s)] + 8 \end{aligned}$$

with an averaging of the products EPA x EPV obtained for two orthogonal horizontal directions.

To illustrate the use of previous relations, one can consider in Table D-1 two examples of motions of intensity VIII,

D.2.2.2 Idealized Ground Motions and Code Provisions

The accelerograms, as well as the response spectra of actual ground motions, exhibit considerable randomness. Accelerograms and response spectra are always different even for different ground motions occurring at a single site and from the same source zone. Practically speaking, ground motions must be idealized at a level that is compatible with available knowledge and technology for forecasting motions expected to affect a designated site.

Table D-1. Examples for Parameters Characterizing Motions of Intensity I = VIII According to Relation (D.2.8)

PARAMETERS	TYPE OF MOTION	
	LOW PREDOMINANT FREQUENCY	HIGH PREDOMINANT FREQUENCY
f_c (Hz)	0.64	2.55
$\bar{S}_a$ (m/s^2)	5.	10.
$\bar{S}_p$ (m/s)	1.25	0.625
EPA (m/s^2)	2.	4.
EPV (m/s)	0.5	0.25

Current advanced design codes are based on the response spectrum concept. A design coefficient (related, as a rule, to any horizontal direction of translational motion of ground) may be determined in principle on the basis of an expression

$$c_s(T) = k_s \beta(T) \phi, \qquad \text{(D.2.9)}$$

where k_s is a non-dimensional ratio

$$k_s = \frac{PGA}{g} \quad \left(\text{or} = \frac{EPA}{g}\right) \qquad \text{(D.2.10)}$$

(g = acceleration of gravity);

PGA = peak ground acceleration of design motion defined on the basis of expression (D.2.5)

$\beta(T)$ = an idealized, normalized response spectrum for absolute acceleration, corresponding basically to the expression (D.2.6), but extrapolated for future ground motions

ϕ = a reduction factor, depending on the type of structure dealt with, that permits one to take into account the ability of structures to withstand earthquakes as a result of strength reserves as well as ductility reserves

The idealized normalized response spectrum $\beta(T)$ should correspond to the types of motions having occurred (and anticipated to occur) at a site. They should accord with the features of specific earthquake mechanisms and of site conditions. Illustrative examples of such dynamic factors are given in Figure D-4 for low frequency motions and for high frequency motions, respectively. For some sites, different classes of motions of different predominant frequencies may be important to consider along with appropriate, different, dynamic factors $\beta(T)$ to be defined accordingly.

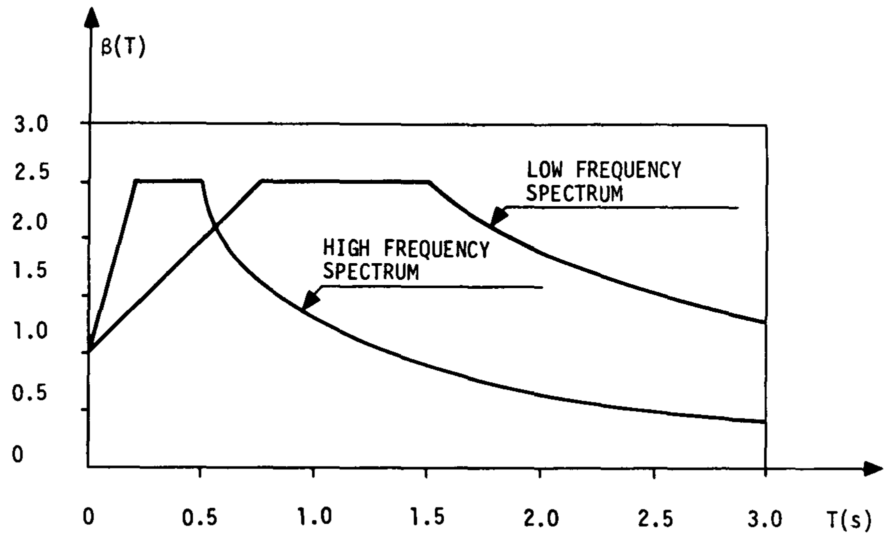


Figure D-4. Examples of Normalized Response Spectra (Dynamic Factors)

D.2.3 Characterization of Sequences of Ground Motion Affecting a Site

D.2.3.1 Sequences of Past Ground Motions

Consider the sequence of past seismic motions having affected a given site over the last decades or centuries, and imagine that the normalized, smoothed response spectra of those earthquakes are rather similar (more precisely, that corner frequencies, as defined by expression (D.2.4), are about the same). In this case, it is possible to characterize statistically the sequence of past events in terms of such statistics as spectral accelerations ($\bar{S}_a$), effective peak accelerations (EPA), peak ground accelerations, (PGA), and macroseismic intensities (I). These intensity parameters may be denoted by q, and the time duration referred to is T. It is then possible to consider the numbers of events N(q,T) having exceeded given thresholds q during the time T. The number N(q,T) will monotonically decrease when q increases and monotonically increase when T increases.

In contrast, the response spectra of past events may have been greatly dissimilar -- the corner frequencies may be relatively low for some events (e.g., for motions due to larger magnitude earthquakes with remote sources) and relatively high for other events (e.g., for motions due to smaller magnitude earthquakes with nearby sources). In this case, it is suitable to consider distinct classes of motions corresponding to different shapes of the normalized response spectra, or, more specifically, to different corner frequencies. One could, for instance, consider two classes of earthquakes with corner frequencies of approximately 0.6 Hz. and of approximately 2.0 Hz., respectively. In this case, it makes sense to consider separate statistics for each of the classes of motions referred to.

D.2.3.2 Probabilistic Characterization of the Sequences of Future Events

The forecast of seismic events expected to affect a definite site represents an extrapolation from past experience. The sequence of future events will

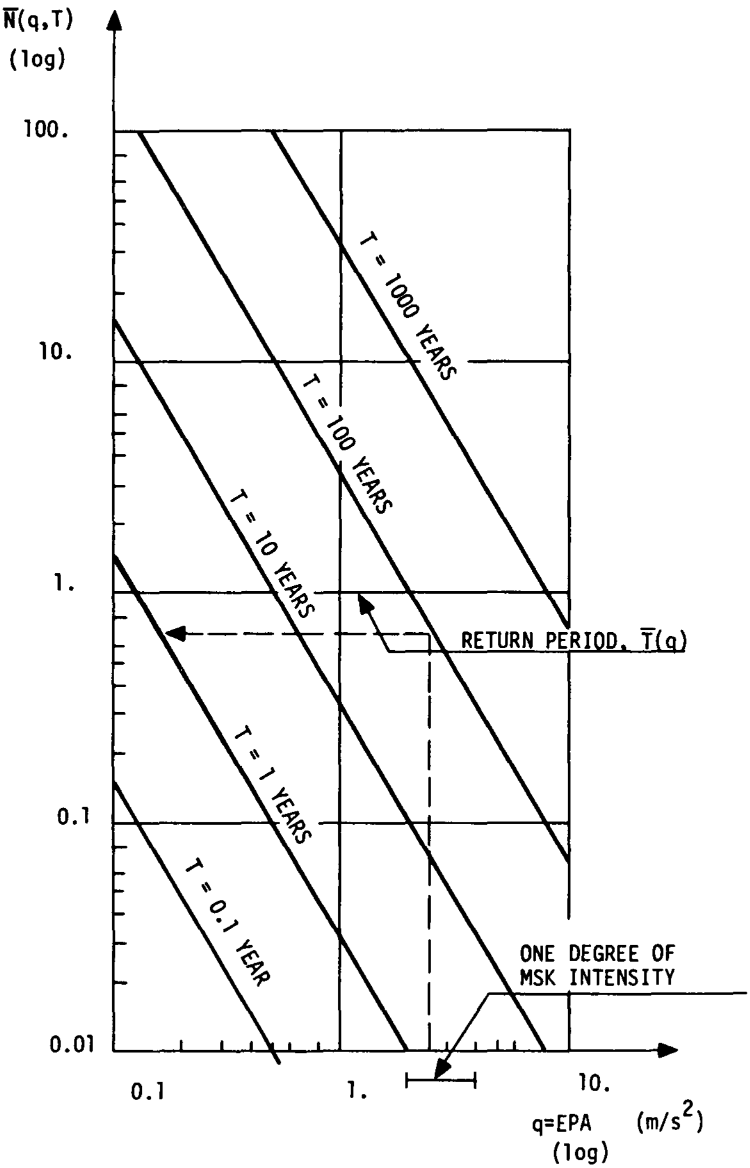


Figure D-5. The Expected Number of Exceedance Cases for a Type II Extreme Value Distribution (Maxima) be characterized basically by probabilistic concepts and will hence be considered to be a stochastic process. If no evidence exists to the contrary, it may be assumed that this process is stationary (i.e., the expected sequence of events for a time interval does not depend on the origin of the time interval considered) and that the sequence of future events does not depend in a probabilistic sense on the sequence of past events. These assumptions imply that the sequence of seismic events is a Poisson process.

Statistical analysis of the seismic activity of various source zones, as of the earth as a whole, shows that a law such as

$$\log N = a - bM, \qquad (D.2.11)$$

fits well for magnitudes M not exceeding some upper bound for the zone (in the above equations, a and b are constants, M is the Richter magnitude and N is the number of events of magnitudes higher than M, occurring during a time interval of definite (long) duration). In fact, the constant "a" depends on the time interval considered. This type of law is valid, under similar limitations, for the macroseismic intensities affecting a specific site. In a similar way, a law such as

$$\log N = a' - b' \log PGA \qquad (D.2.12)$$

will be valid, under homologous limitations, for peak ground accelerations affecting a site, or for such other kinematic parameters as $\bar{S}_a$ or EPA. The law (D.2.11) represents an extreme value distribution of type I (maxima), while the law (D.2.12) represents an extreme value distribution of type II (maxima).

An accurate hazard characterization requires more accuracy in describing extreme value distributions and, also, a discussion of the connection between these laws and the probabilities of non-exceedance of some given thresholds of intensity, peak ground acceleration, etc. Any of the laws (D.2.11) or (D.2.12) is compatible with the relation

$$\bar{N}\,(q;T) = T \int_q^\infty s(q)\,dq, \qquad (D.2.13)$$

where $\bar{N}$ (q; T) is the expected number of occurrences of ground motions of intensity exceeding q, during a time interval of duration T, while s(q) is the density of expected number of ground motions of intensity q for a unit time interval (unit may mean one year, or possibly a century, etc.; the intensity q is understood in a broad sense and it may represent MSK intensity, PGA, etc.). The (average, or expected) return period of a value q, $\bar{T}$ (q), is given by the condition

$$\bar{N}\,(q;\ \bar{T}\,(q)) = 1 \qquad (D.2.14)$$

For two different values q_1 and q_2, one may use the following conventions:

$$\bar{T}\,(q_1) = \bar{T}_1 \qquad (D.2.15)$$

$$\bar{T}\,(q_2) = \bar{T}_2$$

The type II law (D.2.12) may be rewritten more accurately in the form

$$\ln \bar{N}(q;\ T) = \ln T - \frac{(\log_2 q_2 - \log_2 q)\ \ln \bar{T}_1 + (\log_2 q - \log_2 q_1)\ \ln \bar{T}_2}{\log_2 q_2 - \log_2 q_1} \tag{D.2.16}$$

The homogeneity of relation (D.2.16) permits one to replace the bases e of ln and/or 2 of $\log_2$. However, this is the most natural form, since $\log_2 q$ (where q is a kinematic parameter) increases in the same way as the macroseismic intensity, so that it leads directly to a type I law:

$$\ln \bar{N}(I;\ T) = \ln T - \frac{(I_2 - I)\ \ln \bar{T}_1 + (I - I_1)\ \ln \bar{T}_2}{I_2 - I_1} \tag{D.2.17}$$

At the same time, the use of natural logarithms brings some simplification in the calculation of derivatives. The density s(q) introduced by the relation (D.2.13) corresponding to the law (D.2.16) will be

$$s(q) = \frac{(\ln \bar{T}_2 - \ln \bar{T}_1)}{(\ln q_2 - \ln q_1)} \frac{1}{q} \tag{D.2.18}$$

where, again, the common logarithm base could be changed.

The law (D.2.16) is illustrated in Figure D-5. The law (D.2.17) would imply replacement of the logarithmic scale of abscissa by means of a natural scale (for macroseismic intensities or Richter magnitudes).

The probabilities of m occurrences with intensities exceeding q will be

$$G_m(q;T) = \frac{[\bar{N}(q;T)]^m}{m!} \exp\left[-N\ (q;T)\right] = \frac{[T/\bar{T}(q)]^m}{m!} \exp\left[-T/\bar{T}(q)\right] \tag{D.2.19}$$

(m = 0,1,2...)

In particular, the non-exceedance probability of a value q will be

$$G_o\ (q;T) = \exp\left[-\bar{N}(q;T)\right] = \exp\left[-\frac{T}{\bar{T}(q)}\right] \tag{D.2.20}$$

and it is given in Table D-2 as well as in Figure D-6.

The upper part of Table D-2 may be read as follows: during a period of service half as long as the return period of a value q, the non-exceedance probability of that value is .607. The lower part of Table D-2 may be read as follows: in order to provide a non-exceedance probability of .7, the return period must be 2.80 times longer than the duration considered (e.g., for a service period of 50 years, the corresponding return period must be 140 years).

Table D-2. Values of the Non-Exceedance Probability, $G_o(q;T)$

$\bar{N}(q;T) = \frac{T}{\bar{T}(q)}$	$G_o(q;T)$
.1	.905
.2	.819
.3	.741
.4	.670
.5	.607
.6	.549
.7	.497
.8	.449
.9	.407
1.	.368
1/9.49	.9
1/4.48	.8
1/2.80	.7
1/1.96	.6
1/1.44	.5

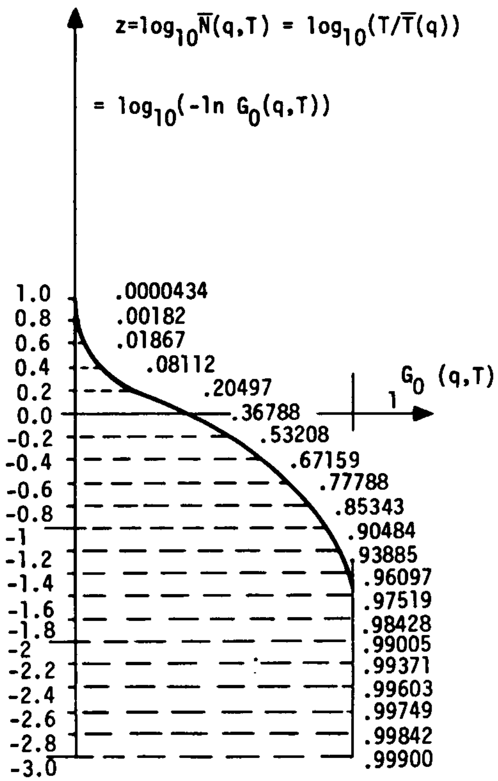


Figure D-6. Relation Between Expected Number of Exceedance Cases and Non-Exceedance Probability

The hazard characterization given by the previous relations is basically appropriate in case one can consider, for a definite site, a unique class of seismic motions (characterized by rather similar normalized response spectra). In contrast, when several distinct classes of motions must be considered, relations (D.2.13) through (D.2.20) apply separately to each of the classes. Consideration of the combined effects of all classes of motions will be made in connection with specific risk evaluation (see Section D.4).

D.2.4 Additional Comments on Seismic Hazard in Connection with Code Provisions

It was mentioned previously that relation (D.2.9) is used to specify a design spectrum, that the factors k_s and $\beta(T)$ occurring in the member of that relation are connected with the features of ground motions, and that the reduction factor ψ is related to the features of structural performance. The dynamic factor $\beta(T)$ will be adopted to lead to basically equal exceedance probabilities for sites with different local conditions. In one way or another, codes prescribe some differentiations of the values of the factor k_s for a similar site but in order to provide different safety levels to structures of varying importance.

In order to analyze the influence of variations of the factor k_s on the level of safety, it is useful to consider the definition (D.2.14) of the return period $\bar{T}$ (q) and to apply it in connection with the relation (D.2.16). Based on expression (D.2.16), condition (D.2.14) leads to the expression

$$\Delta \ln\bar{T}(q) = \frac{\ln\bar{T}_2 - \ln\bar{T}_1}{\log_2 q_2 - \log_2 q_1} \log_2 q = \frac{\ln\bar{T}_2 - \ln\bar{T}_1}{\log_2 q_2 - \log_2 q_1} \Delta \log_2 k_s \qquad (D.2.21)$$

or, what is equivalent, using base 10 logarithms,

$$\Delta \log\bar{T}(q) = \frac{\log\bar{T}_2 - \log\bar{T}_1}{\log q_2 - \log q_1} \Delta \log k_s \qquad (D.2.21')$$

The ratio $\frac{\log\bar{T}_2 - \log\bar{T}_1}{\log q_2 - \log q_1}$ should belong, as a rule, to the interval (1.5, 2). Thus, an increase of k_s by 50% implies an increase of $\bar{T}$ (q) by some 100%, while an increase of k_s by some 100% implies an increase of $\bar{T}$ (q) by 200% to 300%.

Should exceedance probabilities $1 - G_o(q;T)$ be low, then, as a consequence of (D.2.20),

$$1 - G_o(q;T) = 1 - \exp[- \frac{T}{\bar{T}(q)}] \approx \frac{T}{\bar{T}(q)} \qquad (D.2.22)$$

A doubling of $\bar{T}(q)$ leads to a halving of the ratio $\frac{T}{\bar{T}(q)}$, i.e., of the number of cases of exceedance, etc..

D.3 VULNERABILITY CHARACTERIZATION

D.3.1 General

A facility subjected to strong seismic action may be affected in many different ways. Physical damage of various degrees (apparent or hidden) may occur at different points or zones and losses of various sorts (to people, property, cultural values etc.) may occur. The physical damage as well as the earthquake induced losses will depend first on the intensity of seismic action. However, direct post-earthquake observation of damage and other effects evidences considerable randomness in the degree of damage and loss, even for structures designed identically and subjected to nominally equal seismic intensity. This randomness, as well as the practical impossibility of accurate deterministic analysis of individual structures, makes it necessary to consider a statistical distribution of damage and losses. One can consider, for instance, damage and loss histograms derived from post-earthquake surveys; one can also consider damage and loss distributions predicted on the basis of a probabilistic analysis of structural behavior and of earthquake induced losses.

Accordingly, the vulnerability of a type of building or other structure will be represented by a distribution (in the probabilitic or statistical sense) of earthquake induced damage and losses, considered as a function of seismic intensity.

One can distinguish here between observed vulnerability and predicted vulnerability. The observed vulnerability represents basically a system of histograms of the degrees of damage and loss (defined separately for different seismic intensities) as derived from post-earthquake surveys. The observed vulnerability can be related to a given class of buildings or other structures. Each class must be sufficiently homogeneous relative to layout, nature and quality of materials, etc. In contrast, the predicted vulnerability represents a family of distributions of damage and loss, depending on a parameter representing the seismic intensity. The predicted vulnerability can be related to a specific type of building or other structure. Consideration of a broader class of structures including several types of buildings will lead to some corresponding averaging.

The vulnerability of a class or type of buildings will be related to a single occurrence of seismic action. Following a strong earthquake, the intervention of man will as a rule change the characteristics of a building affected by an earthquake so that its vulnerability will be modified (even if no intervention occurs, the damage produced by a strong earthquake will have some more or less serious negative implications for its vulnerability). While vulnerability is related to an individual seismic event, specific risk and risk (as dealt with in subsequent sections of this chapter) are related to sequences of earthquakes expected to affect the site during a specified period of time.

A classification of the possible kinds of seismic action was given in Section D.2.1. That classification is significant from the viewpoint of vulnerability too. In case seismic actions occur in the form (a) ("catastrophic"), the vulnerability of a structure will be in principle total, i.e., total destruction, with corresponding implications for human life and property. Vulnerability ceases in this case to be a characteristic of a structure; the hazard will be directly converted into specific risk. In the normal case, when seismic actions occurs in form (b) ("normal"), damage and losses involved may be very different and the vulnerability of a structure is a significant, basic element in the risk assessment procedure.

Engineering analyses must be focused on the evaluation of predicted vulnerability. These may rely on data on observed vulnerability, as obtained from appropriate damage surveys (see Chapter 2), from techniques of evaluation (see Chapter 5), and from other evaluation methods or sources in the literature.

D.3.2 Parameters of Reference for the Vulnerability Characteristics

Characterization of vulnerability requires adoption of appropriate specifications with respect to:

a) the class or type of structure;
b) the damage or loss due to seismic action;
c) the seismic action.

The specifications given in Chapter 2 of the manual in relation to the classification of structures and to the evaluation of damage should be considered in this regard. Besides those specifications, some general recommendations are given later.

The classification and description of structures should consider the following main characteristics and data:

1. Type of structural system.

2. Foundation conditions.

3. Azimuthal orientation.

4. Dynamic characteristics (primarily fundamental periods corresponding to oscillations along different directions).

5. Age (year of construction).

6. Degree of engineering (design, materials, workmanship, inspection).

7. Estimates on the degree of protection against lateral forces (base shear coefficient corresponding to limit of elastic stage, ductility characteristics and limit deformations for horizontal and/or vertical forces).

8. Data on previous overloading (earthquakes, other accidents), corrosion etc.

9. Function.

10. Outline drawing.

The assessment of degree of damage or loss of function for a given structure should consider following general guidelines:

1. The degree of damage will be estimated by a method consistent with that used to evaluate MSK macroseismic intensity. A calibration from 0 (no damage) to 1 (collapse) is recommended.

2. Whenever possible, a distinction will be made between damage undergone by structural members and damage undergone by non-structural members.

3. For a definite class of structures, specific guidelines should be elaborated to convert degree of damage into percent diminished of the capacity to resist seismic action, given the cumulative nature of damage.

4. For a definite class of structures, specific guidelines should be elaborated to convert degree of damage into losses (economic losses, risk for people, potential chains of effects).

Section D.A.2 provides further details on damage quantification techniques as used in the 1977 engineering post-earthquake surveys in Romania.

The seismic action should be characterized according to the specifications given in Section D.2.2. The parameter q used in further relations may represent $\bar{S}_a$ or EPA, given specification of the predominant frequency (or of the corner frequency) of the class of seismic ground motions considered. For a site expected to be affected by seismic actions belonging to different classes (as referred to in Section D.2.3), vulnerability characteristics may be specified separately for the different classes of motions believed to affect the given site.

D.3.3 Quantification of Vulnerability

The quantification of vulnerability considered in this section is related to the observable damage degree (DD). It is assumed here that a discrete system of increasing damage degrees D_k is defined for a class of structures.

The observed vulnerability is characterized in this case by the relative frequencies $f_k(q)$ of the damage degrees D_k owing to a seismic action of intensity q. The predicted vulnerability is characterized by the probabilities $p_k(q)$ of the damage degrees D_k owing to a seismic action of intensity q. While the parameters $f_k(q)$ represent essentially a family of histograms, the parameters $p_k(q)$ represent a family of discrete probability distributions.

Intensities q may also be discretized or lumped into some values q_j that are consistent with integer macroseismic intensities or with halves of such integers also. In this case, the functions $f_k(q)$ and $p_k(q)$ will be replaced by matrices of values $f_{k/j}$ or $p_{k/j}$, respectively (called also damage probability matrices).

D.3.4 Vulnerability Evaluation

The evaluation of vulnerability of structures should rely first on a deterministic approach, intended to exhibit some qualitative features of structural behavior as well as the specific parameters of reference for a type of structure dealt with. This step should be then completed by a probabilistic approach, aimed basically to estimate the scatter of specific characteristics and thresholds of the type of structure considered.

Some recommendations for the deterministic step are made later. Consider for example a simple, low-rise building, as represented in Figure D-7. The analysis of the performance of such a structure could lead to a relationship between the (static) lateral force f(z) and the (static) internal deflection, u, as in Figure D-8. In order to develop this relationship, it is necessary first to identify the various possible failure mechanisms and next to determine (in a deterministic formulation) the various deflections u_k and forces F_k corresponding to the damage degrees D_k. Consider in this relation a distribution of deflections

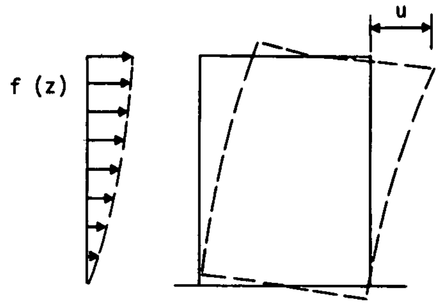


Figure D-7. Scheme of a Simple Structure

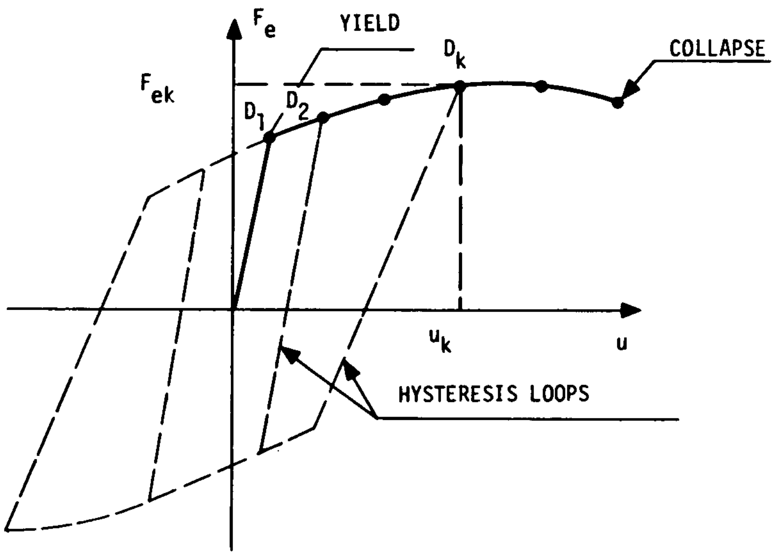


Figure D-8. Force-Deflection Characteristic of the System of Figure D-7

$$v(z) = u\ \eta(z) \tag{D.3.1}$$

where u is the deflection at some reference point (e.g., at the upper floor) and $\eta(z)$ is an influence coefficient. An equivalent mass m_e may be defined on the basis of distributed mass, $\mu(z)$:

$$m_e = \int \mu(z)\ \eta^2(z)\ dz \tag{D.3.2}$$

An equivalent force F_e may also be defined:

$$F_e = \int f(z)\ \eta(z)\ dz \tag{D.3.3}$$

An equivalent static acceleration, a_e, will be given by the expression

$$a_e = \frac{F_e}{m_e} \tag{D.3.4}$$

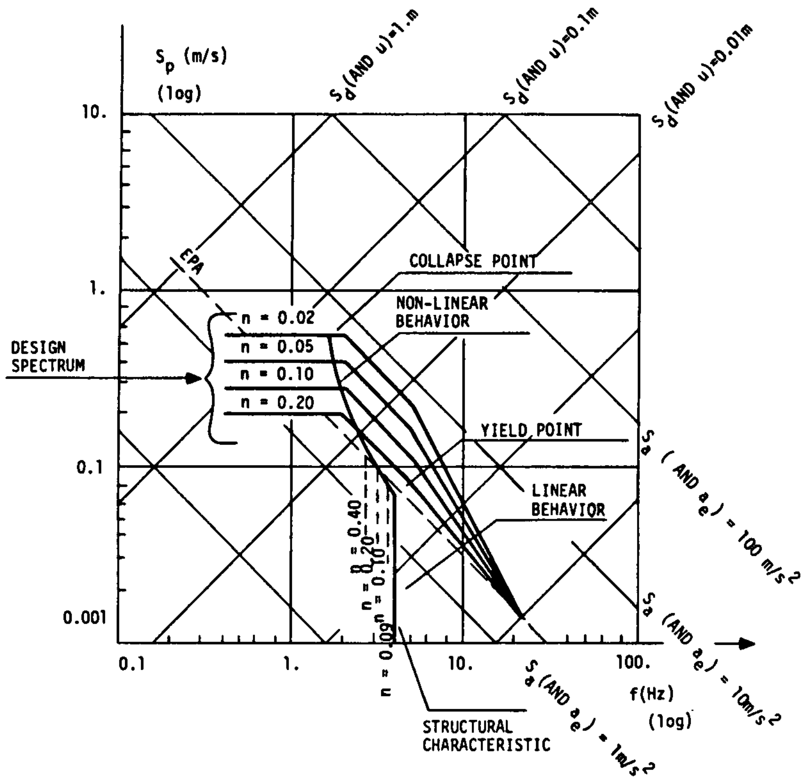


Figure D-9. Structural Characteristics versus Design Spectrum (Deterministic Approach) : Estimated Performance Corresponds to Coincidence pf Values n for Structural Characteristics and Design Spectrum

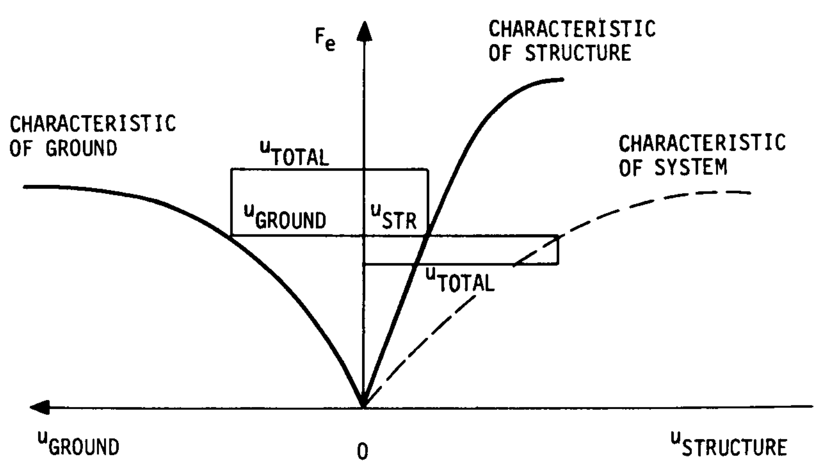


Figure D-10. Consideration of Ground Compliance in Deriving Force-Deflection Characteristic for the System

The natural circular frequency ω_I can be evaluated (in the elastic range) by means of Rayleigh's formula,

$$\omega_I^2 = \frac{a_e}{u} = \frac{F_e}{m_e u} \qquad (D.3.5)$$

The corresponding stiffness, k_e, will be given by the relation

$$k_e = m_e\, \omega_I^2 = \frac{F_e}{u} \qquad (D.3.6)$$

By extension, one can consider the approximate values of ω_I and k_e corresponding to non-linear behavior. A simple way to do this is represented by the "secant" stiffness and the corresponding natural frequency. The points D_k on the plot will correspond to some specific damage degrees, or levels, or thresholds (e.g., failure of infill masonry, plastic hinges in a framed structure, failure of lintels for a bearing wall structure). In Figure D-8, some hysteresis loops corresponding to the various loading amplitudes are also desirable to determine. The equivalent fraction of critical damping, n_e, can be evaluated by means of the relation

$$n_e = \frac{1}{4\pi}\,\frac{\text{hysteresis loop area}}{\text{potential energy area}} \qquad (D.3.7)$$

A representation as in Figure D-8 makes it possible to construct a relationship between the equivalent fraction of critical damping, n_e, and the amplitude (u). A tripartite diagram, as in Figure D-9, can be used to represent the relationship between u and ω_I and to discern from the plot the points (and the associated vertical straight lines) that correspond to various values of the fraction of critical damping, such as n = 0.05, 0.10, and 0.20. Through superimposing the plot representing the relationship between u and ω_I on the plots representing a family of design spectra for various values n, one can obtain an approximate idea about the level of loading and damage. The intersection of the (u, ω_I) plot may be considered with the design spectra corresponding to a value in that is equal to n_e (u) or n_e (w_I) [7]. Especially for rigid buildings, it is important to consider the total equivalent deflection, u_{total}, given by the expression

$$u_{total} = u_{structure} + u_{ground} \qquad (D.3.8)$$

where u_{ground} is the deflection corresponding to ground compliance (Figure D-10). This simplified approach may be suited for evaluation of relatively simple structures, as mentioned previously. But more refined approaches are required for structures where the identification of failure mechanisms requires first the consideration of several D.O.F.

As a first approximation, probabilistic evaluations required by vulnerability analysis could be made through determining the expected values of F_{ek} and u_k in Figure D-8, the corresponding root mean square values and, the correlation between F_{ek} and u_k. As a result, ellipses can be plotted as in Figure D-11, characterizing the r.m.s. (root mean square) scatter of the points D_k. These ellipses can be converted into ellipses on the tripartite plot, as in Figure D-12. The degree of confidence of not exceeding some damage degree D_k may be estimated in this case by means of the ratio of the radius r from the plot to the design spectra, to the radius r_e of the ellipse in the same direction, as for a normal (Gaussian) distribution. Under these assumptions, it is possible to estimate the conditional probabilities $p_{k/j}$ referred to in Section D.3.3. This approximation may be made by first considering the family of smoothed spectra (for different values n) corresponding to a given intensity (e.g., EPA) $q = q_j$. Next, for the damage level D_k (which corresponds to a certain value $n = n_k$), one evaluates the minimum ratio r/r_e for the various directions, starting from the center of the ellipse corresponding to the damage degree D_k.

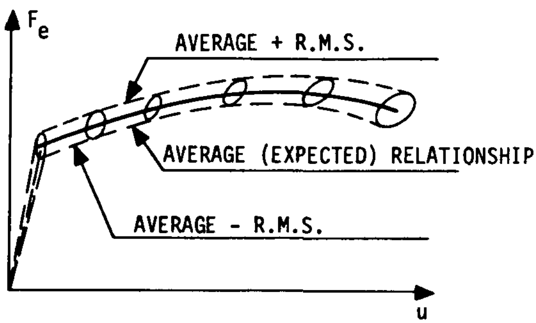


Figure D-11. Force-Deflection Characteristic Considering Randomness of Material Properties

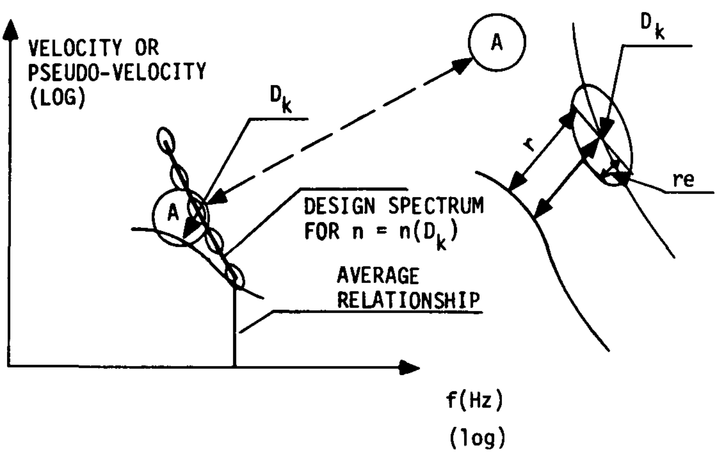


Figure D-12. Stiffness-Deflection Characteristic Versus Spectrum Considering Randomness of Material Properties

Background data on vulnerability characteristics come from some estimates given in [10] about the conditional failure probability as a function of the ratio of EPA characterizing an actual ground motion to the EPA used for design. The data plotted in Figure D-13 represent estimates referring to structures engineered according to U.S. practice. Results obtained from post-earthquake surveys provide another source of relevant information. Reproduced in Figure D-14 histograms on the conditional damage distribution obtained in the 1977 post-earthquake studies in Romania and for a definite class of buildings (bearing wall masonry structures, with reinforced concrete floors, built prior to 1940, and not engineered to resist earthquakes) [12]. The damage degree was evaluated according to the methodology outlined in Section 4.2. Results are highly scattered mainly because the statistical population is non-homogeneous (differences in number of stories, in layout, in material quality and workmanship, etc.). This example, basically involving an averaging about a broader class of different types of structures, exhibits the need for suitable classification of buildings in

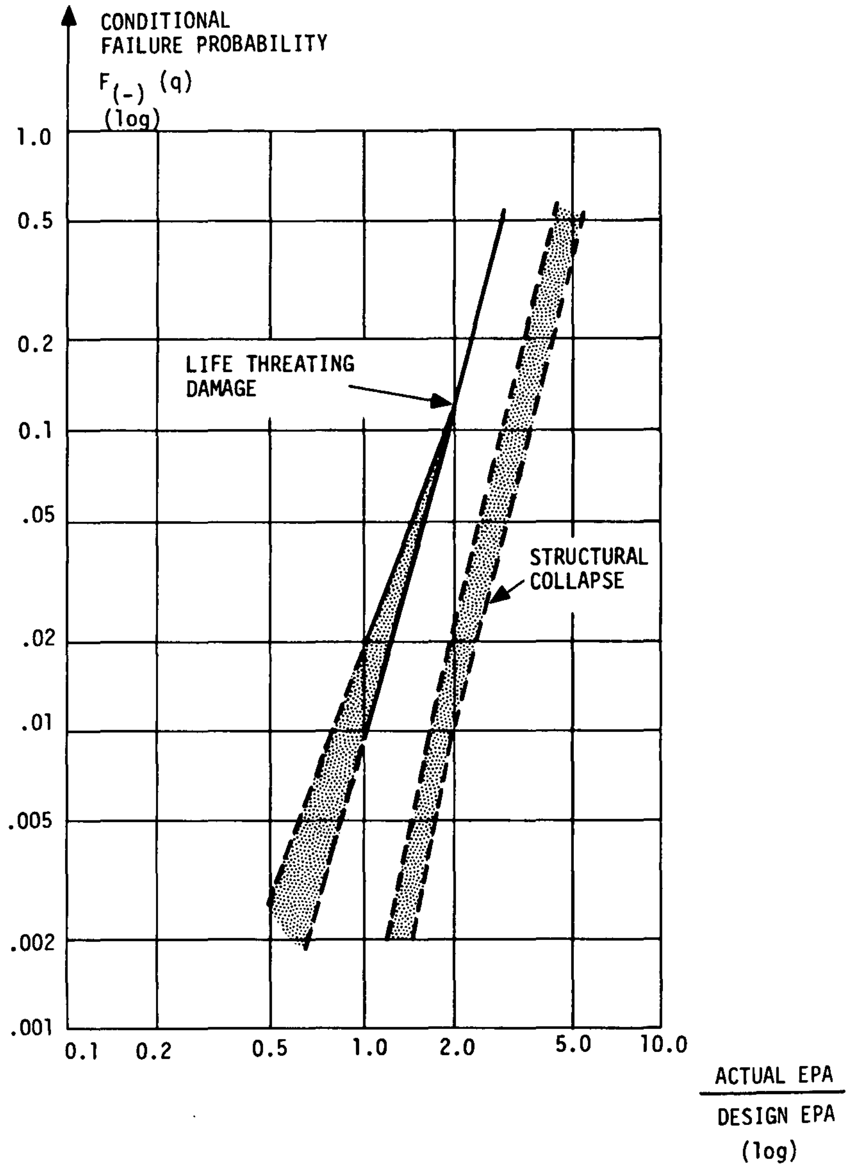


Figure D-13. Conditional Failure Probabilities According to [10]; As Derived for USA Conditions order to use post-earthquake survey results to evaluate the vulnerability of a definite building type.

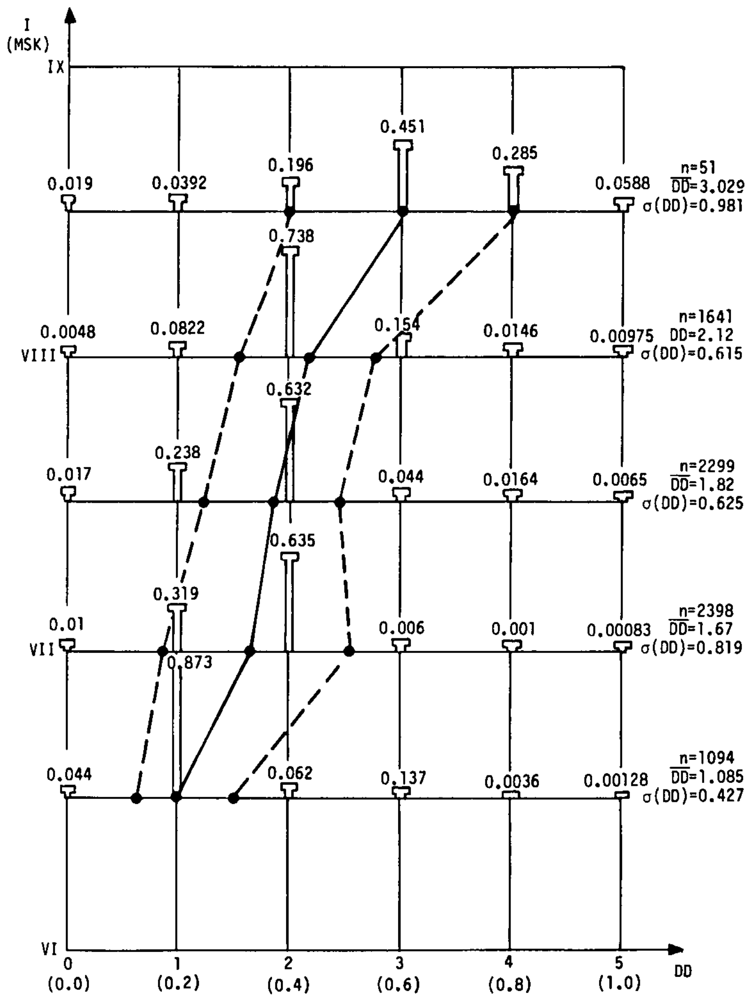


Figure D-14. Vulnerability Characteristics (Histograms, Average Values and R.M.S. of DD) Obtained in Romania for Non-Engineered Old Brick Masonry Buildings with R.C. Floors [12]

In the diagram in Figure D-13, for the solid part of the plot referring to the failure of structures, the approximate relationship

$$\Delta \log F_{(-)} \text{ (actual EPA)} \approx 5 \, \Delta \log \text{ (actual EPA)} \qquad \text{(D.3.9)}$$

holds, where $F_{(-)}(q)$ represents the conditional failure probability (i.e., $p_k(q)$ for the ultimate k). This relation is equivalent, for the range referred to, to the relation

$$\frac{F_{(-)}(EPA_2)}{F_{(-)}(EPA_1)} \approx \frac{EPA_2}{EPA_1}^5 \qquad (D.3.10)$$

D.4 EVALUATION OF SPECIFIC RISK

D.4.1 General

The specific risk represents basically the expected effects of seismic actions anticipated to affect the structures dealt with, and on the assumption that some elements at risk are permanently exposed. Given that the specific risk represents essentially a convolution of hazard and vulnerability, each of them being unpredictable in a deterministic manner, any evaluation of specific risk must rely in some way on a probabilistic forecast. It is therefore necessary to consider the expectation of future earthquake actions and effects in a way that is essentially similar to that of hazard characterization. Bearing in mind the state-of-the-art of engineering practice, one must also consider the connections between the application of probabilistic concepts on one hand and the use of code formats on the other hand.

The specific risk may be present in two basic forms that are homologous to those referred to in Section D.2.1:

a) cases in which the earthquake action in catastrophic forms makes it impossible to protect structures or human lives and the eventual result essentially represents destruction and total loss;

b) cases in which the earthquake action in moderate forms involves the possibility of more or less severe effects, as determined, essentially, by the ratio of seismic intensity to structural resistance.

The basic difference between the two situations referred to is that in cases of category (a) structures are totally vulnerable (so that the measures of hazard and of risk, respectively, must be basically equal), whereas in cases of category (b) the vulnerability of structures in a matter of degree and introduces some kind of weighting of hazard characteristics in the evaluation of specific risk.

The specific risk should be considered relative to a unique future possible seismic event. Subsequent to such an event a structure affected should be considerably modified by human intervention like repair or strengthening or even demolition. It does not make sense, therefore, to consider the effects of a sequence of several strong seismic actions on the same unmodified structure. Nevertheless, as an auxiliary means of measuring the risk, it may be advantageous to consider the cumulated expected losses affecting a structure during a specified period of subsequent service.

D.4.2 Convolutions of Hazard and Vulnerability Characteristics

Consider following basic characteristics of hazard and vulnerability, as referred to in Sections D.2 and D.3, respectively:

a) the density of expected number of occurrences of seismic action of intensity q, $s(q)$;

b) the failure probability in case of seismic action at intensity q, $F_{(-)}(q)$;

The expected number of failures during a time interval of duration T will be

$$\bar{N}_{(-)}(T) = T \int_0^{\infty} s(q)\ F_{(-)}(q)\ dq = T\ I_{(-)} \tag{D.4.1}$$

where it may be useful to consider, as an auxiliary measure of specific risk, the parameter

$$I_{(-)} = \int_0^{\infty} s(q)\ F_{(-)}(q)\ dq, \tag{D.4.2}$$

or, the expected number of exceedance cases during a unit time interval. The survival probability under these conditions will be

$$H_{(+)}(T) = \exp\left[-\bar{N}_{(-)}(T)\right] = \exp\left[-T \int_0^{\infty} s(q) F_{(-)}(q) dq\right] \tag{D.4.3}$$

In case of low exceedance probabilities, the relation

$$1 - H_{(+)}(T) \approx \bar{N}_{(-)}(T) \tag{D.4.4}$$

will be valid, which shows that the failure probability is approximately equal to the expected number of cases of exceedance.

The importance of the duration T is obvious, since this parameter acts as a factor for the expected number of cases of failure $\bar{N}_{(-)}(T)$ and as an exponent for the survival probability $H_{(+)}(T)$.

It is possible, indeed, to proceed more complexly, considering several damage degrees (e.g., "non-structural" damage, partial failure, total collapse, as mentioned in Section D.3). In this case, the relation (D.4.1) would be related to the expected number of cases of exceedance of a certain damage threshold, D_k, while the relation (D.4.3) would be related to the probability of non-occurrence and non-exceedance of that damage degree. The number of cases of occurrence of damage degree D_k will be under these conditions (see Section D.3.3).

$$\bar{N}_k(T) = T \int_0^{\infty} s(q)\ p_k(q) dq = T\ I_k \tag{D.4.5}$$

where

$$I_k = \int_0^{\infty} s(q)\ p_k(q)\ dq \tag{D.4.6}$$

while the probability of non-occurrence and non-exceedance will be

$$H_k(T) = \exp\ - \sum_{k'}^{k'>k} \bar{N}_{k'}\ (T) \tag{D.4.7}$$

(where the assumption of Section D.3 that the damage degrees D_k increases monotonically with the index k is further on adopted). In case of low probabilities of occurrence or exceedance of damage degree D_k, their expression will be approximately

$$1 - H_k(T) \approx \sum_{k'}^{k'>k} \bar{N}_{k'}(T) \qquad (D.4.8)$$

D.4.3 Sensitivity of Specific Risk to Variation of Some Parameters

The expressions (D.4.1) and (D.4.5) of the basic characteristics of specific risk, $\bar{N}_{(-)}(T)$ and $\bar{N}_k(T)$ respectively, clarify the main factors on which these characteristics depend. A first conclusion is that the expected number of cases of failure is proportional to the expected number of earthquake occurrences (duration T times the specific frequency s(q) of the previous relation).

It is also interesting to examine the dependence of the specific risk characteristic on the safety factors characterizing a given situation (i.e., a given structure at a given site). Assuming that a type II extreme value distribution (D.2.16) is valid for a kinematic characteristic q (where q may be equal to EPA), Figure D-5 shows the dependence of the expected number of occurrences of seismic motions exceeding an intensity q, on the duration T and on the value q. Given the condition (D.2.14) and, hence, the relation

$$\bar{N}(q;T) = \frac{T}{\bar{T}(q)} \qquad (D.4.9)$$

and given the probability of $\bar{N}_{(-)}(T)$ to $\bar{N}(q;T)$ and also the relation (D.2.21'), the expected number of failures will satisfy the relation

$$\log \bar{N}_{(-)} = - \frac{\log \bar{T}_2 - \log \bar{T}_1}{\log q_2 - \log q_1} \qquad (D.4.10)$$

$$\log c \approx - (1.5...2)$$

$$\log c$$

This relationship is valid, of course, as far as the expression (D.2.16) holds, or, more precisely, as far as an expression of this kind is valid for the interval of values q that provides the major contribution to the integrals (D.4.2) or (D.4.6).

D.4.4 Specific Risk Related to Existing Structures

The relations of Section D.4.2 show the manner in which the hazard and vulnerability characteristics must be considered in order to obtain evaluations for the specific risk. The nature of basic data required for such calculations is hence obvious. Hazard data, represented basically by the specific frequency s(q), characterize a specific site, not a specific structure. Vulnerability data, represented basically by conditional probabilities like $F_{(-)}(q)$ or $p_k(q)$, depend directly on actual structural characteristics. Their nature and the degree of accuracy represent a major concern in cases when specific risk is to be evaluated. Available sources of information, noted in Section D.3 and possibly augmented by other sources too, are to be used to a full extent in order to provide realistic estimates of the vulnerability of a type of structure, and hence, of the specific risk affecting that type of structures at a given site.

D.5 ELEMENTS AT RISK, EXPOSURE AND POTENTIAL LOSSES

D.5.1 General

Potential earthquake-induced losses materialize with the presence of elements at risk, including people, properties, economic activities, and cultural values. An up-to-date risk evaluation thus requires satisfactory information on the nature of elements at risk, on the degree of exposure, and on the nature and likelihood of losses. The previous two sections (D.3 and D.4) are concerned basically with quantification of vulnerability and specific risk, given that some observable and quantifiable damage measure is used. These aspects are now followed by an analysis of elements at risk from the viewpoint of exposure and potential losses.

Two criteria may be used to classify elements at risk: exposure (i.e., amount of time and degree to which the elements at risk are present) and importance or sensitivity (i.e., magnitude of effects or losses if the elements at risk are actually affected).

To structure design combinations, it is possible to consider an exposure classification similar to the classification of actions. Some elements at risk, such as structures themselves, are permanently exposed. Some elements at risk such as people are intermittently exposed. Intermittent exposure varies with building use. The number of people exposed in a residential building varies to a moderate extent in the neighborhood of an average (or of the number of permanent residents of that building). The number of people exposed in an assembly hall varies greatly, from zero to the maximum possible during some meetings, performance, etc. Similar remarks may hold for cultural values or for material goods in an exhibition hall, etc.

From the viewpoint of importance or sensitivity, there again exist various possibilities. One can consider here quantifications such as the number of people exposed (maximum) and value of goods exposed.

D.5.2 Quantifications

Seismic risk evaluations require appropriate quantifications of the elements at risk. The basic topics to be dealt with in this context are:

a) identification of the elements at risk, E_1;

b) quantification on the degrees of exposure (e.g., the number of people at risk in an assembly hall at some definite time);

c) probability of affecting an element at risk, E_1, at a definite degree of exposure, at some definite time;

d) identification and quantification of potential losses (a loss that is proper to an element at risk E_1 will be denoted c_1);

e) probabilistic characterization of the expected losses, assuming a damage degree D_k to have occurred (conditional probability density: $g_{1/k}(c_1)$; conditional probability function, $G_{1/k}(c_1)$; expected conditional loss, $\bar{c}_{1/k}$);

f) such additional important factors as cross effects or correlations and potential chain effects.

The appropriate symbols and measures of deterministic and probabilistic nature for all entities listed under item (a) to (f) may, of course, be introduced. In any case, the final result should be an estimate of the quantities or functions listed under items (e), i.e., $g_{1/k}(c_1)$, $G_{1/k}(c_1)$ and $\bar{c}_{1/k}$ and an appropriate analysis of the aspects mentioned under item (f). It is assumed, later, that the outcome of such analyses is at hand.

For existing structures, one should consider the relatively high likelihood of complete destruction. For older structures, which are as a rule not engineered, an earthquake-induced collapse is a definite possibility. Special attention should therefore be given to the evaluation of loss characteristics like $g_{1/k}(c_1)$, $G_{1/k}(c_1)$ and $\bar{c}_{1/k}$, for the ultimate or collapse damage degree, $k = k_u$. These loss characteristics are of course also important when the seismic hazard is present in catastrophic forms (landslide, ground factors etc.).

D.6 EVALUATION OF RISK

D.6.1 General

The evaluation of seismic risk represents the main concern of this chapter. As defined in broad qualitative terms in Section D.1, the seismic risk represents a convolution of specific risk and of elements at risk (in particular, this convolution may be replaced by a product). In Section D.4, the specific risk was expressed in terms of probabilities of exceeding at least once during a certain period of time certain levels of observable damage. The elements at risk were dealt with in Section D.5 from a qualitative viewpoint and their quantifications were related to the conditional distributions of various losses, given that a certain degree of physical damage was undergone by a structure.

In the same way as in Sections D.2 and D.4, two kinds of exposure to seismic hazard must be considered: catastrophic and of moderate forms, respectively. Catastrophic forms were not explicitly dealt with in Section D.4, where attention was given mainly to moderate forms for which more complete relations, including the contribution of structural vulnerability, were developed. Attention is given in Section D.6 to both forms.

The seismic risk should be considered, essentially, in connection with a single future possible seismic event, given the argument outlined in Section D.4.1: strong seismic actions will affect in some way a structure exposed and possibly (and desirably) will lead to human intervention intended to reduce the risk implied by subsequent earthquakes. Still, some further quantifications of seismic risk are related to cumulative effects (cumulative losses over a definite period of time, number of cases of exceedance of a certain level of loss). These measures of risk, in spite of their purely formal character, are useful in cost-benefit analyses implied by the decision on intervention on existing buildings [8] which represent the object of Sections D.7 and D.8.

This section discusses

- some measures of risk (relative to both types of seismic hazard),
- the sensitivity of those measures with respect to parameters characterizing basic data,

- possible correlations and comparisons with use of code formats,
- specific features of risk as applied to existing buildings.

D.6.2 Measures and Evaluation of Seismic Risk

The evaluation of seismic risk implies a quantification both of eventual losses and of likelihood of their occurrence. Measures of losses are the same in Section D.5, i.e., a system of measures c_1 appropriate to the different elements at risk E_1. Measures of the likelihood of the losses, considered in this section, are:

a) expected cumulative losses for a definite period of time;

b) the expected number of cases of exceedance of a certain level of loss during a definite period of time;

c) the probability of non-exceedance of a certain level of loss during a definite period of time;

d) auxiliary measures, expressing the sensitivity of the previously mentioned measures with respect to such basic data as period of time, frequency of occurrence of earthquakes, and conventional safety factors.

The losses resulting from seismic damage may occur at different times. For some losses, it is reasonable to use discount factors. This approach is accepted at present for economic losses. The discounting function considered is an exponential,

$$z_1(t) = \exp(-a_1 t) \qquad \text{(D.6.1)}$$

where the origin of time should coincide with the initial moment (the moment of cost-benefit analysis required by eventual decision making), while the constant a_1 will be related to interest rates, rates of economic growth, etc. Note here that the integral of $z_1(t)$ is

$$Z_1(t) = \int_0^t z_1(t')dt' = \frac{1}{a_1}\left[1 - \exp(-a_1 t)\right] \qquad \text{(D.6.2)}$$

Although functions like $z_1(t)$ or $Z_1(t)$ may be used to discount economic losses, this approach cannot be applied to possible loss of lives or of cultural values. One cannot determine whether a loss of life is more or less serious a loss at present than twenty years hence. Functions $z_1(t)$ should therefore be evaluated separately for each component of possible losses.

A first measure of seismic risk, as referred to, is given by the expected cumulative losses for a period of time of duration T, $\bar{C}_1(T)$. The corresponding expression is

$$\bar{C}_1(T) = \int_0^T \sum_k \bar{c}_{1/k}\, I_k\, z_1(t)dt = \sum_k \bar{c}_{1/k}\, I_k\, Z_1(T) \qquad \text{(D.6.3)}$$

where I_k is given by the expression (D.4.6). A second measure of seismic risk is given by the expected number of cases of exceedance, $N_1(c_1;\, T)$, of a level of loss c_1, during a period of time of duration T. The corresponding expression is

$$\bar{N}_1(c_1;T) = \sum_k [1-G_{1/k}(c_1)]\,\bar{N}_k(T) = T\sum_k [1-G_{1/k}(c_1)]I_k \quad (D.6.4)$$

in case no discounting is considered. When discounting is used, a more general expression

$$\bar{N}_1(c_1;T) = \int_0^T \sum_k \{1 - G_{1/k}[c_1 z_1(t)]\}\, I_k dt \quad (D.6.5)$$

must be used. Finally, the probability of non-exceedance of c_1 will be

$$H_1(c_1;T) = \exp\,[-\bar{N}_1(c_1;T)] \quad (D.6.6)$$

The expressions (D.6.3) to (D.6.6) may be adapted to cases of catastrophic forms of seismic hazard. The specific basic elements in this case will be S_c (the expected number of occurrences of catastrophic seismic actions for a unit time interval) and the expected loss characteristics $\bar{c}_{1/k_u}$, $g_{1/k_u}(c_1)$ or $G_{1/k_u}(c_1)$, corresponding to the case of collapse, $k = k_u$.

The expression (D.6.3) will be replaced in this case by the expression

$$\bar{C}_1(T) = \int_0^T \bar{c}_{1/k_u}\, S_c\, z_1(t)dt = \bar{c}_{1/k_u}\, S_c\, Z_1\,(T) \quad (D.6.3')$$

The expressions (D.6.4) and (D.6.5) will be replaced by

$$\bar{N}_1(c_1;T) = T\,[1 - G_{1/k_u}(c_1)]\, S_c \quad (D.6.4')$$

and

$$\bar{N}_1(c_1;T) = \int_0^T 1-G_{1/k_u}\,[c_1 z_1(t)]\, S_c dt \quad (D.6.5')$$

while the non-exceedance probability $N_1(c_1;T)$ will be expressed by the relation (D.6.4).

D.6.3 Sensitivity of Risk Measures with Respect to the Variation of Basic Parameters

The relations given in this chapter exhibit how the following basic factors influence seismic risk: duration of service (T), frequency of occurrence of seismic events (s(q), etc.), structural vulnerability ($p_k(q)$), and eventual effects of damage on elements at risk ($c_{1/k}$, etc.). It is difficult to give some direct quantitative estimates for the risk, as expressed by the relations of Section D.6.2, given the lack of basic information as well as the diversity of situations as related to different countries, to different sites, and to different structural systems. It is easier, however, to analyze the sensitivity of risk measures with respect to the variation of the basic parameters previously mentioned.

The risk measures of cumulative nature, i.e., $\bar{C}_1(T)$ or $\bar{N}_1(c_1;T)$ depend directly on the duration T, which tends to appear as a factor. Still, discounting functions $z_1(t)$ may seriously alter this picture, especially in case of longer subsequent service duration. The characteristics of earthquake occurrence frequency $s(q)$ or S_c act as factors in the expressions of these cumulative measures. The consequences for the measure $H_1(c_1;T)$ that represents a non-exceedance probability may be easily deduced. Both service duration and frequency will influence the exponent of the non-exceedance probability. However, for the components of losses which are strongly affected by a discounting function $z_1(t)$, the role of service duration T is seriously reduced since the significant contribution of service duration is reduced to that of a few initial decades or even years.

The influence of vulnerability characteristics $p_k(q)$ on risk characteristics is somewhat more complex, since $p(q)$ represent a system of probabilities that must satisfy the corresponding normalization condition. The tendency of the values of the cumulative measures $\bar{C}_1(T)$ and $\bar{N}_1(c_1;T)$ to increase with increasing probabilities of heavy damage is nevertheless obvious. The consequences for the non-exceedance probability $H_1(c_1;T)$ are obvious too. The last factor referred to is represented by the expected losses. Here again, the proportionalities of $\bar{C}_1(T)$ with $\bar{c}_{1/k}$, and of $\bar{N}_1(c_1;T)$ with the exceedance probability $1 - G_{1/k}(c_1)$, are obvious.

The influence of the conventional safety factors is of the same nature as discussed in Section D.4.3 and expressed by the relation (D.4.10) for the range where the type II (maxima) extreme value distribution is valid for the kinematic parameter of ground motion, q.

D.6.4 Some Aspects Related to Existing Structures

The evaluation of seismic risk measures is basically a necessary step for the cost-benefit analysis required by decisions to intervene in existing structures (Sections D.7 and D.8). Risk measures must therefore be derived to fit the format used in the subsequent cost-benefit analysis. The expressions of Section D.6.2 permit alternative approaches. These expressions can be obviously generalized, but the most significant features of the seismic risk are nevertheless brought out by them.

The main practical problem in dealing with the existing building stock is represented by the search for the most suitable way to reduce the seismic risk in case the risk is unacceptably high. The main ways to reduce the seismic risk are:

a) reduction of the duration of subsequent service, T (set deadline for intervention);

b) reduction of vulnerability (adopt efficient upgrading measures and set a corresponding deadline);

c) reduction of the degree of exposure of elements at risk (e.g., change functions of a building, prohibit large groups from using unsafe assembly halls etc. and again set a corresponding deadline).

Since modification of hazard characteristics $s(q)$ or S_c is currently beyond human capabilities, these elements represent merely hard data of the problem.

Determining more detailed ways of reducing risk and the extent to which measures like those mentioned under items a, b and c may be used, represent tasks of decision making (see Section D.8).

Given the difficulties raised by the practical use of a more sophisticated approach, like that developed in the Sections D.3 to D.6, it is necessary to employ more direct means of seismic risk evaluation. This part is concerned to a considerable extent with topics that represent the objective of Chapter 5 of the manual, but offers at the same time a simple approach to some aspects related to the elements at risk as well as to risk itself.

D.7 COST AND BENEFIT COMPONENTS

D.7.1 General

Any solution adopted for a structure, whether it is the design of a new structure or the intervention on an existing structure, will imply some benefits resulting from the use of that structure, and some costs for construction work, maintenance activities, and various losses. The relationship between benefits and costs should be as favorable as possible. Pursuing this goal implies in principle cost-benefit analyses, as dealt with in Sections D.7 and D.8.

As mentioned in Sections D.5 and D.6, losses due to earthquakes, which imply a major contribution to the quantities to be considered in cost-benefit analyses, consist of several components such as economic losses and losses of human lives. However, it is obvious that each of the cost(loss)-benefit components consist of several terms due to different factors. The main goal of this section is to discuss briefly the components and terms of benefits and costs as well as to comment on the specific input data involved.

D.7.2 Components of Benefits and Costs

The benefits and the costs (losses) to be considered in cost-benefit analyses consist of several qualitatively different components. A first component, the most common, represents the economic benefits and costs. This component can be expressed in monetary terms. A second component represents the human lives affected (in a favorable or unfavorable sense) in relation to a structure. This component can be expressed in terms of human lives. It is possible to consider additional components, e.g., in relation to cultural values (of course, quantifications related to these raise difficult issues).

For subsequent cost-benefit analyses, some general rules should be adopted: definition of the units used for quantifying each of the components and use of a consistent sign convention (the benefits will be assigned positive values while the costs or losses will be assigned negative values).

D.7.3 Terms of Benefits and Costs

The analysis of benefits and costs or losses shows that several factors contribute to the various components referred to in Section D.7.2.

A first (positive) contribution is use of a structure. The benefits resulting from this use, which will be referred to also as gross utility, can be expressed in monetary terms (rent, value related to goods produced, etc.), in terms of lives (lives saved in a hospital, favorable influence on health of comfortable life conditions expressed in the same terms on the basis of equating the expected increase of length of life, etc.), and possibly in other terms. This term will be denoted in principle with B and the components corresponding to it will be denoted by B_1.

A second (negative) contribution results from the investment in constructing new structures or by the intervention into existing ones. This term will be denoted by C' (with components C_1'). Whereas the term B represents a contribution that is distributed along the service time of a structure, the term C' is practically concentrated at one, or a few, moments of time. The moment of intervention on existing structures will be denoted by t_i.

A next (negative) contribution consists of maintenance costs. This contribution denoted M (with components M_1) could be included in (i.e., subtracted from) the term B, or it can be kept separately. Maintenance costs are also distributed along the service time of a structure.

A last (negative) contribution results from various earthquake losses. These losses were denoted in Section D.6 by C (with components C_1). These losses are practically concentrated at the moments of occurrence of earthquakes (indirect losses due to an earthquake can be related in an appropriate manner to the moment of occurrence). These losses can be evaluated on the basis of risk analysis techniques, as presented in the previous sections. Whereas the other terms (B, C', M) may have some components equal to zero (the maintenance work may not affect human lives), the term C will contribute as a rule in case of destructive earthquakes to all components of benefits and losses.

D.7.4 The Net Utility

Net utility here means (U) the difference between the benefits (denoted B) and the various costs to investment, repair (C'), maintenance (M) and earthquake inflicted losses (C). Net utility may be written as

$$U = B - C' - M - C \tag{D.7.1}$$

This expression should be replaced, in a more accurate formulation, by a system of scalar relations considered for each of the components referred to in Section D.7.2.

$$U_1 = B_1 - C_1' - M_1 - C_1 \tag{D.7.2}$$

Each of the terms of the right member will depend on a set of arguments, such as time variables (service duration, moment of intervention), and variables characterizing the nature of eventual interventions.

The terms occurring in the right member are affected various degrees of randomness. Whereas the randomness of the terms B and M is low and the term C' can be analyzed even in a deterministic manner, the randomness of the last term, C, is especially high, owing to the various factors influencing it, as discussed in Section D.6. The net utility is therefore a random entity that must be dealt with using probabilistic concepts. A first approach in this regard is to consider expected values for each of the terms of the right member, which leads to consideration of the expected utility. Some additional discussion in this relation is presented in Section D.8.

D.8 CONSIDERATIONS ON DECISION MAKING

D.8.1 General

The decision as to whether or not an intervention on existing structure is necessary and, in the affirmative case, on the nature and size of intervention, may be made at different levels (governmental level, local level, level of the owner of a building, etc.). In any case, the decision maker should have at least a qualitative insight into the main factors that influence a rational decision. The costs of intervention should be weighed against the potential costs of the consequences from earthquakes that are likely to occur.

Approaches to decision making vary in sophistication. More sophisticated approaches should be adopted basically by highly competent institutes or engineers, when preparing the basis for decisions at a governmental level, which are intended to cover either an important existing building stock or particularly important individual cases. In this case, it is advisable to make use of all the developments of this appendix. Some data are given in this relation in Sections D.8.2 and especially D.8.3. In the opposite case, when a sophisticated approach is not justified (and this may refer to the large majority of buildings dealt with one-by-one), it is advisable to use the codes of current engineering practice, as well as the methods of evaluating the resistance of existing buildings, as given in Section 5 of this manual, and to consider also some amendments to the code requirements, as recommended in Section D.8.4.

D.8.2 Possible Decisions

Decision-making in connection with existing structures must cover basically two features of the problem:

a) their further use
b) the intervention

Decision making will be urgently needed after destructive earthquakes (as well as after other exceptional events such as bombing, explosions, as well as other natural hazard phenomena). This may be necessary also under different conditions where the time pressure is as a rule not so high. The functionality/use of a structure may need to be changed (as a rule in order to increase its utility); a project may be needed to provide a satisfactory degree of safety to a certain category of structures (e.g., the residential building stock of a given seismically hazardous area) and/or of general revision of land use, town planning, etc.

From the viewpoint of further use, it is possible to decide:

a) no change (further use as before);

b) some modification (immediate of use, discontinuation such as through evacuation for a particularly high risk; some decrease, aimed to reduce the elements at risk; some increase, aimed to lead to a higher utility).

From the viewpoint of intervention, it is possible to decide:

a) no intervention;

b) some intervention (immediate demolition, for a specially high risk without sufficient reasons for repair; some modifications aimed to reduce the risk, such as removing upper stories of a taller building; repair and/or strengthening aimed to reduce the risk in order to maintain the previous use, or to introduce some modifications aimed to increase the utility of the structure).

Each possible decision should be accurately defined from the qualitative viewpoint as well as from the quantitative one. Quantifications should refer to the characteristics of intervention, its timing, and the subsequent lifetime considered for a structure.

There exists, of course, a wide spectrum of possible decisions concerning both further use and intervention. Nevertheless, it should be borne in mind that:

a) when required a decision must be made, since postponing a decision is in fact a deciding to do nothing, and may be dramatically mistaken under certain circumstances;

b) any action relating to further use or intervention will involve some immediate cost, such as the cost of moving furniture and equipment or the cost of repair and strengthening work);

c) any action will involve as well some modifications in the costs and benefits related to the further use of a structure, maintenance cost, utility benefit, various costs of the effects of eventual subsequent earthquakes, etc.;

d) most structures represent components of some systems and decisions on individual structures will be in fact decisions on parts of some larger systems (e.g., on development of some town areas) which can be reasonably dealt with only in relation to the requirements of the systems involved;

e) as a rule, decision-makers are faced with considerable uncertainties related to the various aspects of physical phenomena (like seismic hazard or structural behavior) or of human activities (like existence of elements at risk or possible chains of events);

f) the conditions under which a decision is made today may considerably change owing to social evolution, to changes in life style, etc., such that latitude should be allowed for subsequent decisions to be adopted after several decades or even years.

D.8.3 Cost-Benefit-Analysis Based Decision Making

The basic data to be considered in the decision on intervention in an existing structure relate to:

a) seismic hazard
b) seismic vulnerability of the structure
c) costs of intervention
d) implications, or costs of eventual damage
e) maintenance costs
f) utility of service

The arguments on which the functions related to the basic data enumerated depend are:

a) the time variable, t

b) the timing of intervention, t_i

c) the characteristics of intervention (degree of strengthening, like amount of additional reinforcement etc.), represented by a vector $\underline{a}$

d) the various variables characterizing the entities listed previously, which become arguments on which the functions describing subsequent entities depend

The developments of previous sections show how the basic data referred to may be described in mathematical terms as well as how risk characteristics may be determined on the basis of corresponding basic data.

The ultimate goal of decision making should be that of maximizing the net utility dealt with in Section D.7.4. This may be done in principle, provided that one considers the randomness of various terms of the right members of expression (D.7.2). But the high degree of randomness of the terms C or C_1, representing the earthquake-induced losses that are likely to occur, deserve special attention. It is necessary here to consider the social sensitivity to concentrated losses, especially when large numbers of casualties are involved. It is therefore advisable not to carry out a decision process in which only expected values (in the probabilistic sense) are considered for the various components and terms. Rather, various possible solutions should be characterized by numbers related to the various terms involved in the expression (D.7.2). Since the most highly random term is by far C_1, it is reasonable to characterize it probabilistically by means of the various characteristics considered in Section D.6 (more precisely in the expressions (D.6.3) to (D.6.6)). These estimates will represent a useful basis for judgment by decision-makers.

The consideration of expression (D.7.2) leads to a multi-criterial attempt at optimization. It may be useful here to consider a weighting vector of (positive) components w_1 and to use an internal product

$$U = {}_{1}\ w_1\, U_1 \qquad \text{(D.8.1)}$$

The condition of maximum net utility would be in this case

$$U = \max \qquad \text{(D.8.2)}$$

The weighting vector could be used also for the analysis of other characteristics of the predicted losses, like those involved in relations (D.6.3) to (D.6.5).

Treating structures individually may be useful from the methodological viewpoint, but it will not be always relevant for the reality, where decision makers will be asked, as a rule, to deal with some ensembles that may be large (e.g., the residential building stock of a certain town). Under the latter conditions, it will be important, first of all, to define successive priorities of intervention (more precisely, a system of successive deadlines t_i) and to classify the existing building stock into categories to be dealt with by certain deadlines. The highest priorities will be assigned to structures with the highest probabilities of important negative U values for short durations of subsequent service as long as intervention does not take place (a second criterion for assigning priorities would be that of the efficiency of intervention, given by the vector of components C_1'). More specifically, analysts or decision makers could define and use a priority index (or a system of priority indices) on the basis of information summarized according to formats developed and accordingly classify the components of the existing building stock. The main arguments in favor of high priority would be negative values of the components $\bar{U}_1$ (assuming non-intervention), relatively high probabilities of events leading to high values of C_1, relatively high sensitivities of $\bar{U}$ with respect to the design characteristics, etc. It may also make sense to define ratios of the previously mentioned quantities to the number of people at risk, the built area at risk, etc.

It appears to be reasonable to develop decision making formats along following lines:

a) consider the variables of intervention t_i (intervention moment)

   T = (lifetime subsequent to the decision making moment)

   a_m = (characteristics of interventions such as reinforcement at some critical points, amount of concrete in some strengthening members or parts, etc.)

   and discretize the variables t_i, T, a_m, as well as the terms of U (D.7.1), (D.7.2).

b) consider, possibly, high and low variants of the (regularly increasing) term B-M.

c) for each couple (a_m, t_i), derive basic information according to the format described as follows.

d) for a couple (a_m, t_i), the basic information should include: for each T, the distribution of discretized vectors C_1, the expected vector $\bar{C}_1$ as well as characteristic of variance of C_1.

Some auxiliary elements of use for cost-benefit analysis are given in Section D.A.3. They refer on one hand to the relations between risk characteristics and to their dependence on lifetime and on the other hand to the dependence of the optimum safety factor (under certain simplifying assumptions) on some basic data of the cost-benefit analysis problems.

D.8.4 Amendments to Code Provisions

As mentioned in Section D.8.1, the practical approach to existing buildings will be, in most cases, similar to that of design activities related to new buildings. It is possible to recommend some amendments on the application of code provisions if one considers some features of the dependence of risk measures on the values of parameters used in engineering design. Assume that the extreme value distribution of type II (D.2.12), that leads to the expressions (D.2.16) etc. and consequently to the expression (D.4.10), is valid for the range of values most significant for the risk. Consider two different possible subsequent lifetimes, T_1 and T_2, and the design factors c_1 and c_2 that would be adopted in order to protect a structure for the two different lifetimes, respectively. In case the relation

$$\frac{T_2}{T_1} = \left(\frac{c_2}{c_1}\right)^{1.5...2} \qquad \text{(D.8.3)}$$

is satisfied (where the exponent is the same as the numerical factor in relation (D.4.10)), the failure probabilities are about the same for the two cases. Design factors may therefore be corrected using the relation

$$c_{corrected} = \frac{T_{service}}{T_{code}}^{1/2...1/1.5} \times C_{code} \qquad \text{(D.8.4)}$$

In this relation, T_{code} represents the reference lifetime of structures designed according to code provisions, c_{code} represents the design factor for structures of a given class designed according to standard code practice, $T_{service}$ represents the (shorter) subsequent lifetime considered for an existing structure, and $c_{corrected}$ represents the design factor for which that structure should be checked or designed.

D.9 ADDITIONAL COMMENTS

D.9.1 Uncertainties in Risk Evaluation

Risk evaluations are affected by considerable uncertainties, resulting from all categories of input data, i.e., hazard characteristics, vulnerability, and expected conditional losses. Compared to these uncertainties, computational difficulties are of secondary importance.

It is currently difficult to quantify the uncertainties referred to. The basic mathematical model of seismic hazard may prove to be inadequate. The assumptions on stationarity and on independence of occurrence of different seismic events may lead to erroneous estimates if the hazard characteristics are inferred from historical evidence only (historical information may be available for only a few centuries, while analysis of the seismicity of areas with historical information covering two to three millennia, like China and the Eastern Mediterranean, have shown periods of several centuries of higher activity alternating with periods of several centuries of lower activity). The observed vulnerability is affected by a high degree of scatter, as previously mentioned. The information that can be used for calibration of the data on the expected conditional losses is very poor to date too.

All the sources of uncertainty referred to must be considered in risk evaluation. A useful suggestion for overcoming the bias raised by this situation is to consider, in cases of high uncertainties, alternative hypotheses, and to perform sensitivity analyses of the risk characteristics with respect to the input data. This approach is recommended and feasible especially when seismic risk studies cover an important building stock and when these analyses occur under the auspices of government agencies in order to provide the necessary basis for land use planning, for actions during reconstruction, etc.

D.9.2 Additional Comments on Existing Structures

The importance of the task of providing a satisfactory degree of safety to the existing building stock has become increasingly obvious and even occasionally critical especially in regard to earthquake risk. The existence of an important building stock to be used for several subsequent decades, the evolution of earthquake protection standards which often render older code provisions unsatisfactory and deterioration and aging (corrosion, fatigue, etc.) represent more than sufficient reasons for giving this task a high priority. Post-earthquake conditions, when the existing building stock may be seriously affected, dramatically emphasizes this social need.

Destructive earthquakes have demonstrated the lack of appropriate resistance capacity of larger or smaller parts of the building stock affected. Appropriate pre-earthquake intervention would have had considerable mitigating effects. These facts underscore the importance of concern for existing structures.

Remedying existing structures is considerably more difficult than seismically constructing new ones. The education of engineers is currently oriented almost exclusively toward dealing with new structures. Code provisions or other regulations related to existing structures are scarce and incomplete if not totally absent. Even basic concepts are developed currently to a much lesser extent than those related to new construction, not to mention the important specific factors to be considered over and above those related to new construction.

D.10 CONCLUDING REMARKS

This appendix is devoted basically to the analysis of seismic risk for individual structures and the approach considers each structure in isolation. Given the appendix's goal of relating primarily to the activity of government agencies, a more sophisticated mathematical tool was used. This tool may be not well suited for everyday practice, but its use is in the range of possibilities of specialized institutes and engineers charged with the task of providing the required safety to larger building ensembles.

The applications of methodology and relations of this appendix must make use of the basic elements provided by Chapters 2 to 5.

Risk analyses performed for individual systems may prove to be insufficient when dealing with localities, lifelines, or other systems. In those cases, very significant interactions can occur between the different components of the built stock and chain effects can considerably magnify the primary effects resulting from damage to one individual structure.

The state-of-the-art, especially the uncertainties referred to in Section D.9.1, obviously raises the need for existence further research at a scale which obviously requires international cooperation.

D.A.1 MATHEMATICAL SYMBOLS USED IN APPENDIX D

1. Hazard

1.1 Parameters Related to One Motion (T is vibration period)

$S_a(T,n)$: response spectrum - absolute acceleration

$S_p(T,n)$: response spectrum - relative pseudo-velocity

$S_d(T,n)$: response spectrum - relative displacements

$\bar{S}_a$: spectral acceleration

$\bar{S}_p$: spectral pseudo-velocity

f_c: corner frequency

I: macroseismic intensity (MSK)

M: magnitude (Richter)

EPA: effective peak velocity

EPV: effective peak acceleration

PGA: peak ground acceleration

$S_a^o(T,n)$: normalized response spectrum-acceleration

k_s: ratio of EPA to gravity acceleration used in design

$\beta(T)$: dynamic factor used in design (correspondent of $S_a^o(T,n)$)

ϕ: reduction factor, to account for strength reserves and ductility

$c_s(T)$: design coefficient

1.2 Parameters Related to Sequences of Motions (T is service duration)

q: intensity parameter (as a rule proportional to amplitude of motion)

N(q;T): number of occurrences of motions with intensities exceeding q, during a period of time of duration T)

$\bar{N}(q;T)$: expected value of N(q;T)

s(q): density of expected number of occurrences of (moderate) motions of intensity q, for unit time interval

$\bar{T}(q)$: (average) return period of intensity q

$G_m(q;T)$: probability of m occurrences of motion of intensities exceeding q, for a time interval of duration T

$G_o(q;T)$: non-exceedance probability of the intensity q for the same time interval

S_c: expected number of cases of occurrence of catastrophic motions for unit time interval

2. Vulnerability

DD: damage degree

D_k: damage state of kth order

$f_k(q)$: conditional relative frequency of kth damage degree

$p_k(q)$: conditional (discrete) probability of kth damage degree

q_j: discretized q (see hazard parameters)

$f_{k/j}$: conditional relative frequency of kth damage degree

$p_{k/j}$: conditional (discrete) probability of kth damage degree

$F_{(-)}(q)$: conditional failure probability

3. Specific Risk

$\bar{N}_{(-)}(T)$: expected number of failures for a time interval of duration T

$I_{(-)}$: expected number of failures for unit time interval

$H_{(+)}(T)$: survival probability for a time interval of duration

$\bar{N}_k(T)$: expected number of occurrences of damage state D_k during a time interval of duration T

I_k: expected number of occurrences of damage state D_k during unit time interval

c: design safety factor

4. Elements at Risk

E_1: elements at risk

c_1: potential loss proper to element at risk E_1

$g_{1/k}(c_1)$: conditional probability density of c_1 (assuming D_k to have occurred)

$G_{1/k}(c_1)$: conditional probability function of c_1

$\bar{c}_{1/k}$: conditional expected value of c_1 (assuming D_k to have occurred)

$\bar{c}_{1/k_u}$: the same, assuming collapse to have occurred

5. Evaluation of Risk

$z_1(t)$: discounting function, proper to element at risk F_1 ($Z_1(T)$: its integral)

$\bar{C}_1(T)$: expected cumulative loss (related to element at risk E_1) for a time interval of duration T

$\bar{N}_1(c_1;T)$: expected number of cases of exceedance of loss c_1 for a time interval of duration T

$H_1(c_1;T)$: probability of non-exceedance of loss c_1 for a time interval of duration T

D.A.2 DAMAGE EVALUATION METHODOLOGY USED IN ROMANIA

The DD (damage degree) was evaluated in Romania during post-earthquake surveys on which subsequent derivation of vulnerability characteristics relied, as follows:

I. Masonry

Bearing Walls:

- not affected: 0
- slightly affected: 1 (for lack of specification, 0.5)
- cracked: 1.75
- strong cracking: 2.50
- out of vertical direction: 4
- collapsed: 5

Non-Bearing Masonry:

- not affected: 0 (for lack of specification: 0.5)
- cracked: 1
- partially collapsed: 2
- completely collapsed: 3

The DD assessed for a building was the maximum of the DD's assessed for bearing walls and for non-bearing masonry respectively.

II. R.C. Frames

Columns:

- not affected: 0.5
- cracked: 2
- strong cracking: 4
- crushed concrete: 4
- buckled reinforcement: 4

Beams:

- not affected: 0
- slightly affected: 1 (for lack of specification: 0.5)
- cracked: 2
- strong cracking: 3
- failed: 4

Infill Masonry:

- not affected: 0
- boundary cracks: 1
- cracking: 1.5
- strong cracking: 2
- dislocation: 2.5
- collapsed: 3.

The DD assessed for a building was the maximum of DD's assessed to columns and to infill masonry for pre-1950 buildings, and the maximum of DD's assessed to beams and to infill masonry for post-1950 buildings.

III. R.C. Bearing Walls

- not affected: 0
- slightly affected: 1 (for lack of specification: 0.5)
- cracked: 2
- strong cracking: 3
- failed: 4

The same values as in case of R.C. frames were used for infill masonry and for columns.

The DD assessed for a building was the maximum of DD's assessed for R.C. bearing walls and for infill masonry, respectively, for buildings with homogeneous layout or the maximum of DD's assessed for the former ones and for the columns for buildings with flexible first stories.

IV. Buildings Done of Low Quality Materials

Walls:

- not affected: 0
- slightly affected: 1 (for lack of specification: 0.5)
- cracked: 2
- strong cracking: 3
- collapsed: 5

The DD assessed for a building was the DD assessed for the walls.

D.A.3 AUXILIARY ELEMENTS FOR COST-BENEFIT ANALYSIS

D.A.3.1 The Dependence of Risk Characteristics on Lifetime

The relation between the survival probability $H_{(+)}(T)$ and the expected number of failure cases during a unit time interval, $I_{(-)}$, resulting from the expressions (D.4.1) to (D.4.4) may be rewritten as

$$H_{(+)}(T) = \exp\left[-T\ I_{(-)}\right] \tag{D.A.3.1a}$$

$$I_{(-)} = -\frac{1}{T} \ln H_{(+)}(T) = -\ \ln H_{(+)} \tag{D.A.3.1b}$$

It may be useful for effective calculations to consider also the logarithm of $I_{(-)}$,

$$I_{(-)} = \exp\left[-J\right] \tag{D.A.3.2a}$$

$$J = -\ \ln I_{(-)} \tag{D.A.3.2b}$$

The relation

$$\ln\left[TI_{(-)}\right] = \ln I_{(-)} + \ln T = -J + \ell nT \tag{D.A.3.3}$$

The use of the double natural cologarithm J in advantageous primarily owing to its dependence on the safety factor (or on the design acceleration). Some numerical calculations shown a dependence that is not far from a linear one,

$$J \approx J_o + c\ J_1 \tag{D.A.3.4}$$

where c is the safety factor and J_1 is positive. Unlike J, the parameter $I_{(-)}$ presents a tendency of uneven variation, with a sharp increase for low safety factors. Keeping in view these facts, the diagrams of Figure D-15 can be used in practice. The values along the vertical lines correspond to the relations (D.A.3.1) and (D.A.3.2) while the two-axes diagram corresponds to the relation (D.A.3.3).

D.A.3.2 Optimizing Safety Factors

The expression (D.7.1) is considered here in a much simplified form. Instead of the net utility one considers only an expected scalar "total" cost, $\bar{C}$, consisting of two terms, the investment cost C' and the expected cost of earthquake induced losses, $\bar{C}(T)$:

$$\bar{C}_t(T) = C' + \bar{C}(T) \tag{D.A.3.5}$$

Based on expressions (D.6.1) and (D.6.2), it is useful here to consider a reduced lifetime, T_e, obtained from the expression (D.6.2),

$$Z(T) = \frac{1}{a}\left[1 - \exp\left(-aT\right)\right] = T\,\frac{1 - \exp\left(-aT\right)}{aT} = T_e \tag{D.A.3.6}$$

In case one assumes an expression

$$C' \approx C_o' + C_o'\ c \tag{D.A.3.7}$$

and one rewrites the expression (D.6.3) in the simplified form

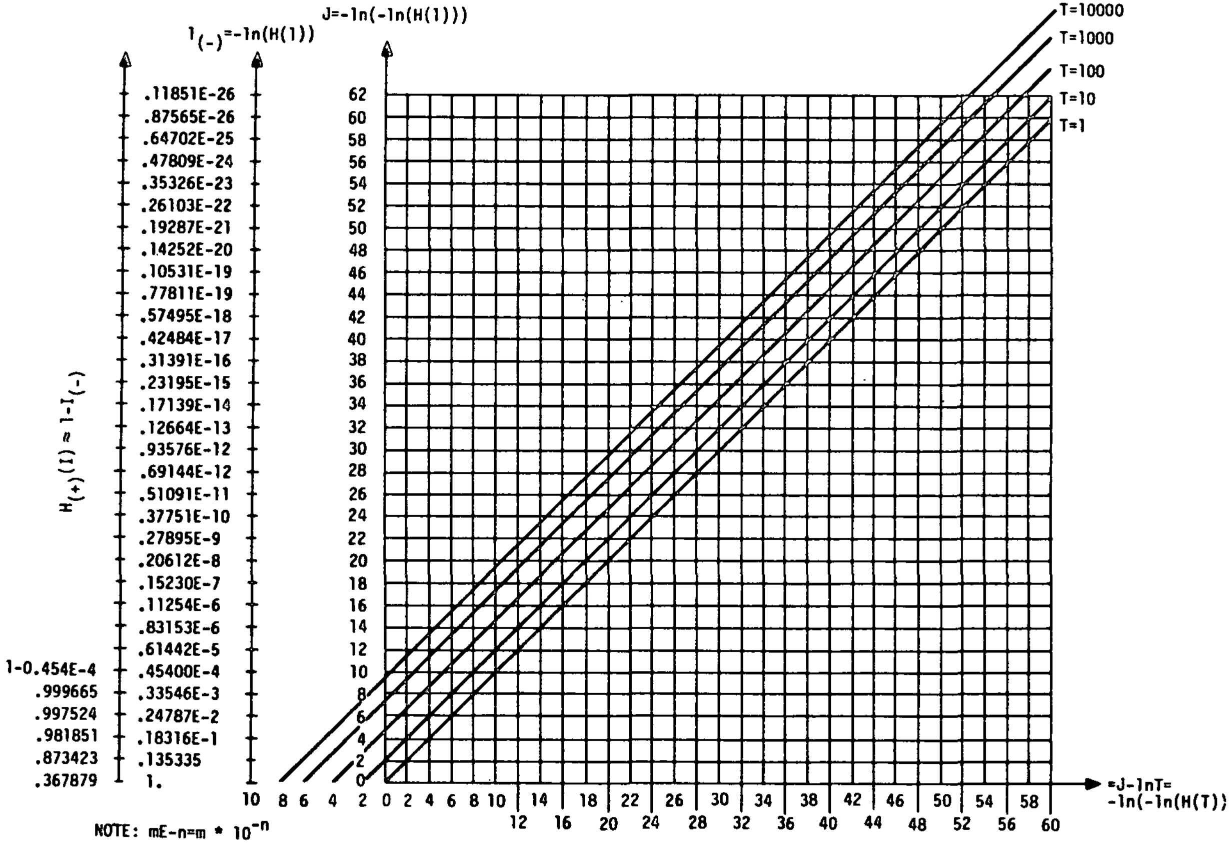

$H_{(+)}(I) \approx 1-I_{(-)}$
$I_{(-)}=-\ln(H(1))$
$J=-\ln(-\ln(H(1)))$
1-0.454E-4
.999665
.997524
.981851
.873423
.367879
.11851E-26
.87565E-26
.64702E-25
.47809E-24
.35326E-23
.26103E-22
.19287E-21
.14252E-20
.10531E-19
.77811E-19
.57495E-18
.42484E-17
.31391E-16
.23195E-15
.17139E-14
.12664E-13
.93576E-12
.69144E-12
.51091E-11
.37751E-10
.27895E-9
.20612E-8
.15230E-7
.11254E-6
.83153E-6
.61442E-5
.45400E-4
.33546E-3
.24787E-2
.18316E-1
.135335
1.
62 60 58 56 54 52 50 48 46 44 42 40 38 36 34 32 30 28 26 24 22 20 18 16 14 12 10 8 6 4 2 0
10 8 6 4 2 0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60
T=10000
T=1000
T=100
T=10
T=1
$=J-\ln T=$
$-\ln(-\ln(H(T)))$
NOTE: mE-n=m * 10^{-n}

Figure D-15.

$$\bar{C}(T) \approx \bar{C}_o \, I_{(-)} \, T_e \qquad \text{(D.A.3.8)}$$

where $\bar{C}_o$ represents the expected total loss due to a failure, the attempt to minimize the function (D.A.4.5), which leads to the condition

$$\frac{\partial \, \bar{C}_t(T)}{\partial c} = \frac{\partial C'}{\partial c} + \frac{\partial \, \bar{C}(T)}{\partial c} = 0 \qquad \text{(D.A.3.9)}$$

may be rewritten as

$$C_1' + \bar{C}_o \, T_e \, \frac{\partial \, I_{(-)}}{\partial c} = 0 \qquad \text{(D.A.3.10)}$$

Introducing the sensitivity parameter d,

$$d = \frac{\bar{C}_o \, \bar{T}_e}{C_1'} \qquad \text{(D.A.3.11)}$$

the condition (D.A.3.10) becomes

$$\frac{\partial \, I_{(-)}}{\partial c} = - \frac{1}{d} \qquad \text{(D.A.3.12)}$$

or, on the basis of (D.A.3.2),

$$I_{(-)} \, \frac{\partial J}{\partial c} = \frac{1}{d} \qquad \text{(D.A.3.13)}$$

The optimum value of the risk characteristic $I_{(-)}$ becomes on this basis

$$I_{(-)} = \frac{1}{d \, (\frac{\partial J}{\partial c})} = \frac{1}{dJ_1} \qquad \text{(D.A.3.14)}$$

in case the relationship (D.A.3.4) is considered.

Figure D-16 may be used in order to obtain the solution $I_{(-)}$ of (D.A.3.14).

D.A.4 ILLUSTRATIVE SEISMIC RISK EVALUATIONS

Some illustrative seismic risk evaluations are presented on the basis of concepts and relations given in this appendix. Two cases are examined -- an apartment house and an assembly hall. The basic data are related to

a) seismic hazard
b) vulnerability (expressed in damage degrees)
c) elements at risk, which here include building and people exposed

Seismic hazard data are assumed to be summarized in Figure D-5, in which the kinematic parameter in the abscissa is q = EPA. The design spectrum is assumed to be constant over the range of fundamental natural periods, either in the elastic range, or, as apparently increased, due to post-elastic deformation. Under these conditions, vulnerability may be simply expressed as related to a single parameter, q.

The vulnerability characteristics are related to six damage degrees, according to the developments presented in [9] by Whitman and Cornell. The damage degrees D_k are given in Table D-3.

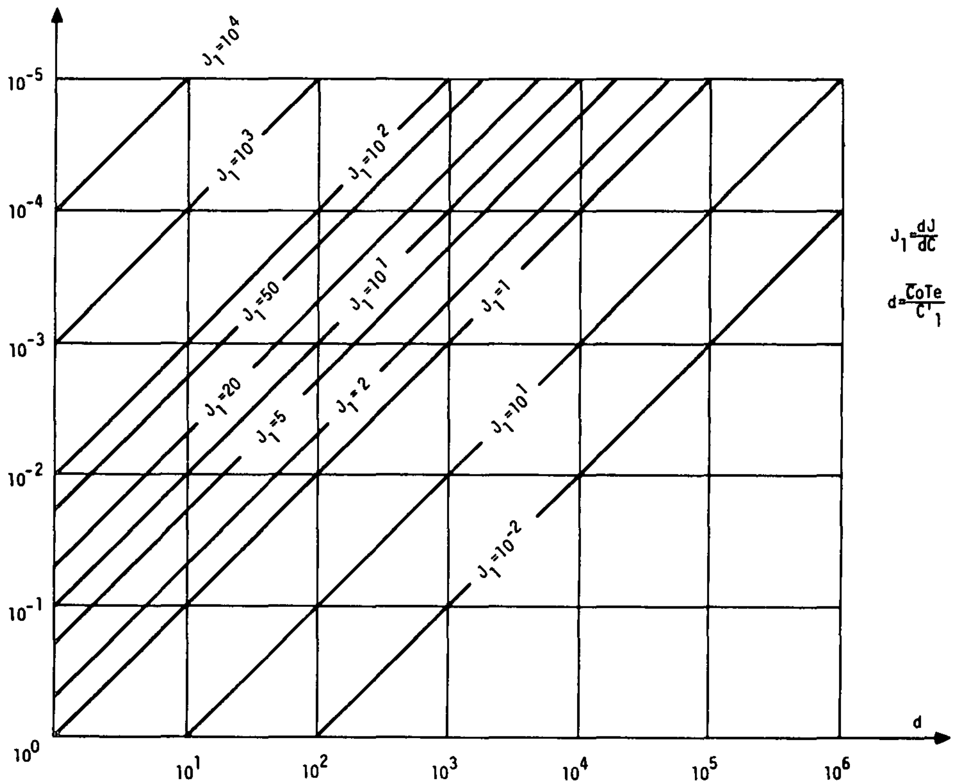


Figure D-16.

Table D-3. Damage Degrees According to [9]

SYMBOL	CHARACTERIZATION
NONE - O	NO, OR INSIGNIFICANT NON-STRUCTURAL DAMAGE
LIGHT - L OR 1	MINOR, LOCALIZED NON-STRUCTURAL DAMAGE
MODERATE - M OR 2	WIDESPREAD, EXTENSIVE NON-STRUCTURAL DAMAGE; READILY REPAIRABLE STRUCTURAL DAMAGE
HEAVY - H OR 3	MAJOR STRUCTURAL DAMAGE; POSSIBLY TOTAL NON-STRUCTURAL DAMAGE
TOTAL - T OR 4	BUILDING CONDEMNED OR REPLACED
COLLAPSE - C OR 5	BUILDING PARTIALLY OR TOTALLY COLLAPSED

These damage degrees parallel approximately the damage degrees considered by the MSK scale.

The vulnerability of both buildings is assumed to correspond to Table D-4, which basically agrees the vulnerability characteristics given in [9] for buildings designed according to the US-UBC 0 design strategy.

Table D-4. Vulnerability Characteristics Assumed (histograms expressed in %)

MSK INTENSITY I	VALUE OF q = EPA (m/s^2)	DAMAGE DEGREE					
		0	1(L)	2(M)	3(H)	4(T)	5(C)
V	0.25	100	0	0	0	0	0
V 1/2	0.35	70	30	0	0	0	0
VI	0.5	27	73	0	0	0	0
VI 1/2	0.7	20	65	15	0	0	0
VII	1.0	15	48	33	4	0	0
VII 1/2	1.4	0	21	45	29	5	0
VIII	2.0	0	0	20	41	34	5
VIII 1/2	2.8	0	0	0	20	65	15
IX	4.0	0	0	0	0	75	25
IX 1/2	5.6	0	0	0	0	50	50
X	8.0	0	0	0	0	25	75

The first building is assumed to be a five-stories with ten apartments, housing nominally 30 persons and having a total replacement value of US $400,000. The distribution of people present in the house corresponds to Figure D-17. Dotted lines replace with a good approximation the real discrete distribution.

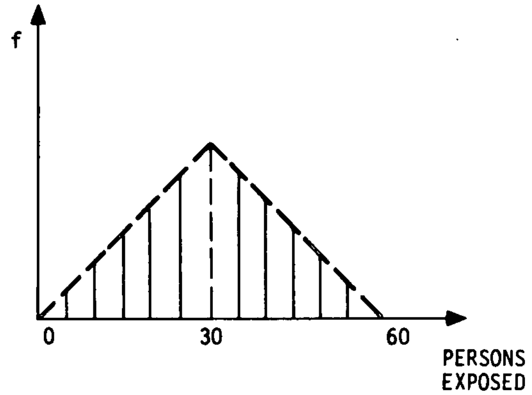
The second building is a one-story assembly hall housing a maximum of 1000 persons and having a replacement value of US $600,000. The distribution of people present in the building corresponds to Figure D-18.

Losses expressed as fractions of the replacement costs are such summarized in Table D-5, for any of the buildings.

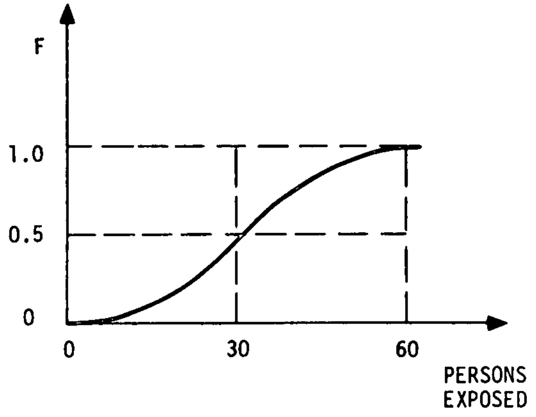
Table D-5. Expected Losses for Various Degrees of Damage

DAMAGE DEGREE	DAMAGE RATIO (2%)	
	RANGE	CENTRAL VALUE
0	0...0.05	0.01
1(L)	0.05...1.25	0.3
2(M)	1.25...20	5
3(H)	20...65	30
4(T)	65...100	95
5(C)	100	100

The losses distributed according to Table D-5 are assumed to correspond to a distribution in the range (a, b), with a central value c, such as in Figure D-19.

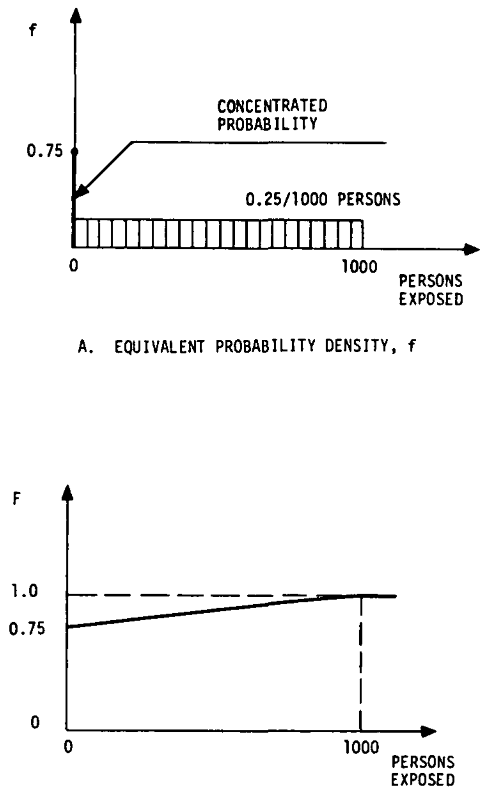


A. EQUIVALENT PROBABILITY DENSITY, f



B. EQUIVALENT PROBABILITY FUNCTION, f

Figure D-17. Exposure of People in an Apartment House (First Illustration)



A. EQUIVALENT PROBABILITY DENSITY, f

B. EQUIVALENT PROBABILITY FUNCTION, F

Figure D-18. Exposure of People in an Assembly Hall (Second Illustration)

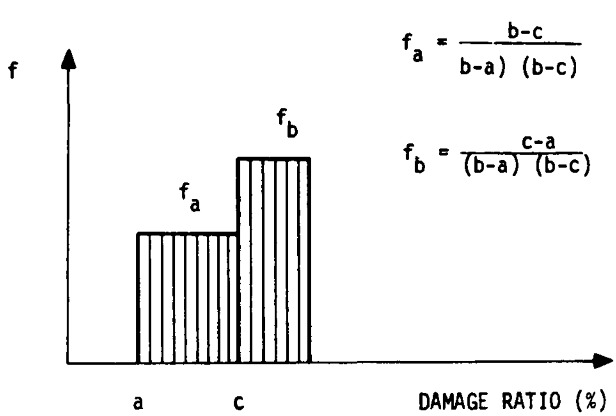


Figure D-19. Conventional Distribution Assumed to Fit Data of Table D-5

Central values of injury and life loss are presented in Table D-6 as a functions of the damage degree undergone by the buildings.

Table D-6. Injury and Loss of Life (Central Value %) as Functions of the Damage Degree

DAMAGE DEGREE	LOSSES (%)			
	CASE 1		CASE 2	
	INJURY (AT LEAST)	LOSS OF LIFE	INJURY (AT LEAST)	LOSS OF LIFE
0	0	0	0	0
1(L)	0	0	1	0
2(M)	1	0	5	1
3(H)	2	.25	20	5
4(T)	10	1	60	20
5(C)	100	20	100	60

The numbers for the first illustration are again in fair agreement with the data of [9].

For any risk evaluation, it is necessary to dispose of the values of the parameters I_k (D.4.6) while it is advantageous to replace the expression (D.4.6) as follows:

$$I_k = \int_0^\infty p_k\, s(q)dq = -\int_0^\infty p_k(q)\, d\bar{N}(q;1) = -\int_0^\infty p_k(T)\, d\bar{N}(I;1) \approx$$

$$\approx -\sum_j p_{k/j}\, \Delta\bar{N}_j \qquad \text{(D.A.4.1)}$$

It is possible to use either the kinematic parameter q, or the macroseismic intensity I as an argument in the integration. For the approximate discretized forces the sense of argument is no longer of interest. In case the expression of $\bar{N}(q;1)$ corresponds to Figure D-5, it may be written

$$\bar{N}(I;1) = 10^{(2-1/2)} \qquad \text{(D.A.4.2a)}$$

$$\bar{N}(q;1) = 10^{-\frac{1}{2}(3+\log_2 q)} \qquad \text{(D.A.4.2b)}$$

(where it is assumed that $q\ (m/s^2) = 2^{I-VII}$). The values $p_{k/j}$ needed for the numerical integration of (D.A.4.1) are given in Table D-4.

The values of $\bar{N}_j$ (i.e., $\bar{N}(q_j;1)$ or $\bar{N}(I_j;1)$) are given in Table D-7.

Table D-7. Values of $\bar{N}_j$

I(MSK)	$q(m/s^2)$	$\bar{N}_j$ (i.e., $\bar{N}(q_j;1)$ or $\bar{N}(I_j;1)$)
V	0.25	0.3162
V 1/2	0.35	0.1778
VI	0.5	0.1
VI 1/2	0.7	0.05623
VII	1.0	0.03162
VII 1/2	1.4	0.01778
VIII	2.0	0.01
VIII 1/2	2.8	0.005623
IX	4.0	0.003162
IX 1/2	5.6	0.001778
X	8.0	0.001

The numerical calculation of expressions (D.A.4.1) gives the values I_k in Table D-8.

Table D-8. Values of I_k (Expected Annual Number of Occurrences of Damage Degree D_k)

DAMAGE DEGREE	0	1	2	3	4	5
I_k	.210	.078	.0125	.0051	.0050	.00164

The sum of I_k is approximately equal to $\bar{N}(V;1)$. The difference results mainly from numerical errors. Given these results, the damage degree 2 (moderate) is expected to occur .0125 times in one year, .125 times in ten years etc., the damage degree 5 (collapse) is expected to occur .00164 times in one year, .164 times in 100 years etc. The expected number of cases of damage of degree 3 (heavy) or higher in one year is .0051 + .0050 + .00164 = .01174, and, for fifty years, .587. According to (D.2.14), the return period of damage of degree 3 or higher is $(.01174)^{-1}$ = 85.18 years. According to (D.2.20) or (D.4.7), the probability of non-occurrence of damage of degree 3 or worse in fifty years is exp (-50 years/85.18 years) = .556. The probability of satisfying the same condition for 10 years is

$$\exp(-10 \text{ years}/85.18 \text{ years}) = .889$$

In relation to expected losses, two categories must be considered: 1) material losses (on the basis of Table D-5) and 2) losses inflicted to people (on the basis of Table D-6) subdivided on the basis of Table D-6 into (2a) injury or loss of life and (2b) loss of life.

Material losses can be expressed in monetary terms, either as a fraction of replacement cost or in absolute values. Accepting the assumptions on which the developments Section D.6.2 relies (primarily, the perfect restoration of a structure affected by a seismic event), one can determine various measures of expected losses. According to (D.6.3), the expected loss in 50 years is

$$\bar{C}_1(50\text{ years}) = \sum_k \bar{c}_{1/k}\, I_k Z_1\ (50\text{ years}) =$$

$$= (.01\text{x}.210 + .3\text{x}.078 + 5\text{x}.0125 + 30\text{x}.0051 +$$

$$+ 95\text{x}.0050 + 100\text{x}.00164) \times Z_1\ (50\text{ years})/100 =$$

$$= .0088\ Z_1\ (50\text{ years})$$

If no discounting occurs, $a_1 = 0$ in expression (D.6.1), Z_1 (50 years) = 50 years, and $\bar{C}_1$ (50 years) = .44.

If the discount rate a_1 = 10%, then, according to (D.6.2),

$$Z_1\ (50\text{ years}) = [1 - \exp(-.1\text{x}50)]/.1 = 9.93\text{ years}$$

and the present value of loss is only

$$\bar{C}_1\ (50\text{ years}) = .087$$

Expressed in absolute terms, the non-discounted expected loss for the first illustration is

.44 x 400,000 US \$ = 176,000 US \$

while the discounted expected loss for the second illustration is

.087 x 600,000 US \$ = 52,000 US \$

The conditional probabilities of non-exceedance of some loss thresholds, $G_{1/k}(c_1)$, used in relations (D.6.4) and (D.6.5) are based on Table D-5 (second column) and distributions presented in Figure D-19, and are given in Table D-9.

Table D-9. Conditional Non-Exceedance Probabilities for Some Loss Thresholds

DAMAGE DEGREE	NON-EXCEEDANCE PROBABILITIES FOR THRESHOLDS OF LOSS	
	10%	30%
0	1.	1.
1(L)	1.	1.
2(M)	.8667	1.
3(H)	0	.5
4(T)	0	0
5(C)	0	0

The calculation of the conditional non-exceedance probability $G_{1/2}$ (10%), which is not immediate, was calculated based on assumptions in Figure D-19, as in Figure D-20.

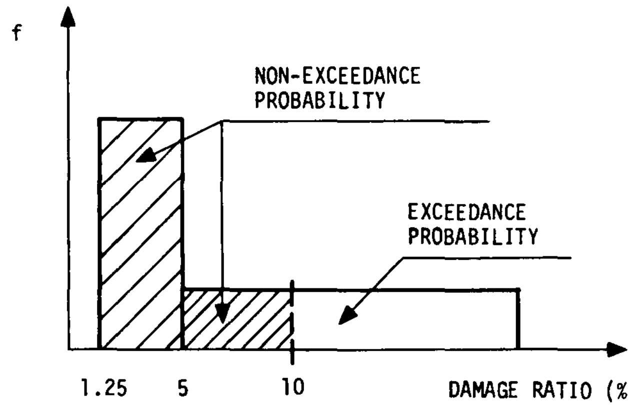


Figure D-20. Calculation of Probability of Non-Exceedance of 10% Loss for Moderate Damage, According to Table D-5

According to (D.6.4), the expected number of cases of exceedance of (non-discounted) loss of 10% in a unit time interval is

$$\bar{N}_1(10\%;1) = 0x.210 + 0x.078 + .1333x.0125 + $$
$$+ 1x.0051 + 1x.0050 + 1x.00164 = .0134$$

The same for a loss of 30% is

$$\bar{N}_1(30\%;1) = 0x.210 + 0x.078 + 0x.0125 + $$
$$+ .5x.0051 + 1x.0050 + 1x.00164 = .00919$$

According to (D.6.6), the probability of non-exceedance of a loss of 10% in 10 years is

$$H_1(10\%;10 \text{ years}) = \exp(-.0134x10) = .8746$$

The same probability for a loss of 30% and 50 years is

$$H_1(30\%;50 \text{ years}) = \exp(-.00919x50) = .6316$$

The expected number of injured persons, $\bar{C}_{2a}(T)$, or of killed persons, $\bar{C}_{2b}(T)$, can be approximately calculated, considering the exposure characteristics of Figures D-17 and D-18 and treating the data on Table D-6 as deterministic values (more accurate evaluations should convolute in an appropriate manner exposure characteristics as in figures referred to with distributions replacing the data of Table D-6). Based on (D.6.3) and derived from Table D-6, the expected number of annually injured persons (%) in the first illustration (apartment building) is

$$\bar{C}_{2a}(1) = (0x.210 + 0x.078 + 1x.0125 + 2x.0051 + 10x.0050 + $$
$$+ 100x.00164)\ \% = .2367\ \%$$

A summary of risk to persons is given in Table D-10, for each illustration. According to Figure D-17, the expected number of exposed persons in the first illustration (apartment building) is 30 and according to Figure D-18, that number for the second illustration (hall) is

$$.75 \times 0 + (25 \times (0 + 1000)/2) = 125$$

The value for the case of collapse (last number) was calculated as follows: the conditional probability of non-exceedance of 60 victims is the same as the probability of exposure of no more than 100 persons, since the probability of loss of life in case of collapse is assumed to be 60% according to the table. The probability of not more than 100 persons exposed is the sum of two terms: .75 (concentrated probability of empty hall) plus 0.1x.25 (10% of the probability of persons being present, due to the constancy of probability density in the interval (0, 1000 persons) in Figure D-20).

According to (D.6.4), the annual expected number of cases of exceedance of this threshold is

$$\bar{N}_{2b}(60 \text{ k.p.};1) = (1 - .825) \times .005 + (1 - .775) \times .00164 =$$

$$= .001244$$

The probability of non-exceedance of the threshold of 60 killed persons during one earthquake for a twenty years period is

$$H_{2b}(60 \text{ k.p.};20 \text{ years}) = \exp(-20 \text{x}.001244) = .9754$$

Table D-10 shows the high risk for people involved by the assembly hall, given the assumptions made (the exposure according to Figure D-18 and conditional expected numbers of Table D-6).

Table D-10. Annual Expected Number of Injured and of Killed Persons

NATURE OF LOSSES	CASE			
	1. APARTMENT BUILDING		2. ASSEMBLY HALL	
	INJURY OR LOSS OF LIFE	LOSS OF LIFE	INJURY OR LOSS OF LIFE	LOSS OF LIFE
RELATIVE (%)	.24	.039	.71	.24
ABSOLUTE (PERSONS)	.072	.012	.89	.3

(Note here that killed persons are also counted as injured.)

Another measure of risk, connected with the expressions (D.6.4) and (D.6.6), pertains to exceedance of a certain number of victims following one earthquake. The calculations are done for the second illustration (assembly hall). The threshold of 60 killed persons is considered. The conditional probabilities of non-exceedance $G_{2b/k}(c_{2b})$ are determined from the distribution of Figure D-18 and the data on Table D-6 (last column). For the six damage degrees in order, the conditional probabilities are

1., 1., 1., 1., .825, .775

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A P P E N D I X E

APPENDIX E

EXAMPLE OF ORDINANCE FOR EARTHQUAKE RISK REDUCTION

Ordinance No. 154,807, EARTHQUAKE HAZARD REDUCTION IN EXISTING BUILDINGS enforced on January 7, 1981 by the City of Los Angeles Division 68 is reproduced below as an example of existing legislation for earthquake risk reduction at the level of a large community.

Ordinance No. 154,807

An ordinance adding Division 68 of Article 1 of Chapter IX of the Los Angeles Municipal Code relative to earthquake hazard reduction in existing buildings.

Section 1. Article 1 of Chapter IX of the Los Angeles Municipal Code is hereby amended to add a Division 68 to read:

DIVISION 68 — EARTHQUAKE HAZARD REDUCTION IN EXISTING BUILDINGS

SEC. 91.6801. PURPOSE.

The purpose of this Division is to promote public safety and welfare by reducing the risk of death or injury that may result from the effects of earthquakes on unreinforced masonry bearing wall buildings constructed before 1934. Such buildings have been widely recognized for their sustaining of life hazardous damage as a result of partial or complete collapse during past moderate to strong earthquakes.

The provisions of this Division are minimum standards for structural seismic resistance established primarily to reduce the risk of life loss or injury and will not necessarily prevent loss of life or injury or prevent earthquake damage to an existing building which complies with these standards. This Division shall not require existing electrical, plumbing, mechanical or fire safety systems to be altered unless they constitute a hazard to life or property.

This Division provides systematic procedures and standards for identification and classification of unreinforced masonry bearing wall buildings based on their present use. Priorities, time periods and standards are also established under which these buildings are required to be structurally analyzed and anchored. Where the analysis determines deficiencies, this Division requires the building to be strengthened or demolished.

Portions of the State Historical Building Code (SHBC) established under Part 8, Title 24 of the California Administrative Code are included in this Division.

SEC. 91.6802. SCOPE.

The provisions of this Division shall apply to all bullings constructed or under construction prior to October 6, 1933, or for which a building permit was issued prior to October 6, 1933, which on the effective date of this ordinance have unreinforced masonry bearing walls as defined herein.

EXCEPTION: This Division shall not apply to detached one or two story family dwellings and detached apartment houses containing less than five dwelling units and used solely for residential purposes.

SEC. 91.6803. DEFINITIONS:

For purposes of this Division, the applicable definitions in Sections 91.2301 and 91.2305 of this Code and the following shall apply:

Essential Building. Any building housing a hospital or other medical facility having surgery or emergency treatment areas, fire or police stations, municipal government disaster operation and communication centers.

High Risk Building. Any building, not classified an essential building, having an occupant load as determined by Section 91.3301(d) of this Code of 100 occupants or more.

EXCEPTION: A high risk building shall not include the following:

1. Any building having exterior walls braced with masonry crosswalls or wood frame crosswalls spaced less than 40 feet apart in each story.

2. Any building used for its intended purpose, as determined by the Department, for less than 20 hours per week.

Historical Building. Any building designated as an historical building by an appropriate Federal, State or City jurisdiction.

Low Risk Building. Any building, not classified an essential building, having an occupant load as determined by Section 91.3301(d) of less than 20 occupants.

Medium Risk Building. Any building, not classified as a high risk building or an essential building, having an occupant load as determined by Section 91.3301(d) of 20 occupants or more.

Unreinforced Masonry Bearing Wall. A masonry wall having all of the following characteristics:

1. Provides the vertical support for a floor or roof.
2. The total superimposed load is over 100 pounds per linear foot.
3. The area of reinforcing steel is less than 50 percent of that required by Section 91.2418(e) of this Code.

SEC. 91.6804. RATING CLASSIFICATIONS.

The rating classifications as exhibited in Table No. 68-A are hereby established and each building within the scope of this Division shall be placed in one such rating classification by the Department. The total occupant load of the entire building as determined by Section 91.3301(d) shall be used to determine the rating classification.

EXCEPTION: For the purpose of this Division, portions of buildings constructed to act independently when resisting seismic forces may be placed in separate rating classifications.

TABLE NO. 68-A
RATING CLASSIFICATIONS

Type of Building	Classification
Essential Building	I
High Risk Building	II
Medium Risk Building	III
Low Risk Building	IV

SEC. 91.6805. GENERAL REQUIREMENTS.

The owner of each building within the scope of this Division shall cause a structural analysis to be made of the building by a civil or structural engineer or arthitect licensed by the State of California, and if the building does not meet the minimum earthquake standards specified in this Division, the owner shall cause it to be structurally altered to conform to such standards, or cause the building to be demolished.

The owner of a building within the scope of this Division shall comply with the requirements set forth above by submitting to the Department for review within the stated time limits:

a. Within 270 days after the service of the order, a structural analysis. Such analysis which is subject to approval by the Department shall demonstrate that the building meets the minimum requirements of this Division; or

b. Within 270 days after the service of the order, the structural analysis and plans for the proposed structural alterations of the building necessary to comply to the minimum requirements of this Division; or

c. Within 120 days after service of the order, plans for the installation of wall anchors in accordance with the requirements specified in Section 91.6808(c); or

d. Within 270 days after the service of the order, plans for the demolition of the building.

After plans are submitted and approved by the Department, the owner shall obtain a building permit, commence and complete the required construction or demolition within the time limits set forth in No. Table 68-B. These time limits shall begin to run from the date the order is served in accordance with Section 91.6806(a) and (b).

TABLE NO. 68-B
TIME LIMITS FOR COMPLIANCE

Required Action By Owner	Obtain Building Permit Within	Commence Construction Within	Complete Construction Within
Complete Structural Alterations or Building Demolition	1 year	180 days*	3 years
Wall Anchor Installation	180 days	270 days	1 year

*Measured from date of building permit issuence.

Owners electing to comply with Item c of this Section are also required to comply with Items b or d of this Section provided, however, that the 270-day period provided for in such Items b and d and the time limits for obtaining a building permit, commencing construction and completing construction for complete structural alterations or building demolition set forth in Table No. 68-B shall be extended in accordance with Table No. 68-C. Each such extended time limit, except the time limit for commencing construction shall begin to run from the date the order is served in accordance with Section 91.6806(b). The time limit for commencing construction shall commence to run from the date the building permit is issued.

TABLE NO. 68-C
EXTENSIONS OF TIME AND SERVICE PRIORITIES

Rating Classification	Occupant Load	Extension of Time if Wall Anchors are Installed	Minimum Time Periods for Service of Order
I (Highest Priority)	Any	1 year	0
II	100 or more	3 years	90 days
III	100 or more	5 years	1 year
	More than 50, but less than 100	6 years	2 years
	More than 19, but less than 51	6 years	3 years
IV (Lowest Priority)	Less than 20	7 years	4 years

SEC. 91.6806. ADMINISTRATION.

(a) Service of Order. The Department shall issue an order, as provided in Section 91.6806(b), to the owner of each building within the scope of this Division in accordance with the minimum time periods for service of such orders set forth in Table No. 68-C. The minimum time period for the service of such orders shall be measured from the effective date of this Division. The Department shall upon receipt of a written request from the owner, order a building to comply with this Division prior to the normal service date for such building set forth in this Section.

(b) Contents of Order. The order shall be written and shall be served either personally or by certified or registered mail upon the owner as shown on the last equalized assessment, and upon the person, if any, in apparent charge or control of the building. The order shall specify that the building has been determined by the Department to be within the scope of this Division and, therefore, is required to meet the minimum seismic standards of this Division. The order shall specify the rating classification of the building and shall be accompanied by a copy of Section 91.6805 which sets forth the owner's alternatives and time limits for compliance.

(c) Appeal From Order. The owner or person in charge or control of the building may appeal the Department's initial determination that the building is within the scope of this Division to the Board of Building and Safety Commissioners. Such appeal shall be filed with the Board within 60 days from the service date of the order described in Section 91.6806(b). Any such appeal shall be decided by the Board no later than 60 days after the date that the appeal is filed. Such appeal shall be made in writing upon appropriate forms provided therefor, by the Department and the grounds thereof shall be stated clearly and concisely. Each appeal shall be accompanied by a filing fee as set forth in Table 4-A of Section 98.0403 of the Los Angeles Municipal Code.

Appeals or requests for slight modifications from any other determinations, orders or actions by the Department pursuant to this Division, shall be made in accordance with the procedures established in Section 98.0403.

(d) Recordation. At the time that the Department serves the aforementioned order, the Superintendent of Building shall file with the Office of the County Recorder a certificate stating that the subject building is within the scope of Division 68 — Earthquake Hazard Reduction in Existing Buildings — of the Los Angeles Municipal Code. The certificate shall also state that the owner thereof has been ordered to structurally analyze the building and to structurally alter or demolish it where compliance with Division 68 is not exhibited.

If the building is either demolished, found not to be within the scope of this Division, or is structurally capable of resisting minimum seismic forces required by this Division as a result of structural alterations or an analysis, the Superintendent of Building shall file with the Office of the County Recorder a certificate terminating the status of the subject building as being classified within the scope of Division 68 — Earthquake Hazard Reduction in Existing Buildings of the Los Angeles Municipal Code.

(e) Enforcement. If the owner or other person in charge or control of the subject building fails to comply with any order issued by the Department pursuant to this Division within any of the time limits set forth in Section 91.6805, the Superintendent of Building shall order that the entire building be vacated and that the building remain vacated until such order has been complied with. If compliance with such order has not been accomplished within 90 days after the date the building has been ordered vacated or such additional time as may have been granted by the Board and the Superintendent may order its demolition in accordance with the provisions of Section 91.0103(a) of this Code.

SEC. 91.6807. HISTORICAL BUILDINGS.

(a) General. The standards and procedures established by this Division shall apply in all respects to an historical building except that as a means to preserve original architectural elements and facilitate restoration, an historical building may, in addition, comply with the special provisions set forth in this Section.

(b) Unburned Clay Masonry or Abode. Existing or re-erected walls of abode construction shall conform to the following:

1. Unreinforced abode masonry wall shall not exceed a height or length to thickness ratio of 5 for exterior bearing walls and must be provided with a reinforced bond beam at the top interconnecting all walls. Minimum beam depth shall be 6 inches and a minimum width

Figure E-1. Los Angeles Muncipal Code: Ordinance No. 154,807; City of Los Angeles - Division 68, Earthquake Hazard Reduction in Existing Buildings. January 7, 1981, (Page 1 of 3) of 8 inches less than the wall width. Minimum wall thickness shall be 18 inches for exterior bearing walls and 10 inches for abode partitions. No abode structure shall exceed one story in height unless the historic evidence indicates a two story height. In such cases the height to thickness ratio shall be the same as above for the first floor based on the total two story height and the second floor wall thickness shall not exceed the ratio 5 by more than 20 percent. Bond beams shall be provided at the roof and second floor levels.

2. Foundation footings shall be reinforced concrete under newly reconstructed walls and shall be 50 percent wider than the wall above, soil conditions permitting, except that the foundation wall may be 4 inches less in width than the wall above if a rock, burned brick, or stabilized adobe facing is necessary to provide authenticity.

3. New or existing unstabilized brick and abode brick masonry shall test to 75 percent of the compressive strength as set forth in Section 91.2405(f) of this Code. Unstabilized brick may be used where existing bricks are unstabilized and where the building is not susceptible to flooding conditions or direct exposure. Abode may be allowed a maximum value of 3 pounds per square inch for shear with no increase for lateral forces.

4. Mortar may be of the same soil composition and stabilization as the brick in lieu of cement mortar.

5. Nominal tension stresses due to seismic forces normal to the wall may be neglected if the wall meets thickness requirements and shear values allowed by this subsection.

(c) Archaic Materials. Allowable stresses for archaic materials not specified in this Code shall be based on substantiating research data or engineering judgment subject to the Department's satisfaction.

(d) Alternative materials and SHBC Advisory Review. Alternative materials, design or methods of construction will be considered as set forth in Section 91.6809(d). In addition, when a request for an alternative proposed design, material or method of construction is being considered, the Department may file written request for opinion to the State Hitorical Building Code Advisory Board for its consideration, advice or findings in accordance with the SHBC.

SEC. 91.6808. ANALYSIS AND DESIGN

(a) General. Every structure within the scope of this Division shall be analyzed and constructed to resist minimum total lateral seismic forces assumed to act noncurrently in the direction of each of the main axes of the structure in accordance with the following equation.

$$V = IKCSW \qquad (68\text{-}1)$$

The value of IKCS need not exceed the values set forth in Table No. 68-D based on the applicable rating classification of the building.

TABLE NO. 68-D
HORIZONTAL FORCE FACTORS BASED ON RATING CLASSIFICATION

Rating Classification	IKCS
I	0.186
II	0.133
III and IV	0.100

(b) Lateral Forces on Elements of Structures. Parts or parts of structures shall be analyzed and designed for lateral loads in accordance with Section 91.2305(d) of this Code but not less than the value from the following equation

$$F_p = IC_pSW_p \qquad (68\text{-}2)$$

For the provisions of this subsection the produce of IS need not exceed the values as set forth in Table No. 68-E.

EXCEPTION: Unreinforced masonry walls in buildings not having a rating classification of I may be analyzed in accordance with Section 91.6809.

TABLE NO. 68-E
HORIZONTAL FORCE FACTORS "IS"
FOR PARTS OR PORTIONS OF STRUCTURES

Rating Classification	IS
I	1.50
II	1.00
III and IV	0.75

(c) Anchorage and Interconnection. Anchorage and interconnection of all parts, portions and elements of the structure shall be analyzed and designed for lateral forces in accordance with Table No. 23-B of this Code and the equation $F_p = IC_pSW_p$ as modified by Table No. 68-E. Minimum anchorage of masonry walls to each floor or roof shall resist a minimum force of 200 pounds per linear foot acting normal to the wall at the level of the floor or roof.

(d) Level of Required Repair. Alterations and repairs required to meet the provisions of this Division shall comply with all other applicable requirements of this Code unless specifically provided for in this Division.

(e) Required Analysis.

1. General. Except as modified herein, the analysis and design relating to the structural alteration of existing structures within the scope of this Division shall be in accordance with the analysis specified in Division 23 of this Code.

2. Continuous Stress Path. A complete, continuous stress path from every part or portion of the structure to the ground shall be provided for the required horizontal forces.

3. Positive Connections. All parts, portions or elements of the structure shall be interconnected by positive means.

(f) Analysis Procedure.

1. General. Stresses in materials and existing construction utilized to transfer seismic forces from the ground to parts or portions of the structure shall conform to those permitted by the Code and those materials and types of construction specified in Section 91.6809.

2. Connections. Materials and connectors used for interconnection of parts and portions of the structure shall conform to the Code.

3. Unreinforced Masonry Walls. Unreinforced masonry walls shall be analyzed as specified in Section 91.2417 to withstand all vertical loads as specified in Division 23 of this Code in addition to the seismic forces required by this Division. Such walls shall meet the minimum requirements set forth in Sections 91.2418 and 91.2419 of this Code. The 50 percent increase in the seismic force factor for shear walls as specified in Table No. 24-H of this Code may be omitted in the computation of seismic loads to existing shear walls.

No allowable tension stress will be permitted in unreinforced masonry walls. Walls not capable of resisting the required design forces specified in this Division shall be strengthened or shall be removed and replaced.

EXCEPTIONS:

1. Unreinforced masonry walls in buildings not classified as Rating I pursuant to Table No. 68-A may be analyzed in accordance with Section 91.6809.

2. Unreinforced masonry walls which carry no design loads other than its own weight may be considered as veneer if they are adequately anchored to new supporting elements.

(g) Combination of Vertical and Seismic Forces.

1. New Materials. All new materials introduced into the structure to meet the requirements of this Section which are subjected to combined vertical and horizontal forces shall comply with Section 91.2305(k) of this Code.

2. Existing Materials. When the stress in existing lateral force resisting elements are due to a combination of dead loads plus live loads plus seismic loads, the allowable working stress specified in the Code may be increased 100 percent. However, no increase will be permitted in the stresses allowed in Section 91.6809 and the stresses in members due only to seismic and dead loads shall not exceed the values permitted by Section 91.2301(g) of this Code.

3. Allowable Reduction of Bending Stress by Vertical Load. In calculating tensile fiber stress due to seismic forces required by this Division, the maximum tensile fiber stress may be reduced by the full direct stress due to vertical dead loads.

SEC. 91.6809. MATERIALS OF CONSTRUCTION

(a) General. All materials permitted by this Code including their appropriate allowable stresses and those existing configurations of materials specified herein may be utilized to meet the requirements of this Division.

(b) Existing Materials.

1. Unreinforced Masonry Walls. Unreinforced masonry walls analyzed in accordance with this Section may provide vertical support for roof and floor construction and resistance to lateral loads. The bonding of such walls shall be as specified in Section 91.2412(b) of this Code.

Tension stresses due to seismic forces normal to the wall may be neglected if the wall does not exceed the height or length to thickness ratio and the in-plane shear stresses due to seismic loads as set forth in Table No. 68-F.

TABLE NO. 68-F
ALLOWABLE VALUE OF UNREINFORCED
MASONRY WALLS WITH MINIMUM QUALITY MORTAR (1)

Rating Classification	Maximum Ratio Unsupported Height or Length to Thickness	Seismic In-Plane Shear Stress Based on Gross Area
I	Not applicable (2)	Not applicable (2)
II	9	3 psi (3)
III	10	3 psi (3)
IV	12	3 psi (3)

NOTES:

(1) Minimum quality mortar shall be determined by laboratory testing in accordance with Section 91.6809(e).

(2) Walls of buildings within rating classification I shall be analyzed in accordance with Section 91.6808(f).

(3) Allowable shear stress may be increased in accordance with Section 91.6809(g).

The wall height or length may be measured horizontally to supporting elements provided the stiffness of the supporting member is at least twice as stiff as the tributary wall. Stiffness shall be based on the gross section.

2. Existing Roof, Floors, Walls, Footings, and Wood Framing. Existing materials including wood shear walls utilized in the described configuration may be used as part of the lateral load resisting system provided that the stresses in these materials do not exceed the values shown in Table No. 68-G.

TABLE NO. 68-G
VALUES FOR EXISTING MATERIALS

Materials or Configuration of materials (1)	Allowable Values
1. Horizontal Diaphragms	
a. Roofs with straight sheathing and roofing applied directly to the sheathing.	150 lbs. per foot for seismic shear.
b. Roofs with diagonal sheathing and roofing applied directly to the sheathing.	400 lbs. per foot for seismic shear.
c. Floors with straight tongue and groove sheathing.	150 lbs. per foot for seismic shear.
d. Floors with straight sheathing and finished wood flooring.	300 lbs. per foot for seismic shear.
e. Floors with diagonal sheathing and finished wood flooring.	600 lbs. per foot for seismic shear.
f. Floors or roofs with straight sheathing and plaster applied to the joist or rafters. (2)	Add 50 lbs. per foot to the allowable values for Items 1a and 1c.
2. Shear Walls	
a. Wood stud walls with wood lath and plaster.	50 lbs. per foot each side for seismic shear.
b. Wood stud walls with plaster and lath other than wood lath.	100 lbs. per foot each side for seismic shear.
3. Plain Concrete Footings	f_c = 1,500 psi unless otherwise shown by tests
4. Douglas Fir Wood	Allowable stress same as No. 1 D.F.
5. Reinforcing Steel	f_t = 20,000 lbs. per square inch maximum
6. Structural Steel	f_t = 20,000 lbs. per square inch maximum.

NOTES

(1) Material must be sound and in good condition.

(2) The wood lath and plaster must be reattached to existing joists or rafters in a manner approved by the Department.

(c) Strengthening of Existing Materials. New materials including wood shear walls may be utilized to strengthen portions of the existing seismic resisting system in the described configurations provided that the stresses do not exceed the values shown in Table No. 68-H.

Figure E-1. Los Angeles Muncipal Code: Ordinance No. 154,807; City of Los Angeles - Division 68, Earthquake Hazard Reduction of Existing Buildings. January 7, 1981, (Page 2 of 3)

TABLE H
ALLOWABLE VALUES OF NEW MATERIALS USED
IN CONJUNCTION WITH EXISTING CONSTRUCTION

New Materials or Configuration of Materials	Allowable Values
1. Horizontal Diaphragms	
Plywood sheathing applied directly over existing straight sheathing with ends of plywood sheets bearing on joists or rafters and edges of plywood located on center of individual sheathing boards.	Same as specified in Table No. 25-J of this Code for blocked diaphragms.
2. Shear Walls	
a. Plywood sheathing applied directly over existing wood studs. No value shall be given to plywood applied over existing plaster or wood sheathing.	Same as values specified in Table No. 25-J for shear walls.
b. Dry wall or plaster applied directly over existing wood studs.	75 percent of the values specified in Table No. 25-N.
c. Dry wall or plaster applied to plywood sheathing over existing wood studs.	33-1/3 percent of the values specified in Table No. 25-N.
3. Shear Bolts	
Shear bolts and shear dowels embedded a minimum of 8 inches into unreinforced masonry walls. Bolt centered in a 2-1/2 inch diameter hole with dry-pack or non-shrink grout around circumference of bolt or dowel. (1)	100 percent of the values for plain masonry specified in Table No. 24-F. No values larger than those given for 3/4 inch bolts shall be used.
4. Tension Bolts	
Tension bolts and tension dowels extending entirely through unreinforced masonry walls secured with bearing plates on far side of wall with at least 30 sq. inches of area. (2)	1,200 lbs. per bolt or dowel.
5. Infilled Walls	
Reinforced masonry infilled openings in existing unreinforced masonry walls with dowels to match reinforcing.	Same as values specified for unreinforced masonry walls.
6. Reinforced Masonry	
Masonry piers and walls reinforced per Section 91.2418 and designed for tributary loads.	Same as values specified in Table No. 24-G
7. Reinforced Concrete	
Concrete footings, walls and piers reinforced as specified in Division 26 and designed for tributary loads.	Same as values specified in Division 26 of this Code
8. Existing Foundation Pressure	
Foundation pressures for structures exhibiting no evidence of settlement.	Calculated existing foundation pressures due to maximum dead load plus live load may be increased 25 percent for dead load, and may be increased 50 percent for dead load plus seismic load required by this Division.

NOTES

(1) Bolts and dowels shall be tested as specified in Section 91.6809(f)

(2) Bolts and dowels shall be 1/2 inch minimum in diameter.

(d) Alternate Materials. Alternate materials, designs and methods of construction may be approved by the Department in accordance with the provisions of Division 5 of Article 8 of Chapter IX of the Los Angeles Municipal Code

(e) Minimum Acceptable Quality of Existing Unreinforced Masonry Walls

1 General Provisions. All unreinforced masonry walls utilized to carry vertical loads and seismic forces parallel and perpendicular to the wall plane shall be tested as specified in this Subsection. All masonry quality shall equal or exceed the minimum standards established herein or shall be removed and replaced by new materials. The quality of mortar in all masonry walls shall be determined by performing in-place shear tests or by testing eight inch diameter cores. Alternative methods of testing may be approved by the Department. Nothing shall prevent pointing with cement mortar of all masonry wall joints before the tests are first made. If the exterior joints are pointed then the inside face must also be pointed. Prior to any pointing the wall surface must be sand or water blasted to remove loose and deteriorated mortar. All preparation and cement mortar pointing shall be done under the continuous inspection of a Registered Deputy Building Inspector. At the conclusion of the inspection, the inspector shall submit a written report to the licensed engineer or architect responsible for the seismic analysis of the building setting forth the result of the work inspected. Such report shall be submitted to the Department for approval as part of the structural analysis. All testing shall be performed in accordance with the requirements specified in this Subsection by a testing agency approved by the Department. An accurate record of all such tests and their location in the building shall be recorded and these results shall be submitted to the Department for approval as part of the structural analysis

2 Number and Location of Tests. The minimum number of tests shall be two per wall or line of wall elements resisting a common force, or 1 per 1,500 square feet of wall surface with a minimum of eight tests in any case. The exact test or core location shall be determined at the building site by the licensed engineer or architect responsible for the seismic analysis of the subject building

3 In Place Shear Tests. The bed joints of the outer wythe of the masonry shall be tested in shear by laterally displacing a single brick relative to the adjacent bricks in the wythe. The opposite head joint of the brick to be tested shall be removed and cleaned prior to testing. The minimum quality mortar in 80 percent of the shear tests shall not be less than the total of 30 psi plus the axial stress in the wall at the point of the test. The shear stress shall be based on the gross area of both bed joints and shall be that at which movement of the brick is first observed

4. Core Tests. A minimum number of mortar test specimens equal to the number of required cores shall be prepared from the cores and tested as specified herein. The mortar joint of the outer wythe of the masonry core shall be tested in shear by placing the circular core section in a compression testing machine with the mortar bed joint rotated 15 degrees from the axis of the applied load. The mortar joint tested in shear shall have an average ultimate stress of 20 psi based on the gross area. The average shall be obtained from the total number of cores made. If test specimens cannot be made from cores taken then the shear value shall be reported as zero

(f) Testing of Shear Bolts. One fourth of all new shear bolts and dowels embedded in unreinforced masonry walls shall be tested by a Registered Deputy Building Inspector using a torque calibrated wrench to the following minimum torques.

1/2" diameter bolts or dowels 40 foot-lbs
5/8" diameter bolts or dowels 50 foot lbs
3/4" diameter bolts or dowels 60 foot lbs

No bolts exceeding 3/4" shall be used. All nuts shall be installed over malleable iron or plate washers when bearing on wood and heavy cut washers when bearing on steel

g Determination of Allowable Stresses for Design Methods Based on Test Results

1 Design Shear Values. Design seismic in-plane shear stresses greater than permitted in Table No 68 F shall be substantiated by tests performed as specified in Section 91.6809(e) 3 and 4

Design stresses shall be related to test results obtained in accordance with Table No 68 I. Intermediate values between 3 and 5 psi may be interpolated

TABLE NO. 68-I
ALLOWABLE SHEAR STRESS FOR
TESTED UNREINFORCED MASONRY WALLS

Eighty Percent of Test Results in psi Not Less Than	Average Test Results of Cores in psi	Seismic In-Plane Shear Based on Gross Area
30 plus axial stress	20	3 psi*
40 plus axial stress	27	4 psi*
50 plus axial stress or more	33 or more	5 psi*

*Allowable shear stress may be increased by addition of 10 percent of the axial stress due to the weight of the wall directly above

2 Design Compression and Tension Values. Compression stresses for unreinforced masonry having a minimum design shear value of 3 psi shall not exceed 100 psi. Design tension values for unreinforced masonry shall not be permitted

SEC 91 6810 INFORMATION RQUIRED ON PLANS

(a) General. In addition to the seismic analysis required elsewhere in this Division the licensed engineer or architect responsible for the seismic analysis of the building shall determine and record the information required by this Section on the approved plans

(b) Construction Details. The following construction details shall be made part of the approved plans

1. All unreinforced masonry walls shall be anchored to all floors and roofs with tension bolts through the wall or by exsting rod anchors at the maximum anchor spacing of six feet. All existing rod anchors shall be secured to joists or rafters by bolting to develop the required forces. The Department may require testing by an approved testing agency to verify adequacy embedded ends of existing rod anchors

2 Diaphragm chord stresses of horizontal diaphragms shall be developed in existing materials or by addition of new materials

3 Where wood roof or floor members other than rafters or joists are supported in masonry pockets, ledgers or columns shall be installed to support vertical loads of the roof or floor members

4. Parapets and exterior wall appendages not capable of resisting the forces specified in this Division shall be removed, stabilized or braced to insure that the parapets and appendages remain in their original position

5. All deteriorated mortar joints in unreinforced masonry walls shall be pointed with cement motar. Prior to any pointing, the wall surface must be sand or water blasted to remove loose and deteriorated mortar. All preparation and pointing shall be done under the continuous inspection of a Registered Deputy Building Inspector certified to inspect masonry or concrete. At the conclusion of the project, the inspector shall submit a written report to the Department setting forth the portion of work inspected

6. Repair details of any cracked or damaged unreinforced masonry wall required to resist forces specified in this Division

(c) Existing Construction. The following existing construction information shall be made part of the approved plans

1. The appropriate age of building

2. The typical footing width, depth and maximum soil bearing for dead plus live loads

3. The type and dimensions of existing walls and the size and spacing of floor and roof members

4 The extent and type of existing wall anchorage to floors and roof.

5 The extent and type of parapet corrections which were performed in accordance with Section 91 0103(b) of this Code

6. Accurately dimensioned floor plans and masonry wall elevations showing dimensioned openings, piers, wall thickness and heights

7 The location of cracks or damaged portions of unreinforced masonry walls requiring repairs

8 The type of interior wall surfaces and if reinstalling or anchoring of ceiling plaster is necessary

9 The general condition of the mortar joints and if the joints need pointing

Sec 2 The City Clerk shall certify to the passage of this ordinance and cause the same to be published in some daily newspaper printed and published in the City of Los Angeles

I hereby certify that the foregoing ordinance introduced at the meeting of the Council of the City of Los Angeles of December 16 1980 and was passed at its meeting of January 7, 1981

REX E LAYTON
City Clerk
By Charles J Port, Deputy

Approved January 7, 1981

JOEL WACHS
Acting Mayor

File No 73 721, 74 4395 and 79 4244
(JD268411) Jan 13

Figure E-1. Los Angeles Muncipal Code: Ordinance No. 154,807; City of Los Angeles - Division 68, Earthquake Hazard Reduction in Existing Buildings. January 7, 1981, (Page 3 of 3)