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Seismic Design Decision Analysis

Report No. 9

SUMMARY OF METHODOLOGY AND PILOT APPLICATION

by

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John Brennan III, C. Allin Cornell
Richard de Neufville, Erik H. Vanmarcke

October, 1973

DEPARTMENT
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Cambridge, Massachusetts 02139

Paper presented to American Society of Civil Engineers
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This paper describes a procedure called Seismic Design Decision Analysis (SDDA). While the procedure potentially has a broad range of application, this paper focuses specifically upon building code requirements. To illustrate the procedure, a specific design situation was selected; the lateral force requirements for five- to twenty-story apartment buildings in Boston. While both steel and concrete design were considered in the study, this paper discusses the results for reinforced concrete buildings. The general floor and structural layout assumed for the study are illustrated. Shear walls were used to resist lateral forces in the transverse direction while longitudinal forces were resisted by moment resisting frames in the exterior walls. All designs have to resist the wind loading required by the Boston Building Code: 20 psf. Drift requirements under both wind (1/600) and earthquake (1/300) were considered as well as permissible stresses. Masonry block walls were assumed for the exterior walls and interior partitions in accordance with usual practice in Boston. Five different design strategies were considered. Four of these are the requirements for seismic zones, 0, 1, 2 and 3 of the Uniform Building Code, 1970 edition. These requirements differ in the magnitude of the lateral forces required in design and also in requirements concerning reinforcement. The fifth design strategy, designated as superzone S, required lateral forces twice as large as for zone 3. The study was to ascertain which design strategy would be most appropriate.

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PREFACE

This is the ninth in a series of reports covering work supported by the National Foundation under Grants GK-27955 and GI-29936, through the offices of Dr. Michael P Gaus and Dr. Charles C. Thiel. All of the authors except Mr. Brennan are members of the faculty of Civil Engineering at M.I.T.; Mr. Brennan is a principal of Le Messurier Associates. Many others have contributed to the study: two former M.I.T. staff members: Dr. J.W. Reed, now with John A. Blume & Associates Research Division in Las Vegas, and Dr. S.-T. Hong, now with Amoco Production Company in Tulsa; several engineering firms: Le Messurier Associates of Cambridge, Mass.; S.B. Barnes & Associates of Los Angeles; J.H. Wiggins Co. of Redondo Beach , Calif.; Ayres, Cohen and Hayakawa of Los Angeles; and Weston Geophysical Engineers, Inc. of Weston, Mass.; the Building Owners and Managers Association (BOMA) of Los Angeles; and numerous students at M.I.T.

The paper reproduced in this report is, in effect, a shortened version of Report 10 scheduled for release at the end of 1973. The development of the damage probability matrices is described in detail in Report 8.

PREVIOUS REPORTS

1. Whitman, R.V., C.A. Cornell, E.H. Vanmarcke, and J.W. Reed, "Methodology and Initial Damage Statistics," Department of Civil Engineering Research Report R72-17, M.I.T., March, 1972.

2. Leslie, S.K., and J.M. Biggs, "Earthquake Code Evolution and the Effect of Seismic Design on the Cost of Buildings," Department of Civil Engineering Research Report R72-20, M.I.T., May, 1972.

3. Anagnostopoulos, S.A., "Non-Linear Dynamic Response and Ductility Requirements of Building Structures Subjected to Earthquakes," Department of Civil Engineering Research Report R72-54, M.I.T., September, 1972.

4. Biggs, J.M., and P.H. Grace, "Seismic Response of Buildings Designed by Code for Different Earthquake Intensities," Department of Civil Engineering Research Report R73-7, M.I.T., January, 1973.

5. Czarnecki, R.M., "Earthquake Damage to Tall Buildings," Department of Civil Engineering Research Report R73-8, M.I.T., January, 1973.

6. Trudeau, P.J., "The Shear Wave Velocity of Boston Blue Clay," Department of Civil Engineering Research Report R73-12, M.I.T., February, 1973.

7. Whitman, R.V., S. Hong, J.W. Reed, "Damage Statistics for High-Rise Buildings in the Vicinity of the San Fernando Earthquake," Department of Civil Engineering Research Report R73-24, M.I.T., April, 1973.

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TABLE OF CONTENTS

INTRODUCTION

It is generally agreed that a building should be designed so as (a) not to collapse during a very large, rare earthquake, and (b) not to incur significant damage from earthquakes which can be expected to take place during the lifetime of the building.

While both of these principles are widely accepted as a basis for seismic design, it is difficult to be precise in the implementation of these principles. The second principle clearly implies a balancing of the risk of future loss against the initial cost of providing a stronger building. Even the first principle implies some balancing of risk, since the phrase "very large, rare earthquake" hardly provides a precise design specification.

The earthquake design requirements developed for use in building codes in California have represented a very serious attempt to implement these principles. In developing these codes, engineers used the then available facts so as to recommend a reasonable balance between increased initial cost and risk of future loss, although seldom has the balance been stated in an explicit way.

Recently, the adequacy and appropriateness of these codes has come under questioning. Following the 1971 San Fernando earthquake, many people in California have suggested that more severe design requirements should be adopted, at least for hospitals and other important public buildings. On the other hand, there has been considerable local resistance against national pressures to increase earthquake design requirements in eastern parts of the country. Certainly it makes little sense, as suggested in the 1970 edition of the Uniform Building Code, to require the same level of seismic resistance for buildings in Boston as in Los Angeles. However, it is not immediately clear whether the requirements for Boston should be decreased or whether those for Los Angeles should be increased.

In order to respond satisfactorily to such questions concerning code requirements, it is necessary to have a more explicit procedure for balancing cost and risk. The overall problem has many diverse aspects, and the interrelationships among these aspects generally are quite complicated. Hence, it is essential to have an organized systematic framework for assembling the available facts and for expressing the complex interrelationships. It also is essential to provide clear statements of the costs and risks that are to be balanced.

This paper describes such a procedure, called Seismic Design Decision Analysis (SDDA)*. While the procedure potentially has a broad range of application, this paper focuses specifically upon building code requirements. To illustrate the procedure, a pilot application is presented involving buildings of moderate height in Boston. The aim of the paper is primarily to present and illustrate the procedure; however, some tentative conclusions concerning design requirements applicable to Boston are indicated. A more complete presentation will appear in a forthcoming research report.

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*The abbreviations used in this paper are listed in Appendix A.

THE METHODOLOGY

Figure 1 outlines the methodology by means of a flow diagram. The heart of the methodology is examination, in probabilistic terms, of the damage which one earthquake will cause to a particular building system designed according to a particular design strategy. This evaluation is repeated for different levels of earthquakes, different design strategies and, where appropriate, for different types of buildings. For each different design strategy, the initial cost required by that strategy is combined with the losses from future earthquakes.

In simplest terms, a particular building system might be defined, for example, as: all buildings having 8 to 13 stories. In a more refined study, a building system might be defined as 8- to 13- story reinforced concrete buildings with ductile moment resisting frames. Other building systems are then defined by different ranges of stories, different construction materials and different lateral force resisting systems.

The simplest statement of a design strategy is: design in accordance with the Uniform Building Code (UBC) for earthquake zone 2 (or 0 or 1 or 3). More refined variations on the design strategies may also be considered, such as requirments concerning ductility, allowable drift, mechanical equipment, etc. The initial cost premium is a function of the design strategy. This cost may be expressed, for example, as the extra cost to design for zone 2 requirements as compared to making no provision for earthquake resistance.

One key step is determing the seismic risk. This is the probability that a ground motion of some stated intensity will occur during, say, one year, at the site of interest. Intensity may be expressed by the modified Mercalli scale, or better yet, by the spectral acceleration for the fundamental dynamic response period of the building system.

The effect of various levels of ground motion upon the building system is expressed by a family of damage probability matrices. Each matrix applies to a particular building system and design strategy, and gives the probability that various levels of damage will result from earthquakes of various intensities. By combining seismic risk with the information in the damage probability matrix, the probability that the building will receive various levels of damage may be determined. The expected future repair costs may then be determined.

For each damage state, there is an incident loss. Such incident losses include loss of function or loss of time during repairs and, in extreme cases, injury and loss of life and impact on community. In general, not all of the incident losses can readily be expressed in dollars.

If it were possible to express all losses in dollars, then the criterion for selecting the optimal design strategy would be minimum present total expected cost. That is to say, the design strategy would be selected that minimizes the sum of initial cost plus the discounted value of expected future losses. Actually, since future losses can be only partly expressed in dollars, alternate criteria for decision making must be considered.

Finally, any such methodology can only provide systematic and rational information concerning risks and benefits; where building codes are concerned, public bodies must still make the final decision concerning the proper balance between these conflicting considerations. The proposed methodology can never (and should never) be a total substitute for judgment and experience, but rather provide for a systematic organization of such experience and judgment. A major benefit of the methodology is to force specific consideration of the many factors. As indicated in Figure 1, there is a need for continuous feedback. As facts are assembled and considered by decision makers, the need for new and different information is revealed. Seismic risk and damage probability must be analyzed from new viewpoints and different forms of design strategy must be considered.

Criteria for Decision Making

The various steps indicated in Figure 1 will be discussed in more detail in the course of the illustrative pilot application that follows. However, it is necessary to say more at the outset concerning the criteria that may be used in judging the proper balance between cost and risk. With the danger of some over-simplification, three approaches to making this judgment may be identified:

1. Cost/benefit analysis
2. Risk of death
3. Multi-attribute (or multi-objective) decision theory.

None of these approaches provides an answer that will satisfy all people, but each has potential advantages and provides some insight.

In cost/benefit analysis, which has been used for many types of studies for many years, all losses -- including fatalities, injuries and social costs -- are expressed in monetary units. This means, in particular, that a monetary value must be assigned to human life, and various methods for arriving at this value have been proposed. Application of cost/benefit analysis also requires a decision as to the value of losses that may occur well into the future as compared to the value of costs incurred "now" during construction of a building; this decision generally takes the form of a choice of a discount rate. With such assumptions, the results of a cost/benefit analysis may typically be graphed as shown in Figure 2. Increasingly stringent design requirements mean greater initial costs but decreased expected future losses. The point of minimum total cost defines the optimum design strategy.

There are many difficulties in the practical use of cost/benefit analysis. Many people find it very difficult to accept the notion of placing any sort of value on human life. Yet today communities that impose earthquake design requirements already make such a judgment implicitly. For example, these communities are in effect deciding that it is better to make the owner of a new building pay extra for added resistance to earthquakes instead of contributing the same sum toward a transit system that will reduce highway deaths. It can effectively be argued that cost/benefit analysis, with consistant values assigned to human life and other social costs, may properly be used to choose from among various ways of spending fixed total resources to alleviate the risk of death and suffering.

As an alternative to placing a monetary value on human life, Starr (1969) evaluated the <u>risk of death</u> from various causes. These risks can be grouped into two general categories: those associated with voluntary activities and those associated with involuntary activities.

In the case of "voluntary" activities, an individual uses his own value system to evaluate his experience, and adjusts his exposure to risk accordingly. Generally, the evaluation and adjustment is done subconsciously. Any individual will evaluate different risks, such as death from crime in the city and death from automobile accidents during commuting, differently. However, there is a general consistency in the average risk associated with accidents of various kinds, and these risks appear to represent a societal norm for such everyday activities. Moreover, when expressed in terms of fatalities per person-hour exposed, the voluntary risks fall very close to the statistical rate of death from disease; hence the rate of death from disease appears to be a psychological yardstick for establishing the level of acceptability of other risks. Table 1 gives some typical voluntary risks, as interpreted by Wiggins and Moran (1970) from Starr's original work. These risks fall in the general range of 10^{-4} fatalities per person-exposed per year.

"Involuntary" activities differ in that the criteria and options are determined not by the individuals affected but by a controlling body. An example is the risk associated with the use of electricity, where the fatalities include those arising from electrocution, electrically caused fires, the operation of power plants and the mining of fossil fuels. The risk of such fatalities is determined by regulations adopted by governmental agencies in response to public pressures, and thus again represents a societal norm. Starr indicates that the public typically is willing to accept voluntary risks roughly 1000 times greater than involuntary risks. On this basis, Wiggins and Moran suggested that 10^{-7} fatalities/person-exposed/ year might be used as a target for seismic design requirements.

Whereas the first two approaches to decision making involved either exclusively monetary units or exclusively lives lost, multi-attribute decision theory strives to evaluate alternatives in terms of several characteristics (de Neufville and Marks, 1974). In simplest terms, this might mean examining the trade-offs between net discounted expected costs (initial cost less discounted expected future repair costs, but without costs of human life or other social costs) and lives lost, as sketched in Figure 3. In more complex studies, other attributes, such as the cost of social disruption caused by an earthquake, might be added, leading to a three-dimensional, and eventually to an n-dimensional, representation. Techniques have been developed for assessing the preferences of individuals or groups with regard to the trade-offs, and thus assisting decision-makers in their choices.

As part of the study described in this paper, Prof. R.L. deNeufville of the M.I.T. Department of Civil Engineering has initiated the application of this potentially powerful approach to decisions concerning seismic design. This effort, which will be the subject of separate publications, will not be described further in this paper. This approach has the great benefit of emphasizing that there is no such thing as <u>the</u> optimum design, since different individuals or groups will view the trade-off differently.

The role of the engineer, acting as an engineer, is to assemble the facts and to display the consequences of the trade-offs effectively and clearly. As a member of the public, and where appropriate in his role as a public official, the engineer may also argue for his own conclusion as to the appropriate decision regarding the trade-offs. These two roles should, however, be kept separate.

Related Studies

Other investigations have used portions of the methodology outlined in Figure 1.

Seismic risk by itself has been used as a basis for building code provisions. For example, the seismic zoning provisions in the Canadian building code are based upon a comparison of the <u>relative</u> seismic risk in various parts of the country (Whitham et al, 1970). However, the choice of the <u>absolute</u> level of the required seismic design forces are made without any explicit balancing of benefits, costs and risks.

Liu and Neghabat (1972), Shah and Vagliente (1972) and Jacobsen et al (1973) have described cost/benefit analysis of earthquake design. In all of these studies, incident losses were either ignored or assigned dollar values.

Generally, the data concerning initial costs and damage probabilities were relatively crude.

Several rather sophisticated methods for predicting damage caused by future ground motions have been developed. Steinbrugge and others (USCGS, 1969) evolved a procedure for determining earthquake damage to dwellings, for the purpose of evaluating insurance risk and policy. Blume and Monroe (1971) have described a procedure, called the Spectral Matrix Method, for predicting damage caused by large underground explosions. This method has also been used primarily in connection with dwellings. Neither of these methods have been used together with initial cost information to determine a desired level of required resistance.

Grandori and Benedetti (1973) have presented an analysis very similar to that outlined in this paper. Using relatively crude information concerning initial cost, damage probability and seismic risk, the cost of saving a human life is computed as a function of the level of lateral forces required for design.

As contrasted to these other studies, Seismic Design Decision Analysis combines all of the elements shown in Figure 1, and also strives to assemble detailed, credible data concerning the various elements.

A PILOT APPLICATION

To provide focus for the study, a specific design situation was selected: the lateral force requirements for 5- to 20- story apartment buildings in Boston. While both steel and concrete design were considered in the study, this paper will discuss on the results for reinforced concrete buildings. The general floor and structural layout assumed for the study is shown in Figure 4. Shear walls were used to resist lateral forces in the transverse direction while longitudinal forces were resisted by moment resisting frames in the exterior walls. All designs has to resist the wind loading required by the Boston Building Code: 20 psf. Drift requirements under both wind (1/600) and earthquake (1/300) were considered as well as permissible stresses. Masonry block walls were assumed for the exterior walls and interior partitions in accordance with usual practice in Boston. Five different design strategies were considered. Four of these are the requirements for seismic zones 0, 1, 2 and 3 of the Uniform Building Code (UBC), 1970 edition. These requirements differ in the magnitude of the lateral forces required in design and also in requirements concerning reinforcement. The fifth design strategy, designated as superzone S, required lateral forces twice as large as for zone 3. The question was: which design strategy would be most appropriate?

INITIAL COST PREMIUM

Designs for the five design strategies were carried to the point where costs could be reasonably estimated. As the design lateral forces increased, it in general became necessary to increase the number of transverse shear walls and to increase the size and reinforcing steel for the members of the longitudinal frames. With the zone 0 and zone 1 seismic requirements, wind loading was found to prevail and the designs were structurally identical for these two design strategies. For zones 2, 3 and S, it was necessary to consider the design of joints to permit placement of the reinforcing steel required by the code. For zones 1, 2, 3 and S, it was assumed that the code required reinforcement of the masonry walls and partitions. It was further assumed that the walls and partitions should be isolated from the frames by the amount of the computed wind or earthquake drift, and yet must be able to withstand the lateral forces required by the code for the various zones.

Using the designs, the increase in cost over that for no seismic design (zone 0) was estimated, based upon current experience with the construction costs in Boston. Assuming that the total cost of the building with zone 0 requirements would be $28/sq. ft., initial cost premiums were computed as a percentage of the cost with seismic requirements. The results are given in Figure 5 for three different heights of building.

The increase for the zone 1 design stems from the requirement that masonry walls be reinforced. The further increase for the zone 2 design comes largely from the additional reinforcement to meet the ductility provisions of the code. The additional increase for zones 3 and S reflect the increased member sizes and reinforcement required to resist the increased lateral forces. It should be remembered that the structural system contributes only about one-quarter of the total cost of a building. Hence, the overall percentage increases shown in Figure 5 correspond to much larger percentage increases in the cost of the structural system.

These initial cost premiums are consistent with the very scant literature concerning such costs (SEAOC, 1970).

SEISMIC RISK

The likelihood of ground motions of different intensities was determined using the procedures developed by Cornell (1963).

The first step is to establish a set of source areas distinguished by identifiably different seismic histories and different geology and tectonics. This is difficult to do for the region of Boston, since the causes of past earthquakes are so poorly understood. Figure 6 shows the source areas identified for this study based upon discussions with local seisnologists and geologists, and also indicates the historical earthquakes having epicentral intensities of modified Mercalli intensity (MMI) of V or greater.* The earthquake of 1755, which is often cited as the basis for concern about earthquakes in the region of Boston, is believed to have had its epicenter in source 2. Recent studies indicate that the epicentral intensity of this earthquake was about MMI VIII, while the intensity in Boston itself was MMI V or VI on firm ground and MMI VI or VII on poor soil. Source 5 is a random source used to represent the background earthquakes not covered by any of the specific sources.

Recurrence rates for earthquakes in each of the sources are based upon a study of the historical record. The ratio of the recurrence rates for earthquake of two different epicentral intensities is known to be very similar for many different parts of the earth, and this same ratio was found to apply in the Boston region. Thus, the frequency at which moderate or strong earthquakes would be expected in any source area can be estimated from the rate at which small earthquakes are occurring in the source.

It generally is presumed that the character of earthquakes in the northeast states region is such that there are inherent limitations on the epicentral

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*A brief resume of the modified Mercalli intensity scale appears in Appendix B.

intensity that can occur. Thus, upper bound epicentral intensities were selected for each of the sources. For the two sources (5 and 9) that encompass and are nearest to Boston, these upper bounds were taken as a low VI and a low VII, respectively. For source 2, the upper bound was set at a high VIII. These estimates on upper bounds are perhaps the most concertain and most controversial part of the entire analysis, as well be seen subsequently.

All of the foregoing information is combined together into an analytical procedure which also incorporates an empirical law giving the attenuation of intensity with distance from an epicenter. This analytical procedure calculates the probability that in any year there will be a ground motion, at the site of interest, equal to or greater than some specified intensity. This result proved to be the same for all locations in Boston and Cambridge.

Figure 7 gives results for several different assumptions concerning various parts of the analysis. Curve 1 represents the best professional estimate of the seismic risk in Boston; this curve is based upon the estimated upper bounds for epicentral intensities for all sources. In computing curve 3, it was assumed that there were no upper bounds to the epicentral intensities in sources 1 and 2 but that the estimated upper bounds applied to the other sources, while in computing curve 4 no upper bounds were assumed for any of the sources. Curves 3 and 4 represent possible but unlikely interpretations of the seismic risk to Boston. Curve 5 was computed using only source 5, using a recurrence rate based upon all historical earthquakes that had occurred anywhere within this source area, and assuming that there is no upper bound to the epicentral intensity. According to these assumptions, an earthquake equal to or larger than the 1755 earthquake is as likely to have its epicenter directly under downtown Boston as at any other point near

Boston. This is the most conservative possible interpretation of the seismic history of the Boston region, and in the professional view of the study group staff it is a very unrealistic and unlikely interpretation. Curves 3, 4 and 5 all extend to MMI X, the largest intensity considered, with constant slope.

Curves 1, 3, 4, and 5 all give the intensity for firm ground such as dense glacial till and outcroppings of rock. The historical record for the region of Boston contains ample evidence that damage during the larger historical earthquakes was greater on soft ground than on firm ground. The specific effect of softer ground is still to be analyzed as part of the study. For purposes of this pilot application, it was assumed that soft ground increases the intensity by one unit on the modified Mercalli scale. Thus curve 2 gives the best estimate of the seismic risk for soft ground in Boston.

It is recognized by all earthquake engineers that the modified Mercalli scale is a poor representation of the intensity of ground motion. For SDDA, it would certainly be desirable to utilize a more quantitative measure of intensity based upon some characteristic (such as peak acceleration, peak velocity, spectral acceleration, etc.) of strong ground motion. However, unfortunately there are no strong motion records from the eastern United States and the entire seismic history of this region can be expressed only in MMI. As noted in the following section, much of the available information concerning the probability of damage during earthquakes also can be related only to MMI. Hence, in this pilot application, MMI has been used as the basic measure of the strength of ground shaking.

DAMAGE PROBABILITY

The general form of the damage probability matrix (DPM) used for this study is shown in Figure 8. Damage to buildings is described by a series of damage states (DS), while the intensity of ground motion is described by the modified Mercalli intensity scale. Each number P_{DSI} in the matrix is the probability that a particular state of damage will occur, given that a certain level of earthquake intensity is experienced. The sum of the probabilities in each column is 100%. There are several reasons why there is a spread in the damage resulting from a particular intensity of ground shaking:

1. Individual buildings, from a group of buildings all designed to meet the same requirements, will have different resistances to earthquake damage, depending upon the skill and inclination of the individual designer and upon the quality of construction.
2. The details of the ground motion, and hence the dynamic response of identical structures, will differ significantly at different locations all experiencing the same general intensity.

Hence the damage to be expected in future earthquakes must be expressed in probabilistic terms.

Each damage state is defined in two ways: (a) by a set of words describing the degree of structural and non-structural damage, and (b) by a ratio of the cost of repairing the damage to the replacement cost of the building. If the actual cost of damage is known, then the damage ratio (DR) is the best method for identifying the damage state. However, the record of damage during past earthquakes often does not indicate the actual costs of damage, and in these cases the alternate word description must be used to characterize damage states. For the work of the study, the brief one-word damage descriptions appearing in Figure 8 were supplemented by more detailed descriptions.

As already noted, MMI is a rather poor way of characterizing the strength of ground shaking. However, the damage probabilities used in this study are expressed in terms of MMI because: (a) the historical record of earthquakes in the eastern United States is entirely in terms of MMI, and (b) in many of the earthquakes causing important change to buildings, no strong motion records were obtained and the strength of shaking can be expressed only by MMI. Many schemes have been suggested for converting from MMI to a quantitative measure of intensity, but as yet there is little agreement on this point. Once better agreement is possible, the DPM developed in this study can readily be converted into terms of a quantitative intensity scale.

For many applications, it suffices to replace the full set of probabilities in each column of a DPM by a mean damage ratio (MDR), defined as:

$$MDR = \sum_{DS} (P_{DS\,I})\,(CDR_{DS})$$

where CDR_{DS} is the central damage ratio for damage state DS. The summation is made over all damage states, and the resulting MDR is a function of MMI. In the few cases where the actual damage ratio (DR) is known for each building, a more accurate value of MDR may be obtained by simply averaging the individual DR.

Evaluation of Damage Probabilities

The best way to evaluate damage probabilities is from experience during actual earthquakes. For this reason, a considerable portion of the study has been devoted to documenting the damage (and non-damage) to buildings shaken by the San Fernando earthquake of 1971. Damage ratios were documented for about 370 buildings out of a total of about 1600 buildings having 5 stories or more. Many of these buildings had been built prior to 1933 when the codes contained no requirements for design against earthquakes; many others had been built since 1947 under code requirements similar to those for zone 3 of the current UBC. The buildings could be further subdivided by structural material (steel or concrete) and by the number of stories. Where possible, damage was divided into structural damage and various types of non-structural damage. Table 2 shows one of the many DPM that were evaluated by this effort, while values of MDR as a function of intensity are plotted in Figure 9. More complete details of the study may be found in several papers and reports (Whitman et al, 1973a, 1973b).

Several other past earthquakes have also been analyzed so as to develop DPM: the Caracas earthquake of 1967, two earthquakes in Japan during 1968, the damage in Anchorage during the Alaskan earthquake of 1964, the San Francisco earthquake of 1957 and the Puget Sound earthquake of 1965. In all of these cases, only descriptions of the damage, and not actual damage ratios, were available. The MDR from all of these past earthquakes have been plotted in Figure 10. Most of these earthquakes involved shaking of predominantly concrete buildings. Overall, there is an encouraging degree of consistency, especially since some of the data for MDR are relatively crude and MMI is only a crude indicator of the intensity of ground shaking.

However, the sum total of such empirical data proved inadequate for the purpose of determining DPM for the pilot application of SDDA. The data were especially scant for the higher intensities. Morever, the empirical data are not necessarily applicable to buildings in a particular city, such as Boston, without further interpretation. Hence the empirical data were supplemented by theoretical studies (Anagnostopoulos, 1972; Biggs and Grace, 1973; Czarnecki, 1973). The designs described in the section on initial cost premiums were modelled mathematically and dynamic response analyses were carried out. These theoretical studies provided considerable insight into the effect of strengthening a building upon expected dynamic response of the building. The buildings designed without seismic requirements were found to yield at MMI VI. Because strengthening a building also causes stiffening which in turn means greater induced forces during an earthquake, designing for seismic forces led to only modest increases in the intensity of ground motion that would first yield a building (see Fig. 10)and in the damage predicted at various intensities.

At this stage, it became apparent that theoretical analyses by themselves did not reflect all of the subtle ways in which designing for seismic forces improves the resistance of a building. For example, increasing design forces undoubtedly leads to better details at joints between members, simply because the designer is forced to pay more attention to these joints. Hence, in order to supplement the empirical data and theoretical results, a structural engineering firm from Los Angeles was asked to evaluate DPM for these same buildings, using their subjective judgment.

These efforts led finally to the MDR shown in Figure 11 and the corresponding DPM in Table 3. The reader should keep in mind that these results apply to concrete buildings as they might be designed and constructed in Boston today. By more attention to the detailing of non-structural portions, the damage to buildings at the lower intensities might be reduced. By giving great attention to the reinforcement of shear walls and columns, the probability of great damage and collapse at higher intensities might be reduced.

The total effort of assembling the results given in Table 3 and Figure 11 is described in a forthcoming research report (Whitman, 1973).

Incident losses include all of the consequences of an earthquake beyond the cost of repairs to the building. These consequences include: damage to building contents, disruption of normal users' activities both during and after the event, injuries, lives lost, cost of rescue and victims assistance operations, impact on local economy and other similar factors. These consequences may be subdivided into those where an economic value may reasonably be assigned (damage to contents, disruption of normal activities), and those where it is very difficult, and perhaps even meaningless, to assign an economic value (loss of life, impact on economy).

As part of the overall study, an attempt was made to ascertain the cost of the first class of incident losses: those to which economic value could reasonably be assigned. A first step was to determine the type of incident loss typically associated with each of the damage states. Toward this end, a set of photographs taken inside buildings affected by the San Fernando earthquake was assembled; the overall damage state for these buildings had already been established. These photographs were shown to engineers and building owners who were then asked to estimate the incident costs suggested by these pictures. Owners and managers of buildings shaken by the San Fernando earthquake were interviewed to determine the actual incident costs, if any. Finally, cost estimates were made by the staff of the study project. All of these efforts led to the incident cost ratios in the 3rd column of Table 4. Except for damage state L, these incident costs are small compared to the repair costs, and hence they were ignored (including those for DS L) in the subsequent analysis.

In order to make some study of the role of injury and loss of life, experience was used to estimate the fraction of the building occupants who might be killed and injured corresponding to the several damage states. These fractions, which are given in columns 4 and 5 of Table 4, are influenced by a number of considerations: the fraction of the total occupants that are, on the average, present in a building at any time; that collapse may be partial rather than total; and that passersby may be killed and injured by falling objects or by collapse. By using typical data for the cost of an apartment building per occupant, and by assigning values to death and injury ($300,000 per life and $10,000 per injury), the percentages in the last column of Table 4 were determined. This column gives the cost of injury and life lost as a percentage of the replacement cost of a building.

As discussed earlier, cost/benefit analyses incorporating a monetary value on human life are unpalatable to many people. However, it has seemed desirable to pursue this approach at least to the point of seeing its implications. From the results in Table 4, it is evident that the human factor will be of great importance no matter what value one might choose to place on life.

By combining the damage probabilities in Table 3 with the ratios in Table 4, two additional mean ratios may be computed. Figure 12 gives the life loss ratio as a function of MMI and design strategy; this ratio is based upon the fractions in column 4 of Table 4. Figure 13 similarly gives the total cost ratio, based upon the sum of columns 2 and 6 in Table 4. Each of these two new loss ratios is computed in the same way as the mean damage ratio.

RESULTS

Having assembled all of the necessary information, it is a relatively simple matter to calculate the costs and expected benefits associated with the different design strategies. The method of calculation is illustrated in Appendix C. Calculations have been made using all of the seismic risk curves in Figure 7. Expected repair cost (Figure 11), expected total cost (Figure 13) and expected loss of life (Figure 12) have all been computed. In the computation of the present value of future dollar losses, it has been necessary to assume a discount rate: 5% per year has been used. Where appropriate, an average of the initial cost curves in Figure 5 has been introduced.

Damage Repair Costs

The second column in Table 5 shows the present value of expected future repair costs, expressed as a percentage of the replacement cost of a building, for multi-story reinforced concrete buildings with no design against earthquake forces (i.e., designed for UBC zone 0). Comparing these losses with the initial cost premiums in Figure 5, it may be seen that, except for the most conservative estimate of seismic risk, the expected losses without seismic design are less than the cost to introduce seismic design. Even for the most conservative estimate of seismic risk, the net discounted cost (the sum of the initial cost plus the discounted expected losses) is smallest when no seismic design is required. This result is shown in Figure 14.

Thus, it would appear that, over the long run in Boston, the cost of required seismic design will not be recovered through decreased cost of earthquake-caused repairs.

Total Costs

When the above-mentioned values for human life and injury are introduced, the total discounted expected costs are given by the 3rd column in Table 5. For the most conservative seismic risk curve is used, design for UBC zone 3 requirements appears to lead to minimum net cost, as is shown in Figure 14. However, for all other seismic risks the cost of providing seismic resistance is found to increase more rapidly than the reduction in losses brought about by the increased seismic resistance. Of course, this last conclusion would change if a greater value were to be assigned to a human life.

Loss of Life

Figure 15 summarizes the computed annual fatality rate for the five seismic risk curves, assuming no seismic design requirements. For comparison, the fatality rate from "normal" accidents and the average earthquake-caused fatality rate for all of California during the present century are also shown on the figure. The reader should also keep in mind the earlier observation that the public seems to desire that involuntary risks be 1000 times smaller than the normal accident risk, which would mean a limit of about 10^{-7} fatalities/person exposed/year.

The results in Figure 15 suggest that the fatality rate may be unacceptably large for several of the seismic risk curves. There are several questions that may reasonably be asked.

First, why do the life loss calculations suggest unacceptable conditions while the previous total cost calculations (involving the cost of lives lost) indicate benefit to seismic design only for the most conservative seismic risk? The apparent answer is: when choosing an acceptable involuntary risk, the public must place a very large value on human life. This is confirmed by the study of Grandori and Benedetti, who found that current practice in several countries implies an expenditure of over a million dollars to save one life.

Second, why are some of the computed earthquake fatality rates in Boston greater than the rate in California? The answer to this question has several parts. (a). The rate for California is an average for all of California; it would be higher if only the highly seismic portions of California were considered. (b). The occurrence of one very large earthquake near either San Francisco or Los Angeles might well up the historical California rate by several orders of magnitude. (c). Most of the computed life loss is associated with MMI IX and X, and hence result directly from the assumption that there is no upper bound to the epicentral intensities of earthquakes near Boston. The resulting probabilities for MMI IX and X are large even for California.

Third, will providing increased seismic resistance significantly decrease the expected loss of life? The answer here is yes; Figure 16 shows some of the computed results.

Recurrence of 1755 Earthquake

The foregoing results are based upon average annual losses. Such results should be meaningful to a person who likes to gamble with long term odds. However, it is also meaningful to ask: what would happen if the 1755 earthquake were to reoccur tomorrow?

This question can also be answered using the information that has been assembled. To make the question more specific, assume that MMI VI occurs on firm ground in Boston. According to the best estimate seismic risk curves in Figure 7, such an intensity might be expected to occur once every 167 to 900 years.

According to Figure 11, there is zero probability of loss of life in concrete buildings on firm ground, even if such buildings have not been designed for seismic resistance. However, this same earthquake can be assumed to cause MMI VII on soft ground. Now the mean life loss ratio becomes 10^{-4}. Thus, if 50,000 people are living in multi-story apartments built over soft ground, 5 deaths might be expected on the average -- which means that the actual number of deaths in a particular earthquake might range from zero to perhaps 50 or 100. The possibility of these deaths would be entirely eliminated by going to UBC zone 3 requirements.

IMPLICATIONS FOR BOSTON

Each reader should reach his own conclusions based upon his own personal reaction to risk. However, two points should be emphasized. First, in all of this study, it has been assumed that the typical reinforced concrete building has at least a nominal amount of ductility and will not collapse as soon as it starts to yield. A much more pessimistic picture would result if reaching yield point indicated imminent collapse. Second, this paper has introduced the effect of soil conditions in a very crude fashion. Further study may indicate that the effect of poor soil may be greater than increasing MMI by one unit.

With these points in mind, the writers have reached several personal conclusions.

1. Normal concrete buildings located on firm ground in Boston do not need to be designed for seismic resistance. It seems likely that further study will lead to the same conclusion even for poor ground.
2. Considering the uncertainty in the estimates of seismic risk, Boston should not totally ignore the danger of earthquakes. Buildings which may not have nominal ductility, such as buildings using prefabricated elements, buildings with relatively few vertical load carrying members or buildings with unusual shapes should receive special attention. This is particularly true when such buildings are located over poor ground. The form of these special requirements is a subject for further study.
3. In buildings, such as hospitals, where even a small amount of non-structural damage might impede vital functioning, special attention should be given to the design of certain features such as elevators, power supply and storage racks.

It must be remarked that these conclusions are intended to apply to the City of Boston. More attention to seismic design may well be warranted in communities lying closer to source areas 1 and 2 on Figure 6.

CLOSING REMARKS

As stated at the outset, the primary purpose of this paper has been to describe and illustrate a procedure for organizing into a useful format the information required to arrive at a balance between the cost of designing to give earthquake resistance and rhe risk of damage and loss of life vs future earthquakes. The illustration selected involved a particular type of building in a specific city. However, the methodology developed by the study hopefully is applicable to other types of buildings in other locations. The methodology potentially can be extended to include engineered facilities other than just buildings.

The illustrative example has looked at only part of the earthquake problem in Boston, and has served primarily to indicate the types of conclusions that may be reached by such a study. As has been indicated, SDDA is intended as a tool for engineers, building officials and public bodies, and much more interaction is required with such people before firm recommendations can be given. Moreover, other aspects of the earthquake threat to Boston, such as the damage susceptibility of the city's lifelines (utilities, transportation and communication facilities) and what to do about old buildings, may well be much more important than the problem thus far examined.

Table 1

DEATH RISK RATE FOR VOLUNTARY ACTIVITIES*

Risk Category	Risk Rate Deaths/10^6 persons/year
Motor vehicle	266
All work	190
Public accidents	100
Home accidents	143

*From Wiggins and Moran (1970), using an interpretation of original data by Starr (1969).

Table 2

SAN FERNANDO EARTHQUAKE

DAMAGE PROBABILITIES FOR POST-1947 BUILDINGS

DAMAGE STATE	INTENSITY		
	VI	VII	VII-VIII
0	79%	33%	6%
1	18%	34%	19%
2	3%	20%	44%
3	-	10%	13%
4	-	3%	6%
5	-	-	12%
6	-	-	-
7	-	-	-
8	-	-	-
MDR	0.05%	0.5%	2.74%
NO BLDGS	57	156	16

Table 3

DAMAGE PROBABILITIES (%) FOR PILOT APPLICATION
OF SEISMIC DESIGN DECISION ANALYSIS

DESIGN STRATEGY	DAMAGE STATE	MODIFIED MERCALLI INTENSITY V	VI	VII	VII.5	VIII	IX	X
	O	100	27	15	0	0	0	0
	L	0	73	48	21	0	0	0
UBC 0.1	M	0	0	33	45	20	0	0
	H	0	0	4	29	41	0	0
	T	0	0	0	5	34	75	25
	C	0	0	0	0	5	25	75
	O	100	47	20	0	0	0	0
	L	0	53	50	36	10	0	0
UBC 2	M	0	0	29	52	53	0	0
	H	0	0	1	11	31	0	0
	T	0	0	0	1	5	80	60
	C	0	0	0	0	1	20	40
	O	100	57	25	5	0	0	0
	L	0	43	50	48	25	0	0
UBC 3	M	0	0	25	41	53	20	0
	H	0	0	0	6	21	52	0
	T	0	0	0	0	1	23	80
	C	0	0	0	0	0	5	20
	O	100	67	30	10	0	0	0
	L	0	33	49	58	40	10	0
S	M	0	0	21	29	52	30	0
	H	0	0	0	3	8	58	0
	T	0	0	0	0	0	2	90
	C	0	0	0	0	0	0	10

Table 4

INCIDENT COSTS

Damage State	Central Damage Ratio - %	Incident Cost Ratio - %	Fraction Dead	Fraction Injured	Human Cost Ratio - %
None (O)	0	0	0	0	0
Light (L)	0.3	0.3	0	0	0
Moderate (M)	5	0.4	0	1/100	0.6%
Heavy (H)	30	2	1/400	1/50	7%
Total (T)	100	3	1/100	1/10	30%
Collapse (C)	100	-	1/5	1	600%

Table 5

EARTHQUAKE LOSSES FOR BUILDINGS

DESIGNED FOR UBC ZONE 0[a]

	Discounted Losses as % of Replacement Cost	
Seismic Risk[b]	Repair Cost	Total Cost[c]
Curve 1	0.0064	0.0064
Curve 2	0.17	0.20
Curve 3	0.17	0.42
Curve 4	1.2	2.9
Curve 5	3.3	8.3

(a) Computed using 5% discount rate

(b) See Figure 7

(c) Includes $300,000 per life and $10,000 per injury

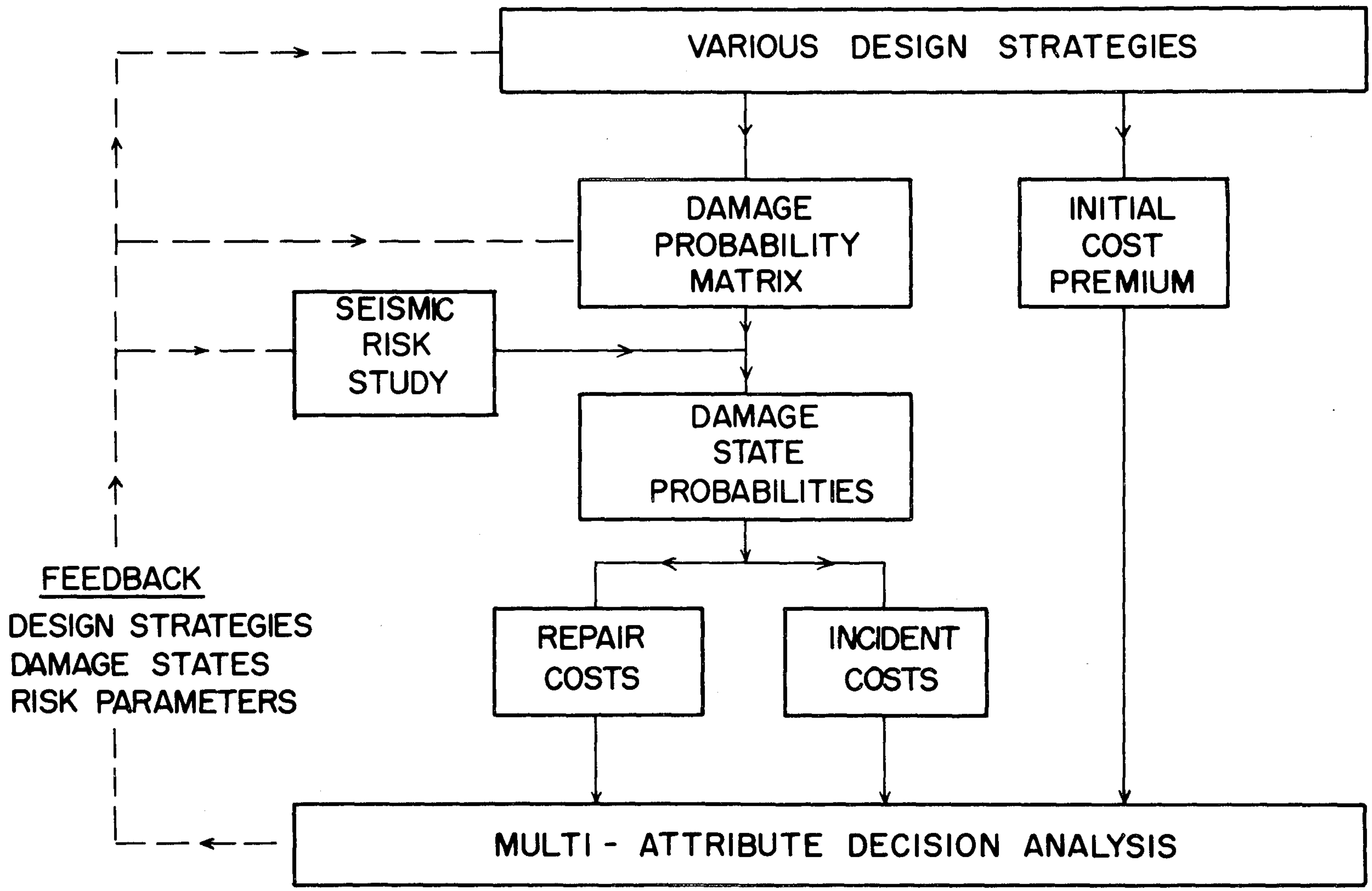


FIGURE I FLOW DIAGRAM FOR GENERAL METHODOLOGY

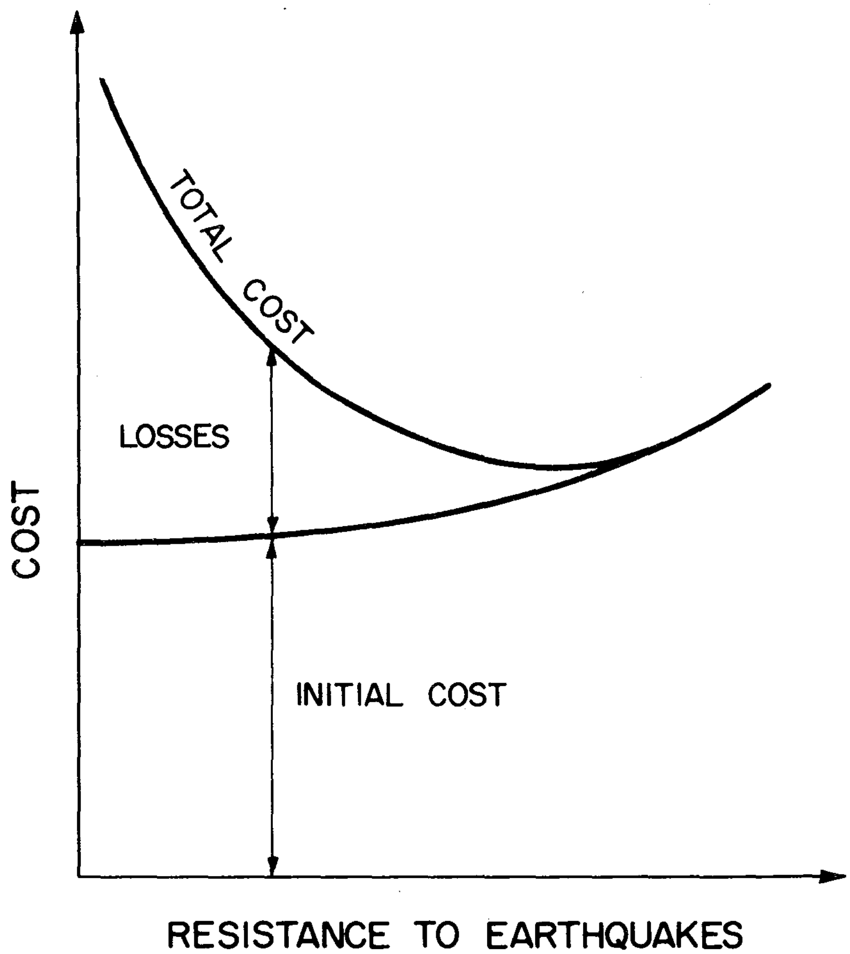


FIGURE 2 COST / BENEFIT ANALYSIS

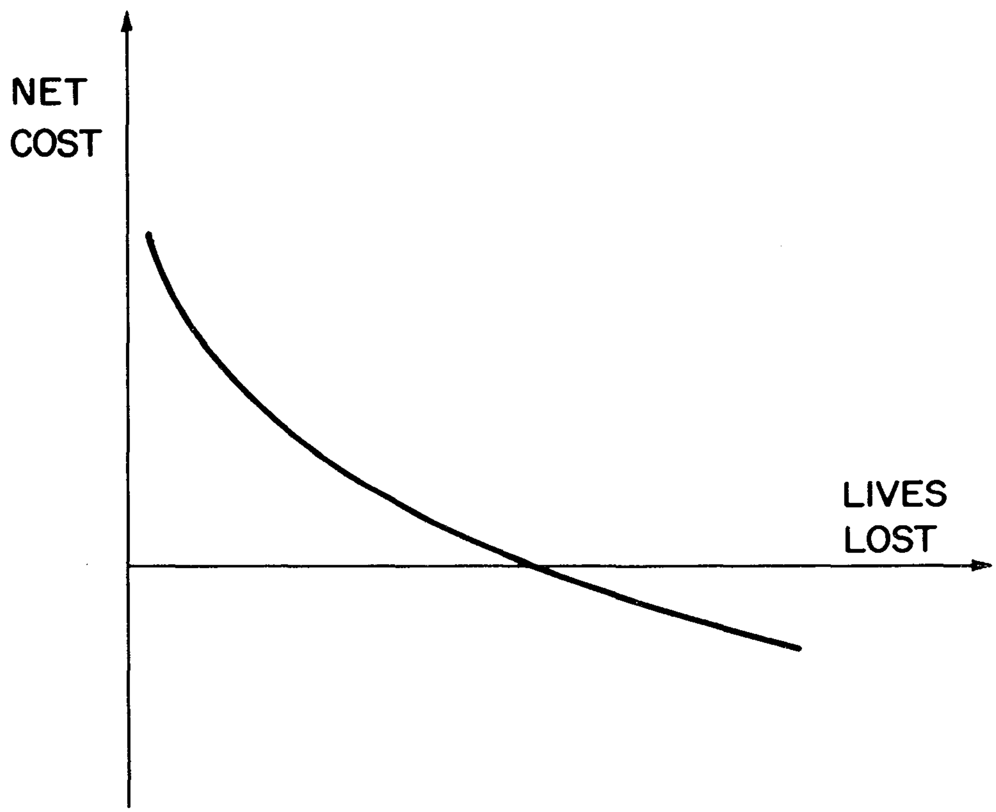


FIGURE 3 TRADE-OFF BETWEEN MULTIPLE OBJECTIVES

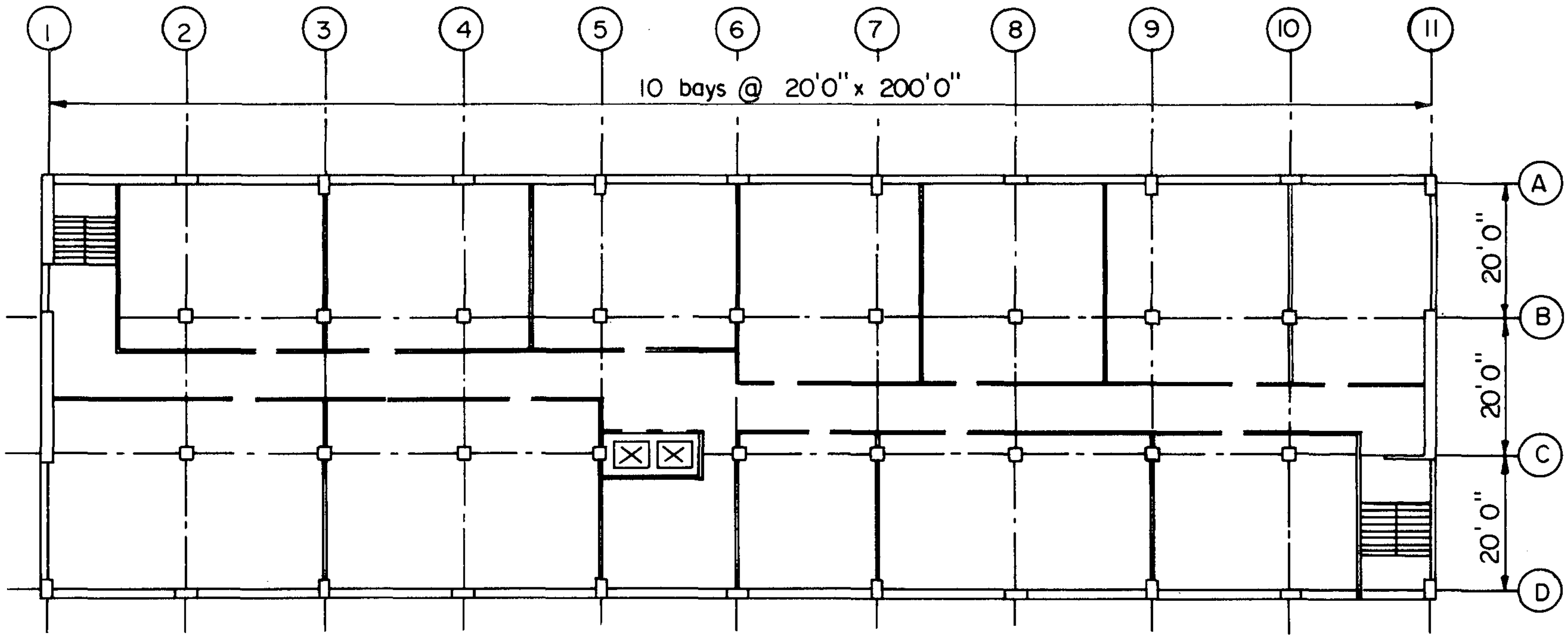


FIGURE 4 TYPICAL FLOOR LAYOUT

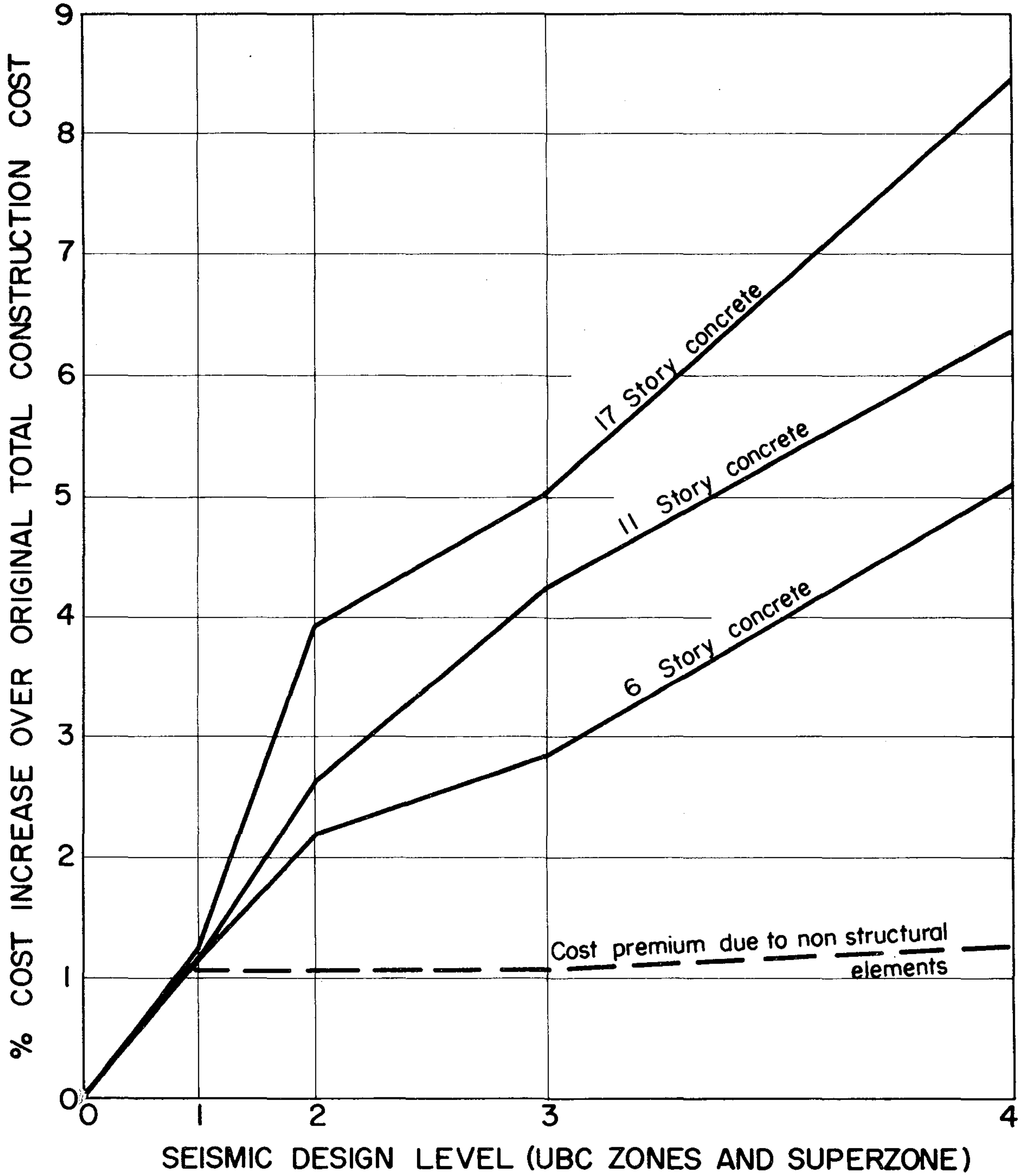


FIGURE 5 INITIAL COST PREMIUMS FOR TYPICAL CONCRETE APARTMENT BUILDINGS IN BOSTON

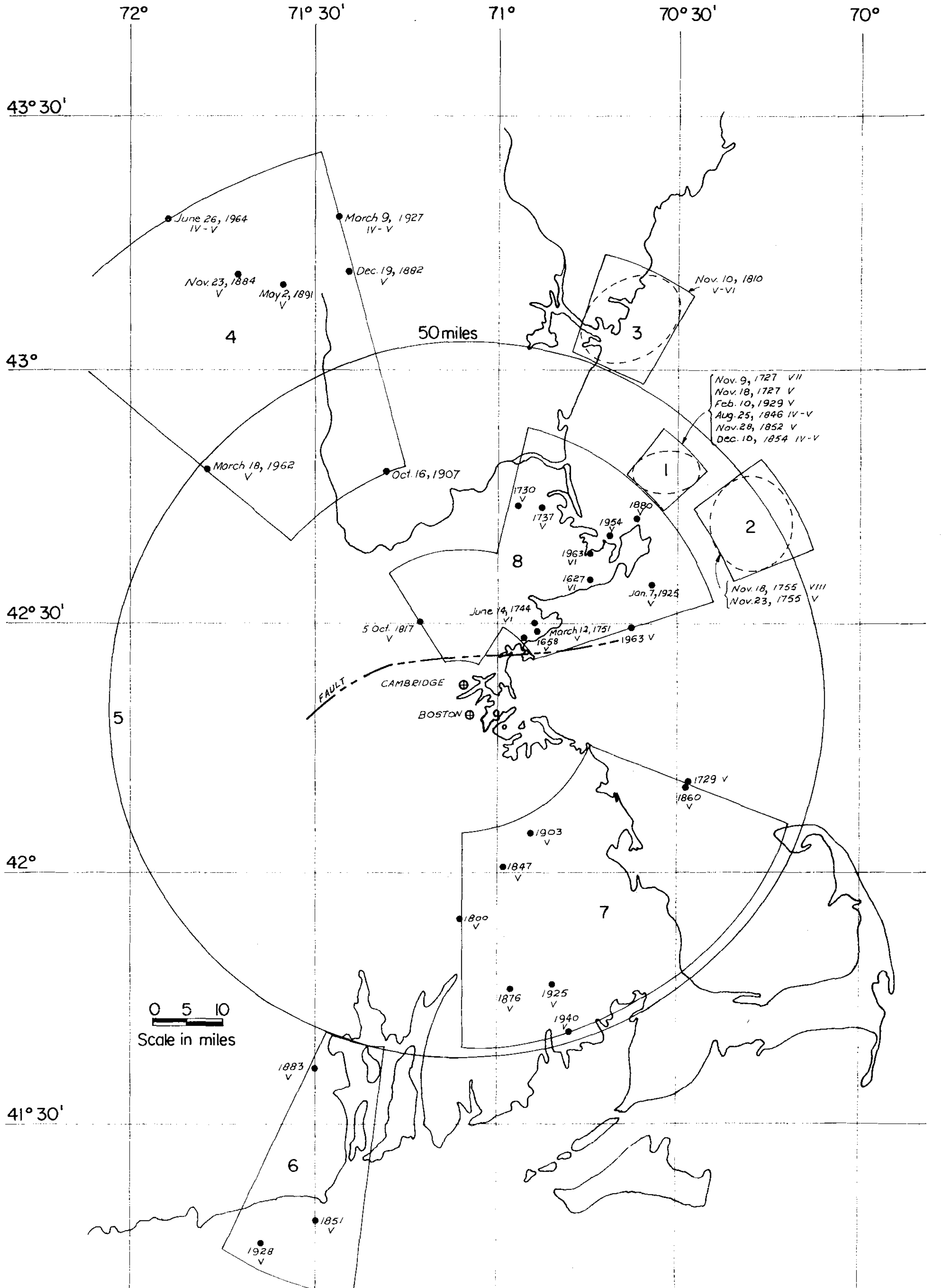


FIGURE 6 HISTORICAL EVENTS WITH (MM) I > V, SHOWING GROSS SOURCES

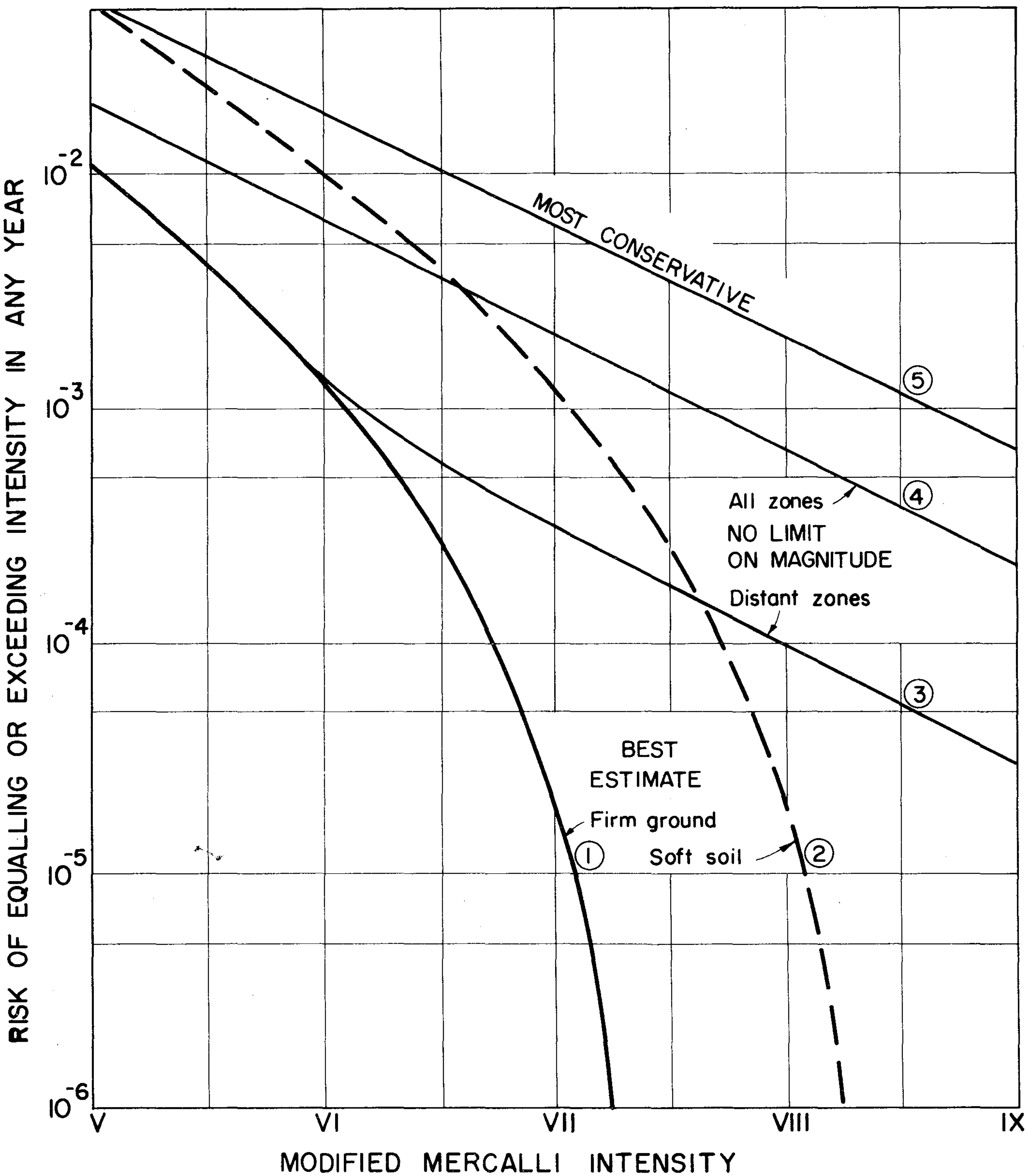
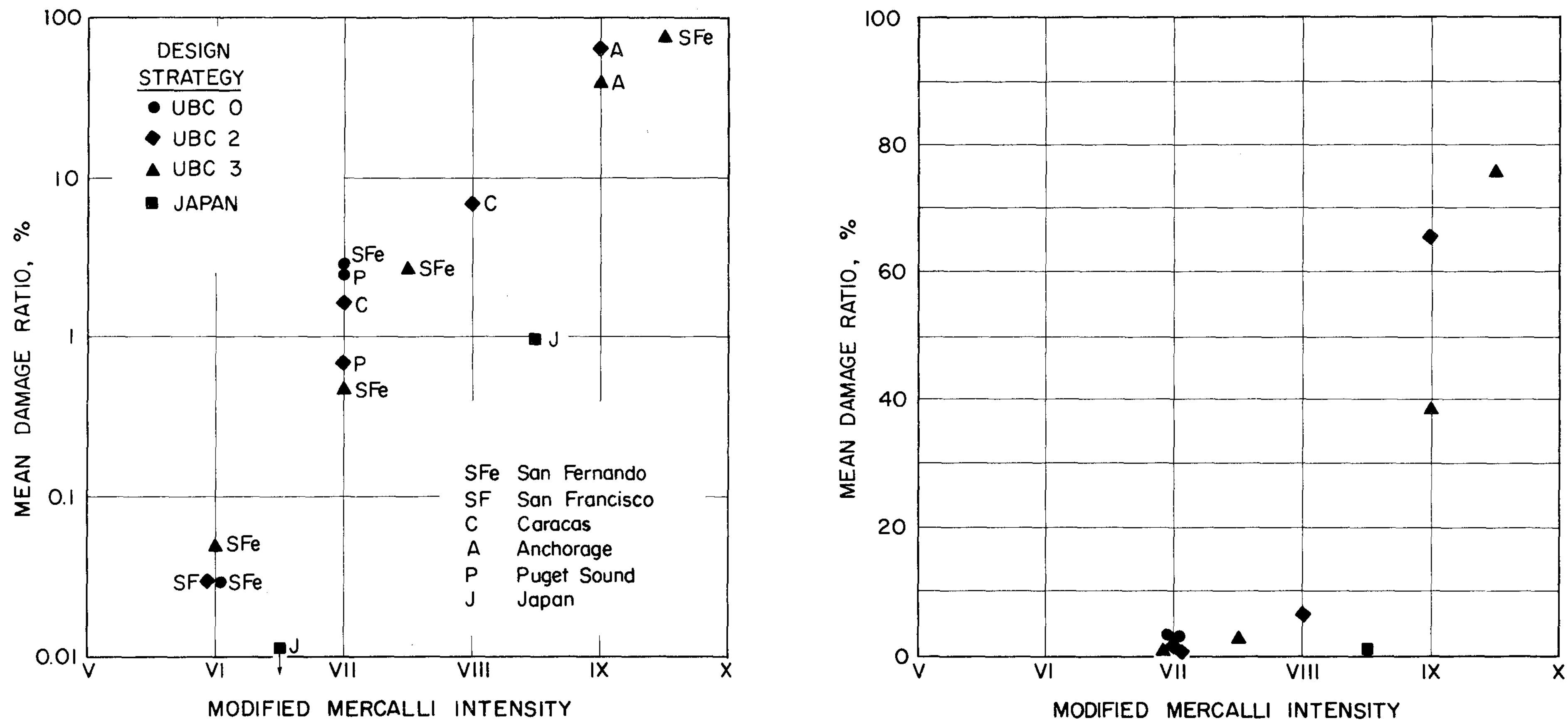


FIGURE 7 VARIOUS ESTIMATES OF SEISMIC RISK IN BOSTON

41.

DAMAGE STATE	CENTRAL DAMAGE RATIO, %	MM INTENSITY				
		VI	VII	VIII	IX	X
O - NONE	0					
L - LIGHT	0.3		P_{DSI}			
M - MODERATE	5					
H - HEAVY	30					
T - TOTAL	100					
C - COLLAPSE	100					

FIGURE 8 FORM OF DAMAGE PROBABILITY MATRIX



(a) (b)

FIGURE 9 MEAN DAMAGE RATIOS FROM HISTORICAL EARTHQUAKES

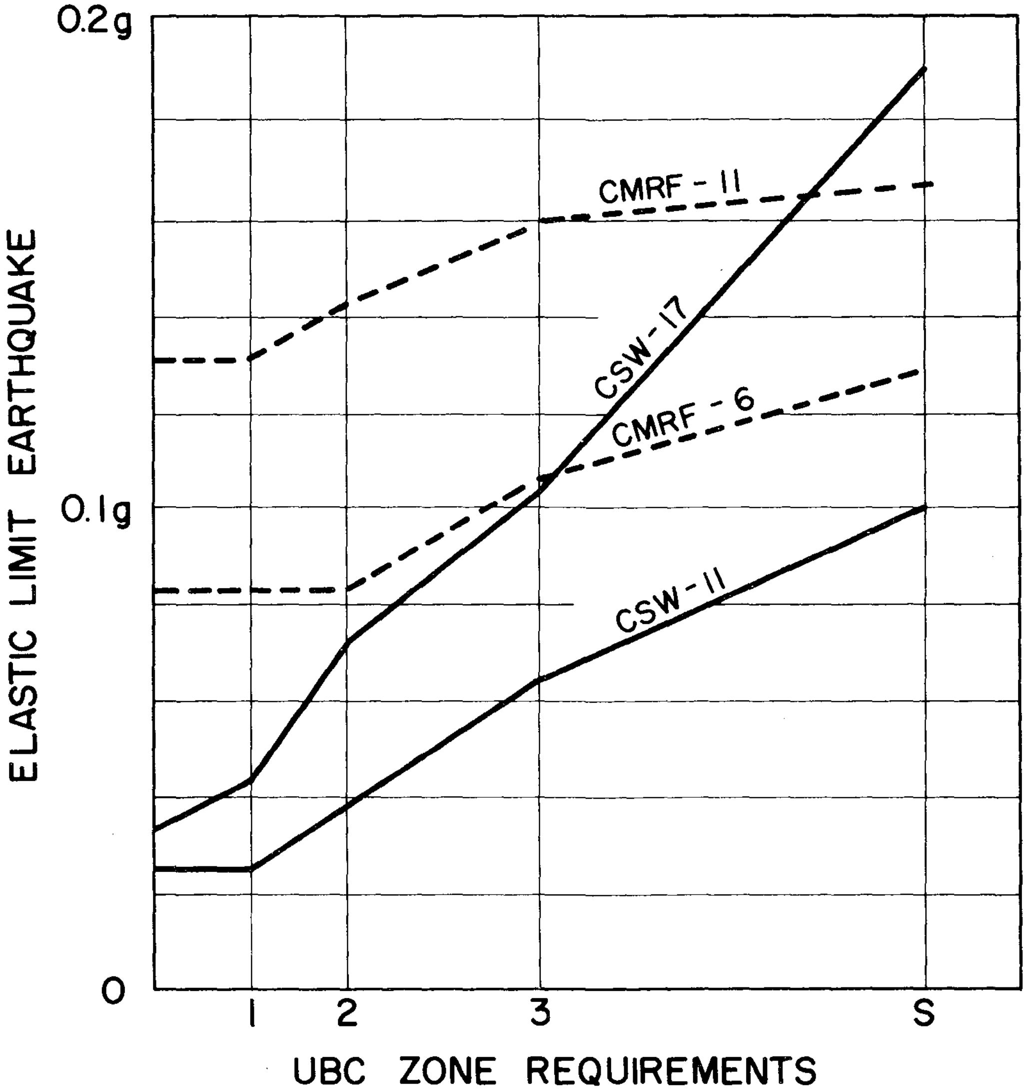


FIGURE 10 EFFECT OF DESIGN STRATEGY UPON INTENSITY OF EARTHQUAKE FIRST CAUSING YIELD

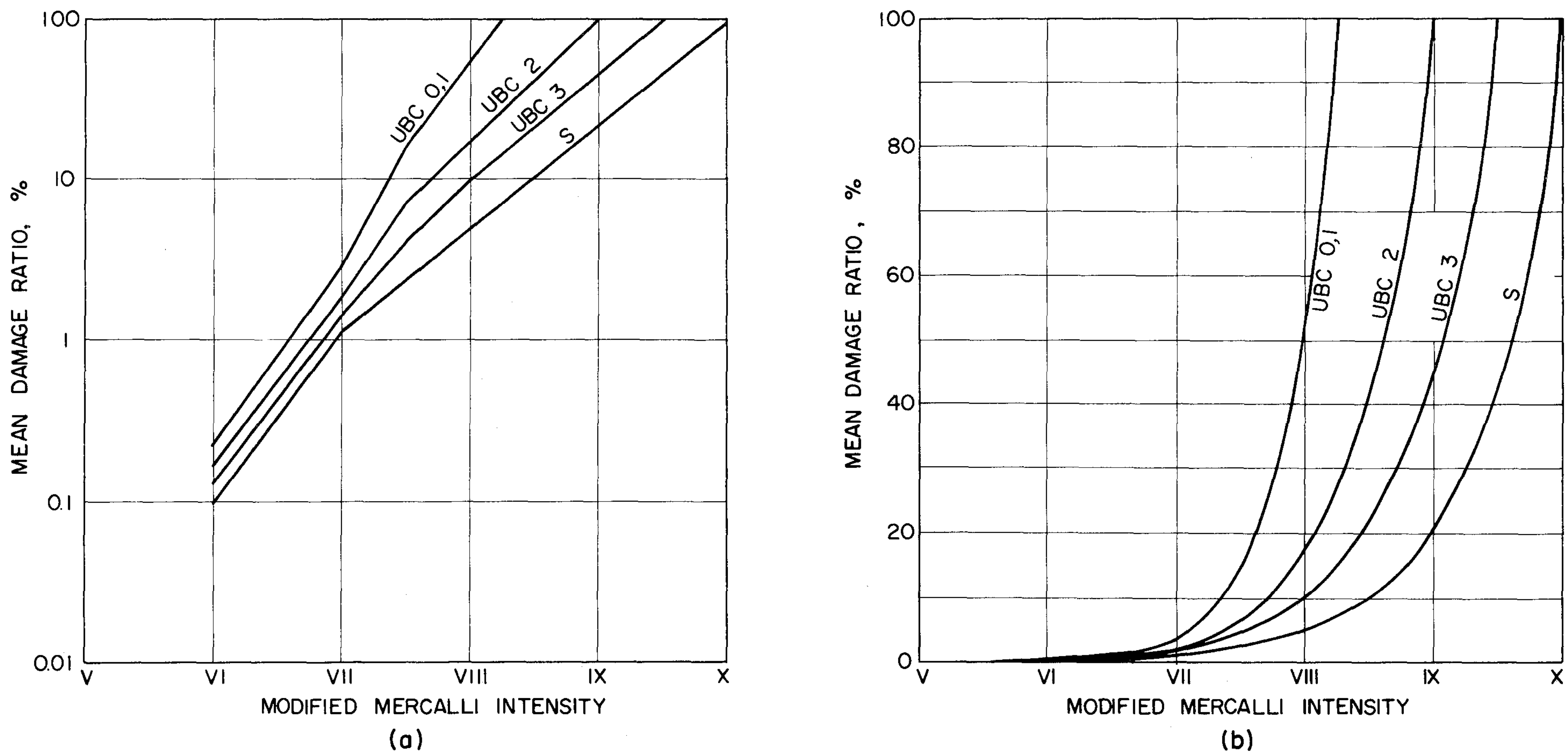


FIGURE 11 MEAN DAMAGE RATIOS FOR PILOT APPLICATION OF SEISMIC DESIGN DECISION ANALYSIS

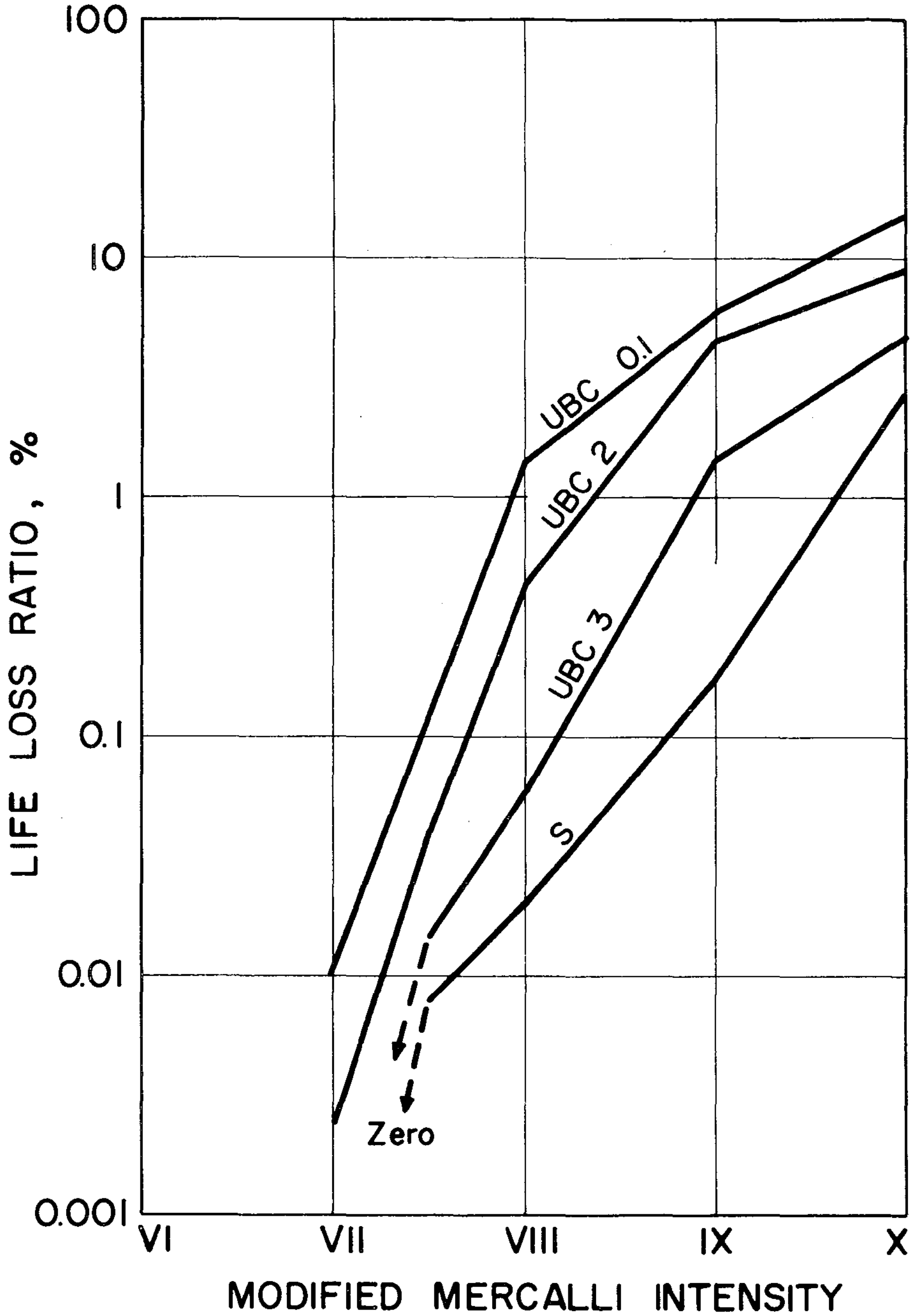


FIGURE 12 LIFE LOSS RATIO FOR UBC 0 DESIGN STRATEGY

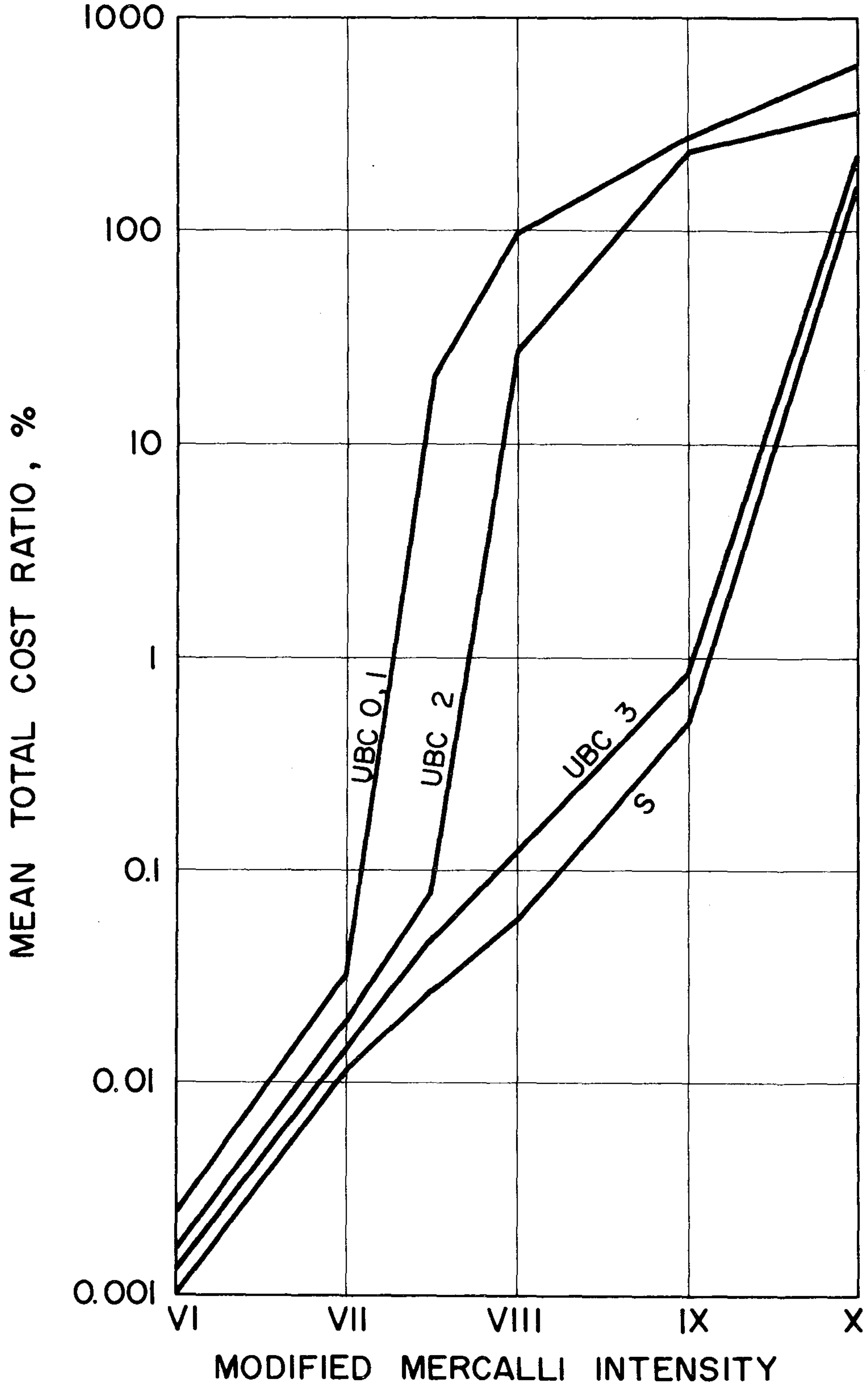


FIGURE 13 MEAN TOTAL COST RATIO FOR UBC 0 DESIGN STRATEGY

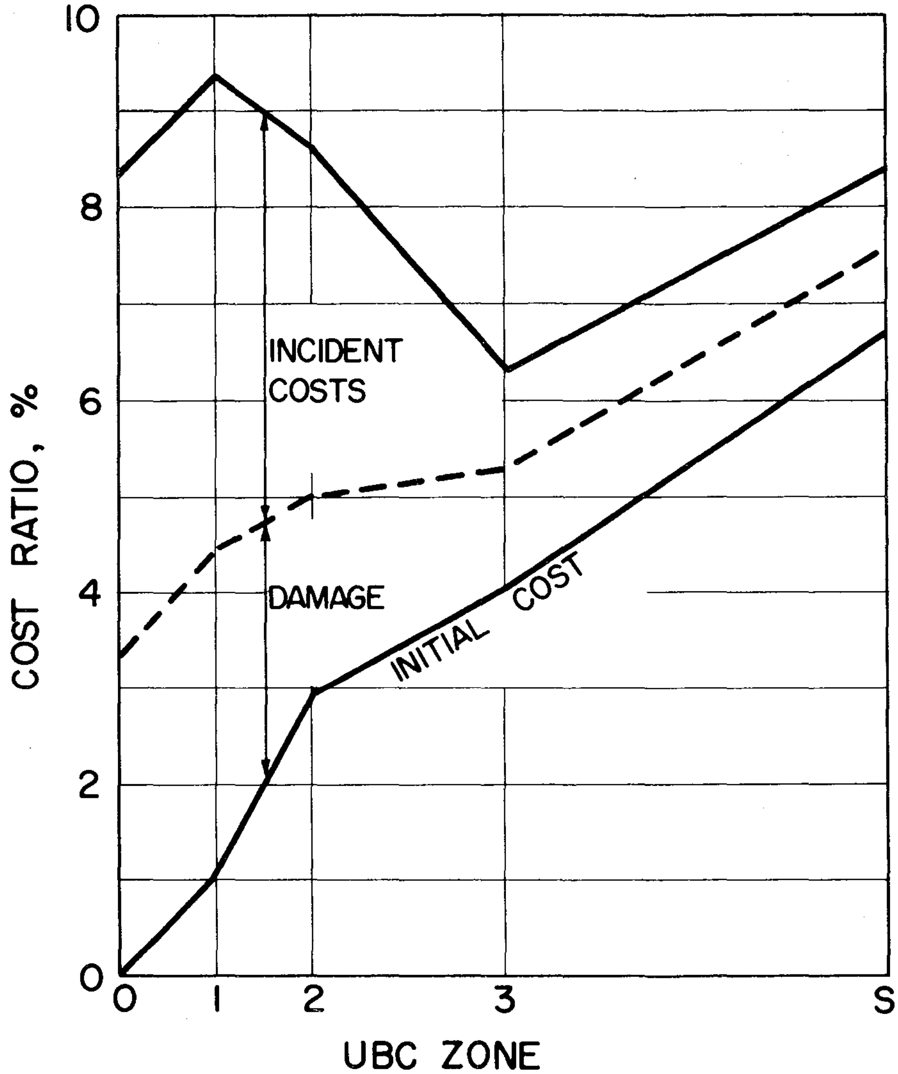


FIGURE 14 COST RATIOS FOR MOST CONSERVATIVE ESTIMATE OF SEISMIC RISK ON FIRM GROUND

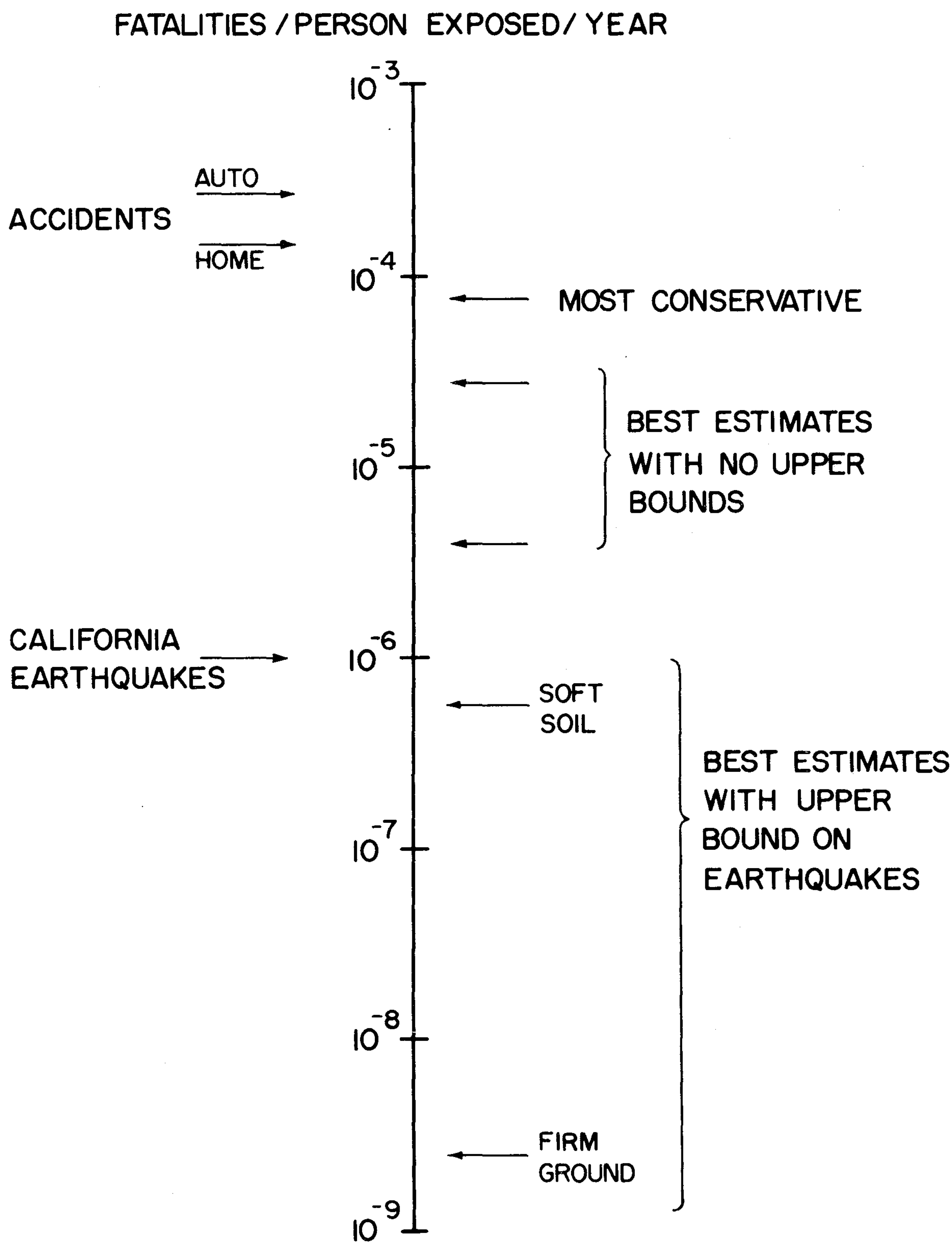


FIGURE 15 RISK OF FATALITIES

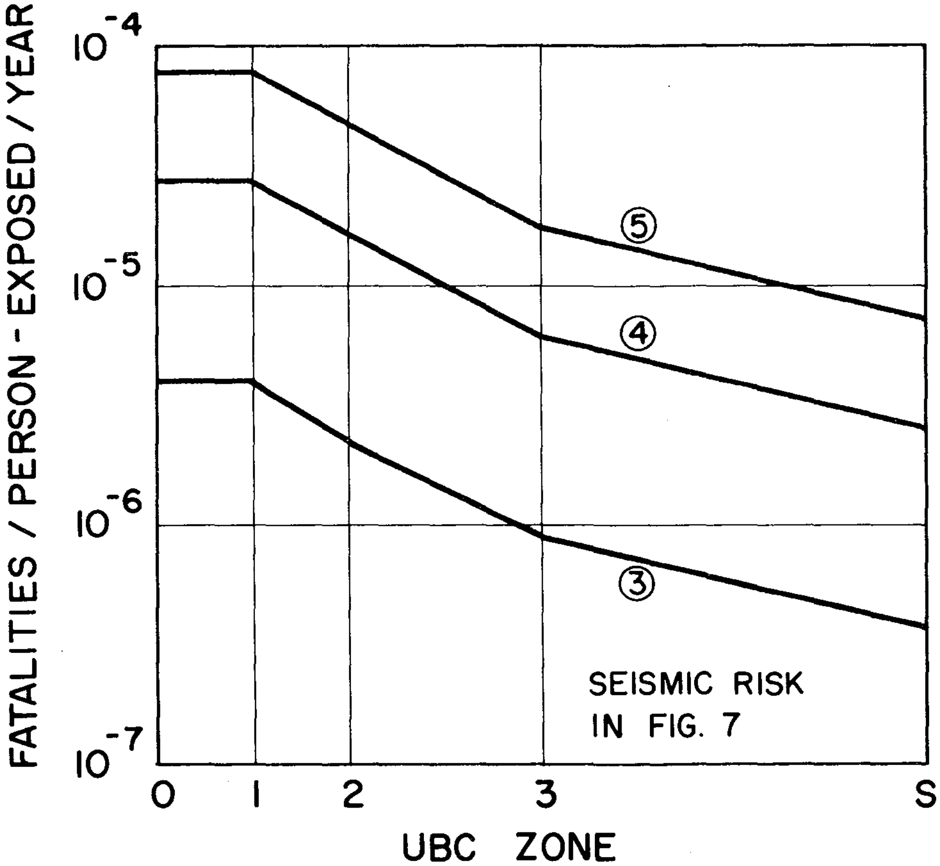


FIGURE 16 EFFECT OF DESIGN STRATEGY UPON MEAN FATALITY RATE

Appendix A

LIST OF ABBREVIATIONS

CDR_{DS}	Central damage ratio for damage state DS
DPM	Damage Probability Matrix
DR	Damage ratio
DS	Damage state
MDR	Mean damage ratio
MMI	Modified Mercalli intensity
P_{DSI}	Probability that intensity I will cause damage state DS
SDDA	Seismic Design Decision Analysis
UBC	Uniform Building Code

Appendix B

MODIFIED MERCALLI INTENSITY

The modified Mercalli intensity (MMI) scale is a method for describing the strength of earthquake ground motions at a specific location. It differs from Richter magnitude, which is a measure of the total energy released by an earthquake. Epicentral intensity can be correlated to magnitude, but intensity itself decreases with distance from an epicenter. The following is an abbreviated version of the MMI scale. The numbers in parantheses indicate very approximate correlations between peak ground acceleration and MMI.

Modified Mercalli Intensity Scale (Abridged)

I. Not felt except by a very few under especially favorable circumstances.

II. Felt only by a few persons at rest, especially on upper floors of building. Delicately suspended objects may swing.

III. Felt noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motor cars rocked.

V. Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop. (0.01g)

VI. Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight. (0.03g)

VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons dirving motor cars. (0.09g)

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimney, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Disturbs persons driving motor cars. (0.2g)

IX. Damage considerable in specially designed sturctures; well designed frame structures thrown out of plumb; damage great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken. (0.5g)

X. Some well built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.

XI. Few, if any, (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipe lines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.

XII. Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

Appendix C

SAMPLE CALCULATIONS

The calculation procedure has been described by Vanmarcke et al (1973). It is illustrated here using the seismic risk (curve 4 on Figure 7) determined using the best assumption concerning source areas but with no upper limit to the epicentral intensities in any of these areas. The curves in Figure 7 give the risk that an intensity will be equalled or exceeded. For the following computations, it is necessary to have the risk that an intensity is equalled. This is done, for example, by the following computation:

$$R(MMI=VIII)=R(MMI\geq VIII)-R(MMI\geq IX)$$

This leads to the following annual risks:

MODIFIED MERCALLI INTENSITY					
VI	VII	VII.5	VIII	IX	X
4.2×10^{-3}	0.87×10^{-3}	0.52×10^{-3}	0.45×10^{-3}	0.16×10^{-3}	0.077×10^{-3}

Mean Damage Ratio

Each entry in the following table is the product of the risk and the appropriate mean damage ratio from Figure 10. Summing each column gives the mean annual damage (in cost per year per replacement cost) for each design strategy. The discounted cost is computed as the mean annual damage divided by the discount rate, which was taken as 5%. The discounted cost has units of cost per replacement cost. Adding the initial cost premium (obtained by averaging the values in Figure 5) gives the net discounted cost in the last line. All cost ratios are expressed as decimals and not as percentage.

MMI	DESIGN STRATEGY			
	ZONE 0	ZONE 2	ZONE 3	S
VI	0.09×10^{-4}	0.07×10^{-4}	0.05×10^{-4}	0.04×10^{-4}
VII	0.26	0.17	0.12	0.10
VII.5	0.83	0.36	0.21	0.13
VIII	2.3	0.81	0.45	0.22
IX	1.6	1.6	0.72	0.34
X	0.77	0.77	0.77	0.77
\$/yr/\$	5.8×10^{-4}	3.8×10^{-4}	2.3×10^{-4}	1.6×10^{-4}
\$/\$	0.012	0.008	0.005	0.003
INIT. COST	0	0.029	0.040	0.067
NET COST	0.012	0.037	0.045	0.070

Mean Total Cost Ratio

Now Figure 12 is used.

MMI	DESIGN STRATEGY			
	ZONE 0	ZONE 2	ZONE 3	S
VI	0.1×10^{-4}	0.1×10^{-4}	–	–
VII	0.3	0.2	0.1×10^{-4}	0.1×10^{-4}
VII.5	1.0	0.4	0.2	0.1
VIII	4.3	1.3	0.5	0.3
IX	4.4	3.9	1.4	0.8
X	4.3	4.3	1.8	1.5
\$/yr/\$	14.4×10^{-4}	10.2×10^{-4}	4.0×10^{-4}	2.8×10^{-4}
\$/\$	0.029	0.020	0.008	0.006
INIT. COST	0	0.029	0.040	0.067
NET COST	0.029	0.049	0.048	0.073

Mean Life Loss Ratio

Now Figure 11 is used. The units are lives lost/life exposed/year.

MMI	DESIGN STRATEGY			
	ZONE 0	ZONE 2	ZONE 3	S
VII	0.9×10^{-7}	0.2×10^{-7}	–	–
VII.5	6.2	2.1	0.8×10^{-7}	0.4×10^{-7}
VIII	63	19.4	2.7	0.9
IX	93	77	22	2.7
X	115	66	37	22
Sum	278×10^{-7}	165×10^{-7}	62×10^{-7}	26×10^{-7}

Appendix D

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