R73-57 Structures Publication No. 380

Seismic Design Decision Analysis

Report No. 8

DAMAGE PROBABILITY MATRICES FOR PROTOTYPE BUILDINGS

by

Robert V. Whitman

October, 1973

DEPARTMENT
OF
CIVIL
ENGINEERING

SCHOOL OF ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
Cambridge, Massachusetts 02139

Sponsored by National Science Foundation
Grants GK-27955 and GI-29936

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF-RA-E-73-550	2.	3. Recipient's Accession No. PB295059

4. Title and Subtitle
Damage Probability Matrices for Prototype Buildings (Seismic Design Decision Analysis Report 8, R73-57, Structures Publication 380)

5. Report Date
October 1973

6.

7. Author(s)
R.V. Whitman

8. Performing Organization Rept. No.
Structures Pub. 380

9. Performing Organization Name and Address
Massachusetts Institute of Technology
Department of Civil Engineering
Cambridge, Massachusetts 02139

10. Project/Task/Work Unit No.
R73-57

11. Contract(C) or Grant(G) No.
(C)
(G) GK27955 GI29936

12. Sponsoring Organization Name and Address
Applied Science and Research Applications (ASRA)
National Science Foundation
1800 G Street, N.W.
Washington, D.C. 20550

13. Type of Report & Period Covered

14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

This report presents the damage probability matrices (DPM) for multi-story buildings, developed from documentation of actual earthquake damage, theoretical analysis, and judgment. This effort was specifically aimed at developing DPM for the pilot application of Seismic Design Decision Analysis to multi-story buildings in Boston. More particularly, the DPM presented in this report are intended to apply to five- to twenty-story buildings with reinforced concrete frames or shear walls or with steel frames. To provide a basis for the theoretical approach, several buildings were designed to the point of determining structural layout and the sizes and properties of structural members. In all cases, the layout of the building conformed to common practice in the Boston area and the designs complied with local code requirements including wind loading. For each building, designs were prepared using the seismic requirements of the Uniform Building Code for Zones 0, 1, 2, and 3 and for a Superzone S having twice the lateral force required for Zone 3. In general, the seismic requirements influenced the structural design only for Zones 2, 3 and S. The study was done in two stages. Firstly, an actual existing 13-story steel frame building ("pilot building") was redesigned for various levels of earthquake resistance. Then designs were prepared for a series of hypothetical buildings ("prototype buildings") having dimensions and layout typical of apartment buildings now being constructed in the Boston area.

17. Document Analysis a. Descriptors

Damage	Prototypes
Probability theory	Commercial buildings
Matrices (mathematics)	Dynamic structural analysis
Dynamic loads	Earthquake resistant structures

b. Identifiers/Open-Ended Terms
Boston, Massachusetts

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS		117
	20. Security Class (This Page)	22. Price A06-A01

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77) (Formerly NTIS-35) Department of Commerce

SEISMIC DESIGN DECISION ANALYSIS

Sponsored by National Science Foundation

Grants GK-27955 and GI-29936

Report No. 8

DAMAGE PROBABILITY MATRICES FOR PROTOTYPE BUILDINGS

by

Robert V. Whitman

October 1973

R73-57 Structures Publication No. 380

PREFACE

This is the eighth in a series of reports prepared under National Science Foundation Grants GK-27955 and GI-29936. A list of previous reports appears on the next page. There are three principal investigators for the overall study: Professors Robert V. Whitman, John M. Biggs, and C. Allin Cornell - all faculty in the Department of Civil Engineering. While this report has been written by Professor Whitman, many other staff members contributed to the studeis that have been assembled here. Professor Biggs supervised the theoretical studies which are summarized in Chapter 4. Two former staff members were responsible for collecting much of the empirical data described in Chapter 3; Dr. John W. Reed, formerly Visiting Assistant Professor of Civil Engineering and now with John A. Blume & Associates Research Division in Las Vegas, and Dr. Sheu-Tien Hong, formerly Research Associate in Civil Engineering and now with Amoco Production Company in Tulsa. Professor Cornell, and also Professor Erik H. Vanmarcke, contributed in many aspects of the study. As noted in the text, there were contributions from two engineering firms: Le Messurier & Associates of Cambridge, and S.B. Barnes & Associates of Los Angeles.

Starting with this report, a new title has been assigned to the series. Previously the series was called Optimum Seismic Protection and Building Damage Statistics. The new title, Seismic Design Decision Analysis, more aptly indicates the overall objectives of the study. To date, SDDA has been applied only to multi-story buildings. However, the same basic approach can be applied to a wide range of engineered facilities. Use of the words decision analysis, and omission of the word optimum, reflects the need to consider human and social values rather than relying solely on cost/benefit analysis.

LIST OF 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," Deparment 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.

TABLE OF CONTENTS

		Page
CHAPTER 1	Introduction	1
CHAPTER 2	Form of Damage Probability Matrix	4-7
2.1	General Form of Matrix	4
2.2	Definition of Damage States	4
2.3	Use of Modified Mercalli Intensity	5
2.4	Mean Damage Ratio	6
CHAPTER 3	Empirical Studies	8-18
3.1	San Fernando Damage Study	8
	3.1.1 Intensity Zones	9
	3.1.2 Accuracy of Results	11
	3.1.3 Discussion	11
3.2	Study of Other Earthquakes	12
	3.2.1 Caracas Earthquake of 1967	13
	3.2.2 Earthquakes in Japan in 1968	14
	3.2.3 Anchorage Earthquake of 1964	14
	3.2.4 San Francisco Earthquake of 1957	15
	3.2.5 Puget Sound Earthquake of 1965	15
3.3	Discussion of Empirical Data	15
3.4	Other Empirical Data	16
CHAPTER 4	Theoretical Studies	19-37
4.1	Outline of Studies	19
	4.1.1 Pilot and Prototype Buildings	20
	4.1.2 Ground Motion Inputs	21
4.2	Modelling of Structures	23
4.3	Response Predictions	25
	4.3.1 Fundamental Periods	27
	4.3.2 Yield Accelerations	28
	4.3.3 Maximum Ductility Factors and Collapse	28
	4.3.4 Average Interstory Displacements	29
4.4	Damage Predictions	30
	4.4.1 Prediction Procedure	32

	4.4.2 Results for Prototype Buildings	32
4.5	Discussion	33
4.6	Related Theoretical Studies	35
CHAPTER 5	Subjective Estimates of Damage	38-40
5.1	Subjective Estimates for Prototype Buildings	38
5.2	Subjective Estimates for Dwellings	38
CHAPTER 6	Selection of Damage Probabilities	41-44
6.1	Comparison of Empirical, Theoretical and Subjective Results	41
6.2	Damage Probabilities for Pilot Application of SDDA	42

TABLES AND FIGURES

APPENDICES

A	Symbols Used in Connection with Damage Probability	
B	Determination of Mean Damage Ratio from Dynamic Response	
C	Subjective Damage Probabilities	
D	Determination of Damage Probability Matrices for Pilot Seismic Design Decision Analysis	
E	References	

Chapter 1.

INTRODUCTION

Seismic Design Decision Analysis (SDDA) is a methodology for selecting the level of seismic resistance to be required for an individual structure or, through building codes, for a large group of structures. SDDA considers the cost of providing increased seismic resistance, the damage that may occur during future earthquakes and the human and social consequences of such damage. The methodology is outlined by the flow chart in Figure 1.1. Report No. 1 (Whitman et al., 1972) describes the methodology as originally conceived. A forthcoming report will present an updated version of SDDA, and will illustrate it by means of a pilot application to multi-story buildings in Boston.

One key step in SDDA is to determine what will happen to structures, designed according to some particular set of requirements, during possible future earthquakes. In SDDA, these relationships are expressed by a family of damage probability matrices (DPM). Each matrix applies to a particular type of building and particular design strategy, and gives the probability that various levels of damage will result from earthquakes of various intensities.

For low rise buildings, and especially for single-family residences, considerable information is already available concerning damage probability. The pioneer in this area has been K.V. Steinbrugge, whose efforts have been motivated by the need for suitable earthquake insurance. In many ways, techniques utilized in this report are simply modifications of techniques that Steinbrugge introduced. More recently, John A. Blume & Associates Research Division has developed rather sophisticated techniques for predicting damage caused by underground explosions; these techniques, which are closely related to those described in this report, are equally applicable to earthquake damage prediction. More complete references will be made to these studies in subsequent chapters.

For multi-story buildings, however, there is relatively little specific information available in the literature concerning multi-story buildings. An excellent summary of general experience through 1966 has been written by Steinbrugge (1970). Extensive documentation of damage caused by the 1971 San Fernando earthquake is available. (Jennings, et al., 1971; USGS/NOAA, 1971; NBS, 1971; EERI/NOAA, 1973). All of this documentation has had great influence upon earthquake engineering practice and has influenced many of the specific provisions of current building codes. From this documentation, it is possible to provide general guidance as to the damage that buildings might experience during future earthquakes. Figure 1.2 is one such synthesis of the information appearing in the literature. In this table, construction types A, B, and C represent buildings in which little or no attention is given to earthquake resistance, while types D and E represent recent design practice in California.

Figure 1.2 represents a crude DMP. The difficulty is that the words "few, many," etc. are far too imprecise for a systematic Seismic Design Decision Analysis. Unfortunately, the literature concerning past earthquakes, with a few rare exceptions, is inadequate to permit more precise, quantitative evaluations. While cases of heavy damage to multi-story buildings have been well documented, there is little or no documentation of such buildings with no damage or only very light damage. Thus, it is seldom possible to tell from the literature the actual fraction of all multi-story buildings that were heavily damaged.

This report presents DPM for multi-story buildings, developed from various sources by various techniques: Documentation of actual earthquake damage, theoretical analysis, and judgment. This effort was specifically aimed at developing DPM for the pilot application of SDDA to multi-story buildings in Boston. More particularly, the DPM presented in this report are intended to apply to 5 to 20 story buildings with reinforced concrete frames or shear walls or with steel frames. The pilot study is examining the effect of designing for the lateral forces prescribed for the various seismic zones of the Uniform

Building Code (UBC). Thus, in this report, UBC 3 will be used to denote a level of seismic resistance roughly equivalent to that required for Zone 3 in the UBC*, with similar meanings to the terms UBC 0, UBC 1 and UBC 2. Superzone S denotes a lateral force requirement twice that for Zone 3.

*The requirements for UBC 3 are essentially those which have been in effect in Los Angeles in recent years.

Chapter 2.

FORM OF DAMAGE PROBABILITY MATRIX

2.1 GENERAL FORM OF MATRIX

The general form of DPM adopted for this study is in Figure 2.1. Damage to buildings is described by a series of damage states (DS)*, while the intensity of the earthquake is described by the modified Mercalli intensity (MMI) scale. In a particular column, each number P_{DSI} in the matrix is the probability that a particular state of damage will occur, given that a 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 will differ at different locations all experiencing the same general intensity.

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

2.2 DEFINITION OF DAMAGE STATES

Figure 2.2 gives the set of damage states developed early in this study. Each DS is defined in two ways: (a) by a set of words describing the degree of structural and non-structural damage, and (b) by a

---

*Appendix A. lists and refines the symbols introduced in this chapter.

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 categorize damage states. The set of damage states in Figure 2.2 was evolved by trial and error during the early part of the current study. The particular set of damage states finally selected at that stage reflected ability to distinguish among various degrees of damage. The relationship between the word description and numerical description was chosen to be consistent with the data collected from the San Fernando earthquake experience (see Chapter 3).

As the Seismic Design Decision Analysis study progressed, it proved unnecessary to distinguish among as many degrees of damage in the range of light and moderate damage. Hence it was desirable to reduce the total number of damage states. Figure 2.3 indicates the relationship between the original (extended) set of damage states and the new (shortened) set.

2.3 USE OF MODIFIED MERCALLI INTENSITY

Modified Mercally intensity (MMI) is a rather poor way of characterizing the strength of ground shaking and further comments on this point appear in Chapter 3. However, MMI has been used thus far in this study for several reasons:

1. The historical record of earthquakes in the Eastern United States is expressed entirely in terms of MMI, and hence the seismic exposure analysis for such regions must be expressed in MMI.
2. Quantitative data concerning the intensity of ground shaking were not obtained in many of the historical earthquakes that caused building damage, and in such cases it is again necessary

to resort to MMI. (Even in the case of the San Fernando earthquake, a nearly complete set of digitized strong motion records became available only during the latter states of the current study.) 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 presented in this chapter can be converted to a quantitative intensity scale.

2.4 MEAN DAMAGE RATIO

Different types of buildings and similar buildings with different heights may experience different levels of damage from the same ground shaking. Hence, early in this study, it was though desirable to attempt to prepare a set of DPM for different groups of buildings. Moreover, for each group of buildings, a DPM is required for each different design strategy to be evaluated. Hence, it is potentially necessary to have a very large number of DPM. In addition, each DPM requires many individual probabilities, and - as will be seen - it is not possible at this time to choose the individual probabilities with great confidence. Thus, the task of preparing the needed DPM not only can mount up to staggering proportions, but also potentially involves considerable frustration.

One approach to simplifying the task is to replace the full set of probabilities in each column of a DPM by a mean damage ratio (MDR). Two methods are available for defining MDR. If the damage ratios are known for individual buildings, then

$$MDR_I = \frac{1}{n_I} \sum_{i=1}^{n_I} DR_{iI} \qquad (2.1)$$

when n_I = total number of buildings in a particular category subject to ground motion intensity I

DR_{iI} = damage ratio for the ith building in a particular category and subject to ground motion intensity I.

If only the damage state is known for each building, then MDR must be evaluated as

$$MDR_I = \sum_{DS} P_{DSI} \quad CDR_{DS} = \sum_{DS} \frac{n_{DSI}}{n_I} \quad CDR_{DS} \qquad (2.2)$$

where P_{DSI} = probability that a building in a particular category experiences damage state DS when subjected to ground motion intensity I.

n_{DSI} = number of buildings in a particular category experiencing damage state DS when subjected to ground motion intensity I.

CDR_{DS} = central damage ratio for damage state DS (see Figures 2.2 and 2.3).

These two methods will not necessarily give the same MDR_I even when DR_i is used to determine DS, although the two results should be generally similar. This conclusion was checked using the data for the San Fernando earthquake (Chapter 3). If DR_i is available for each building, then MDR_I is best determined using Equation 2.1. However if only word descriptions of damage are available for determining DS_i, then Equation 2.2 must be used.

For some problems in Seismic Design Decision Analysis, it will suffice to use only mean damage ratios. This usually is true if the only losses to be considered are the repair costs. However, when incident losses are also considered, then it is essential to know or estimate the individual probabilities for the high damage states.

Chapter 3.
EMPIRICAL STUDIES

3.1 SAN FERNANDO DAMAGE STUDY

The best way to evaluate damage probabilities is from experince during actual earthquakes, and for this reason considerable effort was made under NSF Grant GI-29936 to document damage (and non-damage) to buildings shaken by the San Fernando earthquake on 1971. This field study effort is thoroughly documented in Whitman et al. (1973a), which is Report No. 7 in the project series. An initial summary of the results appears in Whitman et al. (1973b); the final results differ somewhat from these initial results. These results include as a subset the damage information for a limited number of buildings reported by Steinbrugge et al. (1971).

The DPM developed from this effort are given in Figure 3.1, and the corresponding intensity zones appear in Figure 3.2. Damage ratios (DR) were documented for 368 buildings, out of a total of about 1600 buildings having 5 stories or more. The damage ratios were used to determine the damage states, and the MDR were computed using Equation 2.1. The buildings have been grouped in several ways:

1. By age. There were no seismic code requirements for buildings constructed prior to 1933, whereas modern code requirements have applied to buildings constructed since 1947. Buildings constructed prior to 1933 are deemed to have been designed in accordance with UBC 0, and those constructed after 1933 in accordance with UBC 3. Very few tall buildings were constructed during the depression and war years between 1933 and 1947.

2. By type of structural material. All but 10 of the buildings are known to have either steel or concrete structural systems. Several of these 10 buildings were of brick masonry construction; the structural system is unknown for the others. For most of the concrete buildings, it is not known whether frames or shear walls (or both) were employed.

3. By height. The groupings in terms of number of stories were selected primarily to give significant sample sizes. For many years, there was a restriction that buildings could not exceed 13 stories. For many of these buildings, information was also collected concerning the nature of the damage; e.g., structural, HVAC, elevators, partitions, etc. All of this information is documented in Report No. 7.

3.1.1 Intensity Zones

A special comment is needed concerning the intensity zones. There is an obvious problem with grouping damage according to modified Mercalli intensity (MMI): the observed damage influences the assigned MMI. The best approach to minimizing this problem is to base the MMI upon observations other than the specific damage being studied. This study involves damage to buildings having 5 or more stories, and hence the aim has been to base the MMI upon damage to smaller buildings and upon the reactions of people in such smaller buildings.

There is no serious question about the isoseismal in Figure 3.2 between MMI VI and MMI VII. This isoseismal was drawn by the NOAA Seismological Field Survey and conforms to the criteria mentioned in the previous paragraph.

There is, however, a problem with the isoseismal separating MMI VII and MII VIII. The isoseismal in Figure 3.2 was drawn by the M.I.T. staff early in the study, based upon a number of sources.

Subsequently, the NOAA Seismological Field Survey was asked to review the M.I.T. isoseismals, and they recommended that the VII-VIII isoseismal be moved toward the epicenter. All of the buildings listed in Report No. 7 under MMI VIII fall between the M.I.T. and NOAA versions of this isoseismal. The NOAA recommendation may be more within the spirit of the criteria stated in the first paragraph of this subsection. Nonetheless, the M.I.T. isoseismal has been retained for this study, but the category formerly called MMI VIII has been treated as being MMI VII.5.

There were four buildings within the area marked "epicentral region" that fell within the scope of this study. All were modern concrete buildings having 5 to 8 stories. They were as follows:

Building	MMI		DR
Holy Cross Hospital	X	est	75%
Indian Hills Medical Center	X		26%
Main Bldg., Olive View Hospital	X		100%
Tower, Pacoima Memorial Lutheran Hospital	IX		100%

In each of these cases, the assigned (by others) MMI was influenced greatly by the damage that occurred to the building. Within this epicentral region, old brick masonry buildings at the Veterans Administration Hospital collapsed, causing most of the deaths that occured during the earthquake.

The comments in the two previous paragraphs emphasize the difficulties inherent in the use of modified Mercalli intensity (or any other subjective intensity scale). Efforts are underway at M.I.T. to relate damage from the San Fernando earthquake to various characteristics of the measured strong ground motions. However, for the reasons discussed in Chapter 2, it is necessary to proceed with great caution in adopting more quantitative measures of intensity for use in Seismic Design Decision Analysis. Use of such quantitative measures easily may serve simply to mask the uncertainties inherent at the present time.

3.1.2 Accuracy of Results

The data upon which the DPM in Figure 2.1 are based are thought to be reasonably accurate. For some buildings, there was doubt as to the number of stories that should be used to characterize the buildings (as when, for example, there was a penthouse). Also, there is always the possibility that an error in evaluation of the damage ratio might have moved a building over the boundary into a different damage state. Finally, the addition of data for one more building would cause all of the damage probabilities to change. These various considerations suggest uncertainties in the P_{DSI} on the order of $1/n_I$.

The value of the mean damage ratio is influenced greatly by the number of buildings in the higher damage states. For example, let us suppose a sample with $n_I = 40$ and one building in damage state 6 (for which CDR = o.3). If this one building had been better detailed so as to experience damage corresponding to state 5 (CDR = 0.1), the change in the mean damage ratio would be ΔMDR = (0.3 - 0.1) / 40 = 0.005.

3.1.3 Discussion

As expected, the results reveal less damage to modern buildings designed to resist seismic forces than to older buildings with no seismic design requirements. This trend is shown most clearly in the results for MMI VII: these results have been reproduced in more compact form in Table 3.1. The MDR for the older buildings was about 2% greater than that for the modern buildings. The estimated replacement cost of the older buildings in MMI VII was about $1.6 billion. Hence the potential savings, had those buildings been designed to modern criteria, was about $32 million.

The results also show clearly the trends in damage with intensity of ground shaking.

1. For MMI VI, most buildings suffered no damage. Some buildings had parition wall cracks but only to a very limited extent.

2. At MMI VII, the pre-1933 buildings suffered significant damage while the performance of the modern earthquake-resistant buildings, both steel and concrete, was very satisfactory. Repair of cracks and partition walls accounted for most of the damage. There was very little structural damage to steel buildings, but noticeable structure damage to concrete buildings.

3. At MMI VII.5, only the post-1947 steel buildings did not suffer extensive damage. The post-1947 concrete buildings were damaged considerably, with obvious structural damage as well as costly non-structural damage.

4. For MMI VIII, modern concrete buildings received very extensive structural damage.

In general, concrete buildings had more damage than steel buildings, although there were exceptions to this generalization.

Figure 3.3 indicates the relationship between story height and damage for MMI VII. For modern buildings, the damage decreased with increasing story height, while for the older buildings the situation was more confused. Figure 3.4 illustrates the effect of story height upon modern buildings as a function of intensity. At least for buildings up to 13 stories, it is difficult to detect any systematic relationship between height and damage.

3.2 STUDY OF OTHER EARTHQUAKES

Several past earthquakes were studied so as to develop DPM. In all of these earthquakes, the quality of the data was much poorer than for the San Fernando earthquake. Only descriptions of the damage, not actual damage ratios, were available. Usually the earthquakes provided data only for a single intensity zone.

The resulting DPM appear in Figure 3.5 while Figure 3.6 gives MDR computed according to Equation 2.2. In all cases, all types and heights of buildings have been lumped together. These results must be interpreted in the light of the following comments.

3.2.1 Caracas Earthquake of 1967

As the result of earlier studies (Whitman, 1969; Seed, Whitman, et al., 1972), data were available concerning the damage state for all buildings having 8 or more stories and for many 5 to 7 story buildings. Damage was described as none, architectural, structural, or collapse.

There were no strong motion records of the ground shaking in Caracas. Based upon the damage to small residences, and reactions of people in these residences, the modified Mercalli intensity in most of Caracas was about VII -- perhaps a low VII.

Damage, especially to buildings having 9 stories or more, was unusually heavy in one portion of the city known as Los Palos Grandes. Geophysical investigations showed that the alluvium underlying the valley was unusually deep beneath this area, and it appears that the concentration of damage to tall buildings resulted from amplification of longer period earthquake motions by this very deep alluvium. Theoretical studies (see references listed above) indicate that the response of tall buildings was several times more severe in Los Palos Grandes than in the remainder of the city. Hence the damage statistics for the buildings in Los Palos Grandes having 9 stories or more have been listed under MMI VIII.

Most of the buildings with 5 or more stories were relatively new, and were required to be designed for lateral earthquake forces corresponding roughly to those for UBC Zone 2. However, the average design practice was not up to Zone 2 standards. In particular, brittle tile was extensively used for both interior partitions and curtain walls, and this meant increased non-structural damage and sometimes even increased structural damage. Virtually every building had concrete as the structural material.

3.2.2 Earthquakes in Japan in 1968

The Higashi-Matsuyama earthquake in July 1, 1968 shook the many tall buildings in Tokyo. Based upon the measured strong motions and the equivalence between MMI and the Japan intensity scale, it appears that the ground shaking was between MMI VI and MMI VII. Japanese engineers with whom this earthquake was discussed stated that there was no damage, although probably at least a few buildings experienced damage state 1.

The Tokachi-Oki earthquake of May 16, 1968, caused damage to most of the 5 and 6 story concrete buildings located in the strongly shaken region. Based upon recorded strong motions and the subjective intensity as documented by Japanese engineers, this ground motion had an intensity of between MMI VIII and MMI IX, although perhaps closer to MMI VIII.

The lateral force requirements for modern Japanese buildings are 2 to 3 times greater than those required by UBC Zone 3. Moreover, Japanese practice results in buildings that are quite stiff. Hence, the design practice is better than that indicated by Superzone S.

3.2.3 Anchorage Earthquake of 1964

Very good damage descriptions were available for the 14 buildings with 5 or more stories in Anchorage at the time of the great Alaskan earthquake in 1964. Buildings built prior to 1955 had requirements corresponding to UBC Zone 2, while Zone 3 requirements had been in effect since that date. Most of the buildings had concrete shear walls as their principal lateral force-resisting system.

There were no strong motion instruments in Anchorage at the time of the earthquake. The probably peak ground accelerations as deduced by several indirect methods would correspond to MMI VII or MMI VII.5. However, the duration of the shaking was unusually long so that once buildings yielded a considerable degree of damage developed. The officially assigned intensity was

MMI IX to MMI X, but this choice was influenced by the serious landslides that occurred in several parts of the city. For these reasons, it is especially difficult to assign an MMI for purposes of this study; MMI IX would seem a reasonable choice.

3.2.4 San Francisco Earthquake of 1957

An attempt was made to develop DPM for this small earthquake. A satisfactory list was prepared for buildings having 5 or more stories and existing at the time of the earthquake. However, information regarding damage is very scant. The DPM in Figure 3.5 has been prepared on the assumption that only the buildings described in the published literature were damaged; it is likely that more buildings had at least light damage.

All of the city was in MMI VI. Most of the buildings had been designed to lateral force requirements corresponding to UBC Zone 2.

3.2.5 Puget Sound Earthquake of 1965

City building officials in Seattle had prepared excellent documentation of damage caused by this earthquake. Damage to each building was classified as none, light, medium, or heavy, and the DPM in Figure 3.5 were reconstructed from this information. No seismic design was required for buildings built prior to 1942; the regulations in effect since that date correspond roughly to UBC Zone 2.

3.3 DISCUSSION OF EMPIRICAL DATA

All of the data for MDR have been plotted against MMI in Figure 3.7. Overall, there is an encouraging degree of consistency, especially when one remembers that some of the data for MDR are relatively crude and that MMI is only a crude indicator of the intensity of ground shaking.

Despite the overall consistency, the data are inadequate for the purpose of drawing curves for MDR. The data are especially scant for the higher intensities. Moreover, it must be kept in mind that the groupings UBC 0, etc. are only convenient ways to indicate a general level of required lateral force resistance and that design practice may vary greatly within a grouping. Thus the data are not necessarily applicable to buildings in, say, Boston without further interpretation and judgement.

Certainly it would help to have available additional quantitative data such as that accumulated from the San Fernando earthquake. However, it must be recognized that the strictly empirical approach described in this section can be of only limited value so long as a subjective scale is used to describe the intensity of ground shaking.

3.4 OTHER EMPIRICAL DATA

As mentioned in Chapter 1, empirical damage probability data are available for low rise buildings, and it is of interest to compare these with the data presented here for multi-story buildings.

After both the Santa Barbara (1925) and Long Beach (1933) earthquakes, surveys were conducted to determine the percentage of buildings sustaining different levels of damage. Table 3.2 presents damage probabilities as observed in the city of Compton during the Long Beach earthquake, in a region where the modified Mercalli intensity was XIII or IX. The MDR in Table 3.2 were calculated by the present writer using CDR in the lower part of each range of DR. These probabilities were based upon market values, and the MDR would be smaller if replacement values were used. The results show the great importance of the type of construction. The fewer total and partial collapses of residential (as opposed to commercial) masonry wall construction apparently resulted from the greater use of wood framing in the interior of the residences. The results also indicate the general effectiveness of wood frame construction.

The general picture given by Table 3.2 has been confirmed by subsequent earthquakes. Extensive data have been gathered from the 1952 Kern County earthquake (USCGS,1967) and the 1971 San Fernando earthquake (Steinbrugge et al., 1971). Table 3.3 gives damage probabilities for wooden frame buildings during the 1971 earthquake, in an area having MMI IX to X. The damage indicated in Table 3.3 is greater than in the last column of Table 3.2, in part, presumably, because the shaking was more intense and in part because some dwellings were damaged by faulting of the ground directly beneath the dwelling. More complete information concerning damage to residences during the San Fernando earthquake will appear in a forthcoming report (EERI/NOAA, 1973).

Crumlish and Wirth (1967) have tabulated mean damage ratios for school buildings in California and Washington. These results are indicated by the solid points in Figure 3.8 (the data in Figure 3.7 has been replotted as open points. The MDR for all schools in Anchorage is also plotted in the figure.) In California, there are data both for buildings constructed both before and after passage of the Field Act which required seismic design of school buildings together with control of construction. In the Puget Sound area, there are data for buildings constructed both before and after 1949 when the building practice for schools was upgraded to correspond roughly to UBC 3. The usual inconsistencies appear in these data for low rise buildings. However, in both sets of data, the beneficial effects of seismic design are evident. The values of mean damage ratio was generally consistent with those for high rise buildings. Again, the MDR for the schools was based upon actual cash value rather than upon replacement cost.

John A. Blume & Associates Research Division has collected considerable data regarding the damage caused to small buildings by underground nuclear tests, and a report containing these results is forthcoming in the October issue of the Bulletin of the Seismological Society of America. Most of these data are ground motions corresponding to low intensities (MMI VI or less), and it appears that the mean damage ratios are generally consistent with those in Figures 3.7 and 3.8. A detailed comparison will be made once the data are published.

In general, these results for low buildings are similar to those for high rise buildings. However, at the higher intensities of ground shaking, the damage to wood frame dwellings is distinctly less than that to multi-story buildings.

Chapter 4.
THEORETICAL STUDIES

4.1 OUTLINE OF STUDIES

Empirical approach described in Chapter 3 potentially provides the best and most reliable information concerning damage probabilities. However, it has been seen that the data available today are inadequate, especially for intensities greater than MMI VII where the type of structural system and other details of the design may be expected to have a significant influence upon the damage caused by an earthquake. Moreover, the nature of the earthquake engineering problem is that more data cannot be gathered rapidly -- for humanitarian reasons, one certainly hopes that there will not be a rapid succession of major earthquakes. While development and use of a quantitative scale of intensity will alleviate some of the problems encountered in Chapter 3, this step alone cannot overcome a basic lack of data.

Whenever a strictly empirical approach fails, it is natural to turn to theoretical methods. During the past decade, much effort has been devoted to theoretical analysis of the dynamic response of buildings during earthquakes. However, the use of such theory for quantitative estimates of damage is still in its infancy.

Any procedure for the theoretical prediction of damage consists basically of three steps:

1. Modelling a structure or group of structures
2. Computing response to earthquake inputs of different intensity
3. Relating expected damage to computed response.

The difficulties arise in the first and third of these steps, and the following sections describe initial attempts to cope with these difficulties.

Theoretical predictions of damage should employ probabalistic considerations. The exact nature of future ground shaking is uncertain; there is uncertainty in the modelling of a structure and hence in the prediction of its response, and there are uncertainties in estimates of damage based upon predicted response. However, for this initial attempt to predict damage by theorectical procedures, the treatment is almost entirely deterministic.

4.1.1 Pilot and Prototype Buildings

To provide a basis for the theoretical approach, several buildings were "designed" to the point of determining structural layout and the sizes and properties of structural members. In all cases, the layout of the building conformed to common practice in the Boston area and the designs complied with local code requirements including wind loading. For each building, designs were prepared using the seismic requirements of the Uniform Building Code for Zones 0,1,2, and 3 and for a Superzone S having twice the lateral force required for Zone 3. In general, the seismic requirements influenced the structural design only for Zones 2,3 and S.

This study was done in two stages. First, an actual existing 13 story steel frame building (the so-called "pilot building") was redesigned for various levels of earthquake resistance. The redesigns are described in Report No. 2 (Leslie and Biggs, 1972). Then designs were prepared for a series of hypothetical buildings (the so-called "prototype buildings") having dimensions and layout typical of apartment buildings now being constructed in the Boston area. Designs were prepared for story heights of 6, 11 and 17 stories, and using steel moment resisting framing (SMRF), concrete moment resisting framing (CMRF) and concrete shear walls (CSW). These designs are partially described in Report No. 4 (Biggs and Grace, 1973) and will be described in more detail in a forthcoming report. There was considerable input to the study of the pilot building from Le Messurier Associates, a structural engineering firm in Cambridge, Massachusetts. The design studies for the prototype buildings was undertaken by Le Messurier Associates. The designs of the prototype buildings were reviewed by S.B. Barnes & Associates, a structural engineering firm in Los Angeles with long experience in earthquake engineering.

The pilot and prototype buildings were also used to study the effect of increased seismic force requirements upon the initial cost of the buildings. For the pilot building, the conclusions are stated in Report No. 2. The results for the prototype buildings will be presented in a forthcoming report.

4.1.2 Ground Motion Inputs

For purposes of dynamic structural analysis of this study, it was necessary to describe quantitatively the ground motion that might be associated with possible future earthquakes in Boston. A smooth response spectrum and an artificial accelerogram were used for this purpose.

Based on the review of the seismic risk analysis (as described in forthcoming reports), it appeared that any motion of consequence is almost certain to be caused by a moderately sized, nearby earthquake. Although no strong motions have been recorded at Boston, on the basis of observations of such records at other sites, it was estimated (a) that the strong motion duration will be relatively short, about 10 seconds perhaps, and (b) that the relative frequency content will be high. This is interpreted to mean that one might expect a peak ground velocity of about 3.6 in/sec. with a motion whose peak ground acceleration was 0.1g. (Scaled to the same ground acceleration, the 1940 N-S El Centro record would have a ground velocity about 5 in/sec; this latter ratio is commonly used for design motions associated with strong earthquakes at moderate distances.) The associated peak ground displacement for a 0.1 g accelerogram was chosen to be 1.5 in.

(compared with 3.6 in. for a scaled El Centro Record). This value seemed consistent with observed records and response spectra for motions of the type anticipated here.

Methods for constructing smooth design response spectra from these three descriptors (peak ground acceleration, velocity, and displacement) have been under development and have been applied for several years (Newmark and Hall, 1973). The design spectra shown in Figure 4.1 were developed for Boston using these methods with mean "amplification factors" based on statistical data derived by Garcia and Roesset (1970) and by Vanmarcke and Cornell (1972) from a number of historical records.

For non-linear structural analyses, accelerograms (or "time histories" of ground motion) are needed. No single historical accelerogram is appropriate for these purposes. To provide a motion consistent with the design response spectrum, and "artificial time history" was produced using ground motion simulation methods. (Those used here were developed at M.I.T.; see, for example, Hou, 1968, and Cornell, 1970). This motion was then modified locally until the response spectra it produced were similar to the design spectra and themselves relatively smooth. Such smoothed spectra are desireable in studies such as these because they help avoid the extreme sensitivity of calculated peak response to small changes in structural period that characterize the use of historical records but that cannot be accepted in design studies for a spectrum of future earthquakes. The response spectra of this artificial accelerogram are shown in Figure 4.2. The resulting motion had a calculated peak acceleration and velocity of 0.1g and 2.4 in/sec, respectively. Thus the peak velocity was somewhat less than the desired value, but the response spectra for the time history still gave a reasonable match to the smooth design spectra in Fig. 4.1.

Finally, it is necessary in this study to relate modified Mercalli intensity to a corresponding set of design motions (i.e., response spectra and accelerogram). Unfortunately, there is not a simple functional relationship between MM site intensity and some motion intensity measure such as peak ground acceleration. For this study, the data in Figure 4.3 correlating these two factors were interpreted as follows. If a particular MM intensity is observed in a given area, the ground acceleration at any site in the area may be above, at, or below the average value over many sites. Therefore, a nominal value of peak ground acceleration was assigned to various intensity levels, but in doing expected damage studies it was presumed that, given a particular predicted intensity, an acceleration value one level below or one level above the nominal value might be experienced. Consistent with Figure 4.3, these upper and lower values were each assigned probabilities of 25% and the nominal value was assigned a weight of 50%. Two sets of nominal values were used, one corresponding to the correlation suggested by Gutenberg and Richter (1956) and an alternate version developed by the writer. These nominal values appear in Table 4.1. For example, if an MM VI occurs, the peak ground acceleration might also be measured when an MM intensity VII is assigned to an area or even when an MMI VIII is assigned.

4.2 MODELLING OF STRUCTURES

When developing a mathematical model of a structure, several key decisions must be made:

1. Whether to model as a two- or three-dimensional structure, or to simplify as a one-dimensional structure with all mass at each floor level lumped together.

2. How much of a non-structural system to include within the model.

3. Whether to include non-linear behavior and if so which kinds of non-linearites and in how much detail.

4. What material properties (stiffness and strength) to assign to the various elements in the model.

For practical work, it is essential to reach a compromise between reality and simplicity. While today it is possible to write computer programs that will handle many degrees of freedom and complex non-linearites, the cost of using such complicated programs inhibits the investigation of many parameters.

In connection with Seismic Design Decision Analysis, it was decided that it would be essential to have a computer program based upon one-dimensional models and capable of handling a wide variety of non-linear behavior. The development and scope of the program developed for this purpose is described by Anagnostopoulos, Roesset and Biggs (1972) in Report No. 3. The input to the program consists of individual member properties; the program then computes the required parameters for the dynamic model. The program subsequently was extended to include a response spectrum analysis to be used for elastic analysis.

Frame structures are modelled as "shear" buildings; i.e., a close-coupled set of equations is used. For the prototype buildings, the story resistance function of a single frame is assumed to be elasto-plastic (for the pilot building, a tri-linear function was used). The stiffness of a single frame is computed by a procedure which takes into account the stiffness of the individual columns and beams in the floors above and below the story. The ultimate resistance is computed by assuming a story shear mechanism in which hinge moments are taken to be the smaller of the column and beam moment capacity. The stiffnesses and ultimate resistance of the individual frames at each story level are then summed to give the overall stiffness and resistance at that story. Thus, if there are frames with different ultimate resistances, the overall story resistance function is not simply elasto-plastic.

Far-coupled equations are used to model shear walls. The moment resistance function at any floor in a wall is assumed to be elasto-plastic. However, the shear resistance is assumed to have no ductility; i.e., if the shear capacity is exceeded the wall is assumed to have failed and to provide no resistance thereafter.

The total internal forces at any time are taken to be the sum of those provided by the individual walls. The forces developed in the frames of shear wall buildings, although small, are superimposed on those developed by the shear walls. The frame action becomes significant if the shear walls fail.

In the case of the pilot building, an attempt was made to account for the effect of the masonry block walls surrounding the elevators. It was concluded that these walls would break and cease to offer resistance after taking loads that were quite small compared to the strength that the structural system, and, correspondingly, that for even moderate earthquakes inclusion of these walls had very little effect upon response. For the other buildings, the effect of the masonry block walls was not included.

For the concrete frame buildings, the moment of inertia of girders was taken as 40% of that for the gross section, to allow for cracking. For concrete columns, the full cross section was assumed to be effective. The moment of inertia of shear walls was taken as 50% of that for the gross section. In the case of steel buildings, the possible stiffening effect of fire-proofing concrete over the steel members was ignored.

It was assumed that the lumped masses at all floors, including the roof, were equal. For the prototype buildings, the overall effective unit weight was 20 pcf for the concrete structures and 12 pcf for the steel frames. These unit weights are large by some standards, owing to the use of masonry block walls.

Constant model damping was assumed, using damping ratios from 2% to 5%.

4.3 RESPONSE PREDICTIONS

Using the modelling procedure and computer program described in the preceeding section, predictions were made concerning the dynamic response of pilot building and some of the prototype buildings. The following specific cases were analyzed.

13-story steel frame pilot building; both directions. There were 3 virtually identical frames in one direction and 4 in the other.

6-story concrete frame (CMRF-6); lateral resistance is provided by the two exterior frames.

11-story steel frame (SMRF-11); long direction. Lateral resistance is provided by two exterior frames.

11-story concrete frame (CMRF-11); short direction. Lateral resistance is provided by 11 frames, not all of which are identical.

11-story concrete shear wall building (CSW-11); short direction. The number of shear walls varies according to the design strategy.

17-story shear wall building (CSW-17); short direction. Again the number of shear walls varies with the design strategy.

With the exception of CSW, all designs (and hence predicted responses) were identical for UBC Zones 0 and 1.

For most of the buildings, the intensity of earthquake causing first yielding was determined using response spectrum analysis, and a single non-linear analysis was made using a ground motion with a peak acceleration of 0.27 g. For a few of the buildings, non-linear analyses were made for ground motions with smaller peak accelerations and linear analyses were carried out with time history input. The nature of the response spectra and time histories is described in section 4.1.2.

The results of these analyses have been presented by Anagnostopoulos, et al. (1972, Report No. 3) and by Biggs and Grace (1973, Report No. 4). There still are some serious questions concerning the adequacy of the models employed, and the results should be viewed with some caution. They are summarized here primarily to illustrate the types of predictions that are of potential use.

4.3.1 Fundamental Periods

The computed fundamental periods are listed in Table 4.2. In many cases, these periods were surprisingly large. The factor relating period T and number of stories N for buildings designed to UBC Zone 3 requirements are given in the following table.

Building	T/N
Pilot - X	0.32
- Y	0.25
CMRF - 6	0.35
SMRF -11	0.22
CMRF -11	0.19
CSW -11	0.12
CSW -17	0.14

For comparison, analysis of accelerograph records atop modern buildings in Los Angeles during the San Fernando earthquake indicated this factor was typically 0.1 for concrete buildings and 0.2 for steel buildings.

Non-structural elements are known to contribute significantly to stiffness of buildings for small levels of vibration. This contribution was studied for the case of the pilot building. Including the stiffness of the block walls surrounding the elevators reduced the fundamental periods very significantly; to 1.53 and 1.95 seconds for the building as designed (UBC 0). The corresponding periods measured on the actual building were 1.7 and 2.0 seconds. However, as experience in California has proven, such walls should not contribute so much stiffness for the level of strain occurring during a potentially damaging earthquake.

It must be concluded that the periods of the buildings analyzed in this study are significantly larger than buildings actually built in California to similar requirements. This is especially true for some of the concrete buildings. The difference apparently results in at least part, from the use of a reduced moment of inertia in the computation of the stiffness of girders and shear walls, and from the use of heavy masonry block walls.

4.3.2 Yield Accelerations

In Figure 4.4, the acceleration level required to just cause yielding at some point within the structure has been plotted against the design strategy. The horizontal scale is proportional to the required earthquake design loads. Except for the shear wall buildings, the increases are only small to moderate.

This result is associated with the relatively long periods of the buildings. Strengthening a building means that a building will be stiffer. For any building with $T > 0.5$ seconds and for the input used in these analyses (see Figure 4.1), increasing stiffness means that larger forces will occur in the building for any particular intensity of ground motion shaking (see Figure 4.5a). This is especially true for buildings with $T > 2.5$ seconds. Thus, for many of the analyzed buildings, the induced force increases almost as rapidly as the required design lateral force.

Greater benefits would be expected for buildings that are stiffer.

4.3.3 Maximum Ductility Factors and Collapse

Table 4.3 summarizes the maximum computed ductility factors for the various buildings and design strategies. For the framed buildings, the maximum ductility factor is the largest ratio of the peak interstory displacement in a story to the yield interstory displacement in that same story; no attempt was made to determine ductility factors for individual members. For the shear wall buildings, the ductility factor is the ratio of the peak displacement at the top of the structure to the top displacement in the earthquake that just causes yield. All results in Table 4.3 are for a peak acceleration of 0.27 g.

Several observations may be made from the results for maximum ductility factor, based upon conventional beliefs concerning the relation of ductility factor to damage:

1. Increasing lateral force requirement does decrease the maximum required ductility, although in some cases the decrease is not as great as might be expected. Where a marked decrease in ductility does occur, it is because some particularly weak story has been strengthened as a result of the increased lateral force requirement.

2. Certain entries in Table 4.3 have been enclosed by a solid box; these entries represent probable partial or total collapse. Shear walls generally are not (although they could be) reinforced to remain intact at ductility factors much greater than 2.

3. Certain other entries in Table 4.3 have been enclosed by a dashed box. These entries indicate possible partial or total collapse unless good reinforcing or detailing practice has been followed.

4. In all of the buildings that rely upon a few frames of considerable length (pilot building, CMRF-6, SMRF-11) an earthquake of this intensity would cause considerable damage regardless of the design strategy.

In general, it was noted that the UBC requirements do not necessarily lead to a structure in which all parts of the structure work with equal efficiency in resisting earthquake forces. This is an area for possible research.

4.3.4 Average Interstory Displacements

Figure 4.6 shows the effect of design strategy upon interstory displacement during an earthquake with a peak acceleration of 0.27 g. (As will be discussed in the next subsection, this parameter probably is most closely related to damage provided that the building does not collapse.) To obtain the values plotted in this figure, the interstory displacements for the several stories in a building have been averaged.

The results indicate that increasing the design lateral force may have little beneficial effect upon average interstory displacement. Indeed, in some cases an increase may result. (Although not shown, results for the pilot building also substantiate this conclusion.) This lack of benefit also is associated with the relatively large periods of the buildings analyzed (see Figure 4.5b) and the conclusion should not be extrapolated to all buildings.

The average interstory displacement at first yielding of the structure may either increase or decrease as a result of increasing the design lateral force, although again the change generally is not very significant. These average interstory displacements at yield are about 0.01 - o.02 feet for the shear wall buildings, about 0.02 ft. for the steel frame building, and 0.03 - 0.06 ft. for the concrete frame buildings.

4.4 DAMAGE PREDICTIONS

Different aspects of dynamic response determine the degree of damage sustained by different portions of a building. Damage to the structural frame itself is determined by the amount of yielding that occurs; that is, by the maximum interstory distortion. Damage to curtain walls and partitions also should be related primarily to interstory displacement and the same should be true for portions of the mechanical and plumbing systems within the building. For light fixtures and other portions of mechanical, electrical and plumbing systems, the onset of damage will be related to peak floor acceleration; the same is true for contents such as furniture. However, the actual degree of damage (the amount that a fixture will distort or furniture will move) to such components is heavily influenced by the peak floor velocity and peak floor displacement. All in all, interstory displacement appears to be the single response parameter most closely associated with degree of damage.

Dynamic response analyses have been made for a few of the buildings that were damaged during the San Fernando earthquake (EERI/NOAA, 1973). As part of the current study, detailed floor-by-floor damage cost was obtained for these buildings. These costs were correlated to several aspects of the computed dynamic response, and it was found that cost correlated best with interstory displacement (Czarnecki and Biggs, 1973; Report No. 5). The resulting data are presented in Figure 4.7. While scattered, the results appear to fall into two groups: One group with larger damage from two buildings with many stiff and brittle partitions not isolated from the structural frame, and a second group with smaller damage from buildings in which there were either few partitions or with flexible and/or isolated partitions.

Using the average interstory displacements reported in subsection 4.3.4 (figure 4.6 and text), the following conclusions might be reached:

1. When the prototype buildings yield, the mean damage ratio might range from 0% to 8%, depending primarily upon the number and type of partitions in the building.
2. For a ground motion with a peak acceleration of 0.27 g, MDR might be on the order of 6% to 10%.

These conclusions apply only to buildings with construction similar to these in Los Angeles. These limited data are insufficient to permit damage to be estimated from response for a wide range of intensities and building systems.

To bridge this gap, data from tests on full size building bays have been used to estimate damage vs. distortion for a variety of curtain wall and partition systems. The approaches to developing these models are also described in Report No. 5; some typical curves are shown in Figure 4.8. In Figure 4.8, damage ratio refers to the particular component; that is, a damage ratio of 100% means that the cost of damage to this component equals the costs of the component. Theory plus test data have been used to construct similar curves for various structural systems. Finally, a similar relation was developed to account for damage to other parts of a building. All these relations must be regarded as preliminary and crude.

4.4.1 Prediction Procedure

To calculate the damage corresponding to a computed dynamic response, the first step is to select the appropriate set of curves of damage ratio vs. distortion. Curves are needed for:

The appropriate structural system

Glazing

Drywall partition and/or the appropriate masonry wall partition

The appropriate curtain wall

The remaining components of the building

The damage ratio for each of these components is determined using the computed average interstory displacement. Then the individual damage ratios are weighted in accordance with the contribution of each component to the total cost of the building. At this stage, the procedure gives MDR as a function of peak acceleration. To convert from acceleration to intensity, use is made of the relations suggested in subsection 4.1.2.

This procedure is described in detail in Report No. 5, and an example is presented in Appendix B. The procedure was applied to several buildings damaged by the San Fernando earthquake, and was found to provide a crude but encouragingly satisfactory "fit" to these data.

4.4.2 Results for Prototype Buildings

Figure 4.9 gives MDR as a function of MMI for concrete frame buildings. (These results are an average for the two concrete frame buildings for which response predictions were made. The alternate relation between intensity vs. nominal acceleration, described in section 4.1.2, was used.) Figure 4.9 indicated that the design strategy appears to have little influence upon damage, except at MMI V. This follows from the results in Figure 4.6 which show that, for the prototype buildings, design strategy has little effect upon average interstory displacement.

Figure 4.10 compares, for UBC 0 and UBC 3, the effect of structural system. There is little apparent effect.

4.5 DISCUSSION

It must be reemphasized that theoretical damage prediction methods still are in a very early stage of development. It would seem that the theoretical method can now provide satisfactory estimates for the size of earthquake that causes first yield, at least rough estimates for the damage associated with this general intensity of ground motion. However, the theoretical procedure described in this section appears:

1. To overestimate damage for earthquakes of small intensity, as a result of neglecting the strength and stiffness of the non-structural components.
2. To underestimate damage during earthquakes of large intensity, apparently for two reasons: (a) overestimating the ductile capacity of structures, and (b) underestimating the damage that occurs to "other components" once major structural damage occurs.

Nonetheless, it is believed that the theoretical models can be improved and made useful for damage prediction. It will be necessary to maintain a close tie between theory and empirical observation.

In the short run, the greatest benefit of the theoretical approach will be to indicate the effect of changes in design strategy upon expected damage. The results presented in this section have already shown that, at moderate intensities of earthquake for which average interstory displacement is a reasonable indicator of damage, increasing the required design lateral force will have only small benefit to buildings of 6 to 20 stories designed according to current practice in Boston.

In the longer run, the greatest potential benefit of the theoretical approach will be in studying the conditions that can produce collapse of a building, and of the effectiveness of design rules in reducing the possibility of collapse. Empirical data concerning the collapse condition will (hopefully) always be sparse, and yet knowledge concerning condition is vital to Seismic Design Decision Analysis. Non-linear analysis is essential for theoretical predictions concerning collapse, and, despite the work that has been done along these lines, much further effort is needed to make non-linear analysis a practical tool.

It is of course, essential to recognize that there are benefits of requiring increased design lateral forces that are difficult to quantify via theoretical analysis. For example:

1. The larger the design lateral forces, the more an engineer must think carefully about his overall design concept in order to make it economical. This more careful thought will usually lead to a more effective design.

2. Increasing design lateral forces often leads to a more redundant structure which will be less likely to collapse when large damage occurs. This is particularly true for shear wall buildings, where greater lateral forces generally mean that additional shear walls must be added.

3. Increasing design lateral forces will mean that more details of the building must actually be designed. For example, with increased lateral forces more structural connections will be designed instead of simply using "standard" connections. This greater attention to detail will mean a stronger and more resistant building.

4.6 RELATED THEORETICAL STUDIES

Blume and Munroe (1971) have developed a procedure for predicting damage from ground motion, called the Spectral Matrix Method. While the procedure was originally developed for studies of damage caused by underground explosions, it is equally applicable to earthquake damage prediction as well. The procedure employs theoretical equations and concepts for linking ground motion to structural response and structural response to damage, and specifically accounts for the uncertain nature of both the ground motion and the structural response.

The Spectral Matrix Method specifically involves the following assumptions and steps:

1. DR (called Damage Factor by Blume) is assumed to be linearly related to the required ductility factor, being zero when the structure remains elastic and reaching 100% at collapse.

2. The required ductility factor is related to the ratio of demand D to capacity C. Demand is represented by the spectral velocity, at the fundamental period of the building, of the ground motion. Capacity is represented by a spectral velocity corresponding to first yielding of the building. (Both D and C could just as well have been expressed by spectral accelerations.) The relation of ductility factor to demand and capacity depends on the nature of the force-deflection relation following yield.

3. Combining the above assumptions and theoretical results leads to curves of Damage Factor (DR) as a function of demand and capacity. Several such curves are shown in Figure 4.11, corresponding to several assumptions concerning the inelastic force-deflection relation.

4. Both demand and capacity are treated as random variables.

The report suggests forms for the probability density functions and some tentative values for the coefficients describing the variation. The possibility of some correlation in the demand at nearby buildings is included in the model.

5. Cumulative probability curves are computed by using a Monte Carlo procedure with random numbers in the probability density functions for demand and capacity. Figure 4.12 shows a typical result.

Such curves may be computed for an individual building or for groups of similar buildings. By subdividing the area under study into regions having different predicted ground motions, and subdividing the buildings in each region by type, total damage estimates may be constructed.

The information contained in a cumulative probability curve is related to the information in a column of a damage probabiltiy matrix. (If the information in the column of a DPM were expressed as a continuous function, it would be the inverse slope of the cumulative probability curve.) Both indicate the distribution of damage caused by a general level of ground shaking, taking into account the uncertainty both in the actual ground motion and in the response of a building. Thus Blume's method for obtaining this information resembles the theoretical approach described in the earlier parts of this chapter, with the extra feature that probability theory has been used to distribute the damage probabilities among the damage states.

Figure 4.13 shows mean damage ratio as a function of intensity using only average demand and capacity. A capacity of 18 in/sec was used, based upon Blume's recommendations for high rise buildings. A ground motion with a peak acceleration of about 0.25 g would be required to give a demand of 18 in/sec (remember demand is expressed by spectral velocity, at say 5% damping).

Using the curves in Figure 4.11 for an assumed maximum ductility ratio of 6, and using the alternate motion-intensity correlation from subsection 4.1.2, the curve in Figure 4.13 was constructed. The curve for UBC Zone 2 from Figure 4.9 is plotted for comparison. Several comments may be made concerning the comparison:

1. The capacities suggested in the Blume report may be too large. The M.I.T. theoretical studies for the prototype buildings (section 5.5) would suggest capacity values only half of those suggested by Blume.

2. More important, the average damage factor predicted by the Spectral Matrix Method is considerably greater than the damage factor based upon average demand and average capacity. For example, more than 50% of the buildings in a sample may have no damage, but the average damage factor may still be much greater than zero if a small fraction of the remaining buildings are severely damaged. One point plotted in Figure 4.13 using results in the Blume-Munroe report.

Considering these comments, the agreement really is encouragingly good. The probability density functions for demand and capacity as proposed in the Blume-Munroe report seemingly lead to overestimates for the mean damage ratio for a given average demand, and Blume is investigating the use of different functions.

Chapter 5.

SUBJECTIVE ESTIMATES OF DAMAGE

5.1 SUBJECTIVE ESTIMATES FOR PROTOTYPE BUILDINGS

Neither the empirical nor the theoretical approach, nor the two approaches taken together, are entirely adequate at this time for establishing adequate damage probability estimates -- although both have yielded valuable information and insights. Hence it becomes necessary to add a third approach in which the subjective judgement of experts -- based upon all their past experience -- is used. This approach helps to fill gaps in the other estimates, to resolve conflicts and to take into account the difficult -to-quantify benefits of designing in accordance with earthquake code provisions.

Messrs. S.B. Barnes and C.W. Pinkham, both of S.B. Barnes & Associates in Los Angeles, agreed to prepare damage probabilties matrices for each of the prototype buildings. In addition to the schematic designs for these buildings, which they had already reviewed, they were furnished with the dynamic response calculations and with the damage statistics from the San Fernando earthquake. They prepared their DPM independently, although with some mutual initial discussion. Results from this effort are summarized in some detail in Appendix C.

A few of the results are presented in Figures 5.1 and 5.2. The results in Figure 5.1 indicate some significant benefit at higher intensities as the result of increasing the design lateral forces. At MMI IX, while a small percentage of buildings might collapse, the expected performance of concrete frames is rather good. Figure 5.2 suggests that the specific structural system has relatively little effect upon MDR.

5.2 SUBJECTIVE ESTIMATES FOR DWELLINGS

A very thorough subjective approach was utilized in the insurance-motivated study of wooden frame dwellings by USCGS (1969).

The approach is described in detail in Appendix A of the reference; that appendix was prepared by Steinbrugge, McClure and Snow.

As described in the reference, the procedure followed in compiling damage data involved the following steps:

1. Determine the most appropriate dwelling construction components which can be isolated for damage and repair cost analysis. Considerable care was used to ensure that the categories did not become more complex than the input data warranted. The main categories were: structural, interior finish, exterior finish and chimneys. These main categories were subdivided according to factors that would affect the degree of damage. For example, the exterior finish was subdivided as plaster (stucco), masonry veneer and wood finish, or their damage susceptibility equivalents.

2. Define appropriate degrees of damage to the selected dwelling construction components. Definitions corresponding to slight, moderate, severe and total damage were prepared, although only some of these damage states were used for some of the construction components. For example, only moderate and severe states were defined in connection with structural damage.

3. Determine the variations in the degrees of damage for each construction component as a function of the earthquake intensity. In order to obtain the best possible results, a group of consultants was retained with the duty of evaluating the damage patterns to wood frame buildings in terms of each modified Mercalli intensity. For example, it was the consultants' problem to determine the percentage of dwellings which would have no damage, slight damage, moderate damage and severe damage for each type of interior finish. These consultants were

structural engineers with substantial experience in field investigations of numerous earthquakes.

4. Determine the repair cost as a function of the degree of damage to each construction component and as varied by dwelling floor area. These repair costs were determined through the use of 5 persons/organizations, all skilled in repair procedures and/or repair cost determination. The estimated repair costs varied in a manner similar to that which might be expected in competitive bidding on construction projects. The final figures used in the study was near to the average figures.

5. Synthesize data obtained in steps 1 through 4 into a form and format usable for computer operations.

While the data developed by this process clearly was influenced very heavily by the accumulated experience during earthquakes, the procedure basically was subjective in character.

There are many similarities between the procedure outlined in the preceeding paragraph and that followed in this report. Step 2 defined damage states by words, while step 4 associated costs with these damage states. Step 3 determined the probabilities of these damage states as a function of MMI. Step 1 described a number of building sub-systems, and the results for the sub-systems were then synthesized together to give overall damage probabilities for typical dwellings. The references do not give the detailed damage probability information. Figure 5.3 shows a typical final result in terms of MDR vs. MMI.

Chapter 6.

SELECTION OF DAMAGE PROBABILITIES

With all of the foregoing results and discussion, it is now possible to draw together the various results, to indicate how damage probabilities may be determined for various situations, and to choose the DPM for the pilot study of a limited class of buildings in Boston.

6.1 COMPARISON OF EMPIRICAL, THEORETICAL AND SUBJECTIVE RESULTS

These comparisons have been presented in Figures 6.1 and 6.2. Concrete moment resisting frames have been used for this comparison, since most of the empirical data came from concrete buildings. (In Figure 6.1, the MDR for concrete buildings at MMI VII.5 during the San Fernando earthquake has been used.) For MMI > VI, two theoretical relations are shown, corresponding to the two MMI vs. acceleration relations discussed in subsection 4.1.2. In studying Figures 6.1 and 6.2, it should be kept in mind that the data are most complete and most reliable for the UBC Zone 3 design strategy. Also, it should be kept in mind that the theoretical and subjective results have been specifically tailored to the prototype buildings designed in accordance with Boston practice.

The range of moderate intensities (MMI VII and VIII) includes the best data. Since this is the range where yielding of a structure begins or is small, the theoretical approach should be most reliable. Moreover, the experience which forms the basis of the subjective method is greatest and most complete for this range. Hence one might hope for agreement in the results of the various methods. Possibly there is a measure of agreement at MMI VII, but the agreement can hardly be said to be good.

For low intensities, the theoretical and subjective approaches give MDR which are considerably larger (on a logarithmic scale) than the empirical data. As previously mentioned, the theoretical approach probably overestimates damage because the stiffening effect of non-structural components has been ignored. On the other hand, the empirical MDR may be too small, because low levels of damage tend to go unreported. The subjective results may well best reflect the potential damage to buildings during earthquake ground motions of low intensity.

For high intensities, the theoretical and subjective approaches give MDR which are significantly smaller than the empirical data. (The interpretation given to the ductility factors reported in section 4.3.3 would also imply greater damage at these intensities than that indicated by the theoretical curves in Figures 6.1 and 6.2). This of course is the range where all of the results are least reliable. One possible explanation for the differences may be offered. The prototype buildings were specifically designed to meet the 1970 UBC, and theoretical and subjective estimates reflect the degree of reinforcement specified by that version of the UBC. The empirical data, on the other hand, came almost entirely from buildings built prior to 1970, and hence presumably were not all provided with as much reinforcing steel at joints or with as satisfactory shear ties in columns. However, it is felt that the theoretical and subjective approaches may well have underestimated the level of damage that can occur at these high intensities.

6.2 DAMAGE PROBABILITIES FOR PILOT APPLICATION OF SDDA

Several basic decisions were made in choosing damage probability matrices for the pilot application of Seismic Design Decision Analysis.

The first decision was to utilize only one set of DPM for all buildings covered by the pilot study, and to base this DPM upon the various results for concrete frame buildings. The empirical data were most complete for concrete structures, and differences among the structural types appear to be less than the uncertainty in the information.

The second decision was that the damage probabilities should be best estimates at the lower intensities and conservative (high) estimates at the larger intensities. Thus the damage probabilities are best estimates where only light or moderate damage is involved, but are conservative where collapse is possible.

The first step in choosing damage probabilities is to determine mean damage ratios. The selected MDR are plotted in Figure 6.3 and listed in Table 6.1. These MDR were selected as follows:

1. For MMI V and VI, MDR between the empirical data and the subjective estimates were used.

2. At MMI VII, the empirical data points for UBC 0 were used. The subjective and theoretical results were used to establish ratios between the MDR for UBC 0 and the MDR for the other design strategies. It must be kept in mind that buildings designed for UBC 3 by "Boston practice" will experience more non-structural damage than buildings built recently in California.

3. At MMI VII.5, the sum total of empirical, theoretical and subjective results were used to choose the MDR. The same was done at MMI VIII for UBC 2 and 3.

4. MMI at which MDR = 100% were selected on the basis of the empirical MDR, the computed ductility ratios, and general experience.

5. Straight lines on the logarithmic plot were drawn from the MDR in step 3 to the MMI in step 4.

The resulting curves on the logarithmic plot have an irregular appearance, but there is nothing in either theory or the data to indicate that these curves should have a regular shape.

The damage probabilities in Table 6.2 were then selected by a process that involved scanning the empirical and subjective damage probability matrices and using Equation 2.2 to insure that the damage probabilities are consistent with the mean damage ratios. This procedure is described in Appendix D. Many of the individual damage probabilities were selected somewhat arbitrarily, and somewhat different sets of probability values would be equally well justified. The most arbitrary part of the selection process was the breakdown between P_T and P_C; in any study where these damage states are important, a sensitivity study should be made of this breakdown.

Table 3.1

DAMAGE PROBABILITIES (%) AND MEAN DAMAGE RATIOS (%) FOR INTENSITY VII ZONE OF SAN FERNANDO EARTHQUAKE

Date Const.	Pre-1933					Post-1947						
No. Stories	5-7		8-13		All	5-7		8-13		14-18	19+	All
Type of Const.	Co	St	Co	St		Co	St	Co	St	St	St	
Damage State												
0	16	18	16	6	14	21	24	27	44	43	21	33
1	16	9	12	13	12	26	28	33	31	43	54	34
2	26	46	28	53	35	16	38	32	6	0	25	20
3	21	27	14	16	18	26	5	8	16	14	0	10
4	11	0	21	0	11	11	5	0	3	0	0	3
5	0	0	7	9	6	0	0	0	0	0	0	0
6	10	0	2	3	4	0	0	0	0	0	0	0
7,8	0	0	0	0	0	0	0	0	0	0	0	0
MDR - %	4.4	1.1	2.7	2.5	2.8	1.1	0.7	0.4	0.5	0.4	0.2	0.5
No. Bldgs	19	11	43	32	114	19	21	37	32	14	24	156

Table 3.2

DAMAGE PROBABILITIES FOR LOW BUILDINGS IN COMPTON DURING LONG BEACH EARTHQUAKE

Damage Rate %	Commercial Masonry Walled Buildings	Residential Masonry Walled Buildings	Wood Frame Dwellings
0-4%	2%	47%	94.7%
5-24%	4%	16%	2.9%
25-29%	21%	27%	1.4%
50-75%	20%	10%	0.8%
100%	53%	0%	0.2%
MDR%	72%	16%	2.3%

Data from Earthquake Investigations in Western United States, 1931-64, in chapter by R.R. Martel, as quoted in Steinbrugge (1970).

Table 3.3

DAMAGE PROBABILITIES FOR WOODEN FRAME DWELLINGS

IN EPICENTRAL REGION OF 1971 SAN FERNANDO EARTHQUAKE

Damage Ratio - %	Damage Probability - %
0–2	42
2–5	33
5–10	12
10–20	6
20–30	4
30–40	2
40–50	1
MDR	5.5

Note: From Figure 24 of Steinbrugge et al (1971), using the curve for "all dwellings." The MDR of 5.5 was computed by the writer from the damage probabilities. The MDR as given in the report was 6.6%. The damage ratio is based upon market value before the earthquake.

Table 4.1

CORRELATIONS BETWEEN GROUND MOTION AND

MODIFIED MERCALLI INTENSITY

Modified Mercalli Intensity	Nominal Peak Acceleration - g	
	Gutenberg-Richter	Alternate
IV	0.007g	
V	0.015g	
VI	0.03g	0.03g
VII	0.07g	0.09g
VIII	0.15g	0.20g
IX	0.27g	0.50g

Table 4.2

COMPUTED FUNDAMENTAL PERIODS OF PILOT AND PROTOTYPE BUILDINGS

Building	Computed Fundamental Period - sec			
	UBC 0	UBC 2	UBC 3	S
Pilot - X dir	5.27	5.27	4.20	3.08
- Y dir	4.50	4.50	3.31	2.95
CMRF - 6	2.81	2.81	2.07	1.38
SMRF - 11	3.53	3.12	2.40	1.71
CMRF - 11	2.66	2.66	2.05	1.47
CSW - 11	1.84	1.84	1.37	1.11
CSW - 17	3.34	2.68	2.35	2.11

Table 4.3

MAXIMUM DUCTILITY FACTORS CAUSED BY GROUND MOTION WITH 0.27g PEAK ACCELERATION

Building	Maximum Ductility Factor			
	UBC 0	UBC 2	UBC 3	S
Pilot	5.2	5.2	3.5	3.2
CMRF - 6	4.6	4.6	3.8	2.8
SMRF - 11	13.5	6.0	6.1	5.5
CMRF - 11	3.5	3.5	2.7	1.2
CSW - 11	$\boxed{8.6}$	$\boxed{6.5}$	2.2	1.7
CSW - 17	3.3	3.1	2.0	1.0

$\boxed{}$ = probable partial or total collapse

(dashed box) = possible partial or total collapse

Table 6.1

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

Design	Modified Mercalli Intensity					
Strategy	VI	VII	VII.5	VIII	IX	X
0, 1	0.22	3.0	16	52	100	100
2	0.16	1.9	7	18	100	100
3	0.13	1.4	4	10	45	100
4	0.10	1.2	2.5	5	21	100

Table 6.2

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

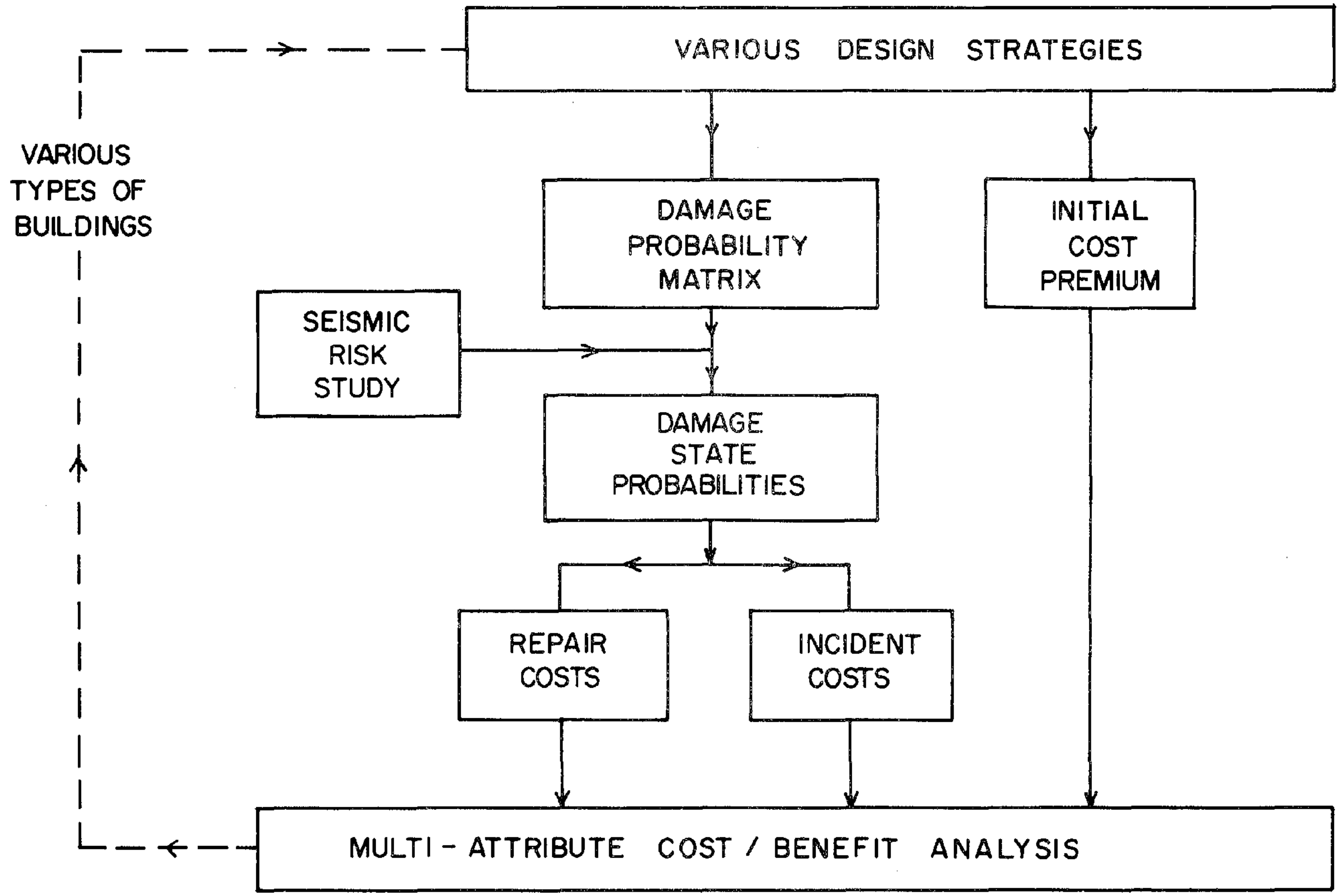


FIGURE 1.1 FLOW DIAGRAM FOR GENERAL METHODOLOGY

DAMAGE STATE	STRUCTURAL DAMAGE	NON-STRUCTURAL DAMAGE	DAMAGE RATIO (%)	INTENSITY OF EARTHQUAKE				
				V	VI	VII	VIII	IX
0	NONE	NONE	0-0.05	95	79	33	6	0
1	NONE	MINOR	0.05-0.3	5	18	34	19	2
2	NONE	LOCALIZED	0.3-1.25	0	3	20	44	18
3	NOT NOTICEABLE	WIDESPREAD	1.25-3.5	0	0	10	13	30
4	MINOR	SUBSTANTIAL	3.5-7.5	0	0	3	6	20
5	SUBSTANTIAL	EXTENSIVE	7.5.20	0	0	0	12	10
6	MAJOR	NEARLY TOTAL	20-65	0	0	0	0	7
7	BUILDING CONDEMNED		100	0	0	0	0	8
8	COLLAPSE		100	0	0	0	0	5

FIGURE 2.1 GENERAL FORM OF DAMAGE PROBABILITY MATRIX

	Description of Level of Damage	Damage Ratio*	
		Central Value	Range
0	No Damage	0	0 - 0.05
1	Minor non-structural damage--a few walls and partitions cracked, incidental mechanical and electrical damage	0.1	0.05 - 0.3
2	Localized non-structural damage--more extensive cracking (but still not widespread); possibly damage to elevators and/or other mechanical/electrical components	0.5	0.3 - 1.25
3	Widespread non-structural damage--possibly a few beams and columns cracked, although not noticeable	2	1.25 - 3.5
4	Minor structural damage--obvious cracking or yielding in a few structural members; substantial non-structural damage with widespread cracking	5	3.5 - 7.5
5	Substantial structural damage requiring repair or replacement of some structural members; associated extensive non-structural damage	10	7.5 - 20
6	Major structural damage requiring repair or replacement of many structural members; associated non-structural damage requiring repairs to major portion of interior; building vacated during repairs	30	20 - 65
7	Building condemned	100	20 - 65
8	Collapse	100	65 - 100

*Ratio of cost of repair to replacement cost.

FIGURE 2.2. EARTHQUAKE DAMAGE STATES

EXTENDED (ORIGINAL) DAMAGE STATES	SHORTENED DAMAGE STATES		
	LEVEL OF DAMAGE	SYMBOL	CENTRAL DAMAGE RATIO-%
0	NONE	O	0
1 2	LIGHT	L	0.3
3 4 5	MODERATE	M	5
6	HEAVY	H	30
7	TOTAL	T	100
8	COLLAPSE	C	100

FIGURE 2.3 RELATION BETWEEN EXTENDED AND SHORTENED DAMAGE STATES

	MMI	VI									VII									VII-VIII		
	AGE	PRE-1933			POST-1947			ALL			PRE-1933			POST-1947			ALL			POST-1947		
	STRUC TYPE	ALL	C	S	ALL	C	S	ALL	C	S	ALL	C	S	ALL	C	S	ALL	C	S	ALL	C	S
5-7	DAMAGE STATE* 0	90	100	80	86	86	86	88	90	83	18	16	18	24	21	24	22	17	24	0	0	0
	1	10	0	20	14	14	14	12	10	17	12	16	9	27	26	28	20	20	21	25	0	37.5
	2	0	0	0	0	0	0	0	0	0	30	26	46	27	16	38	29	23	39	50	25	62.5
	3	0	0	0	0	0	0	0	0	0	24	21	27	15	26	5	20	25	12	17	50	0
	4	0	0	0	0	0	0	0	0	0	9	11	0	7	11	5	8	10	4	8	25	0
	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	6	0	0	0	0	0	0	0	0	0	6	10	0	0	0	0	3	5	0	0	0	0
	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	M.D.R.	0.03	0	0.06	0.03	0.02	0.03	0.03	0.02	0.04	3.20	4.44	1.05	0.82	1.05	0.66	1.83	2.68	0.77	1.17	2.67	0.42
	ST DEVIATION	0.08	0	0.11	0.06	0.05	0.06	0.07	0.05	0.08	7.60	9.71	0.92	1.29	1.27	1.30	5.20	6.96	1.19	1.962	.193	0.23
	NO OF BLDGS	10	3	5	14	7	7	24	10	12	33	19	11	41	19	21	77	40	33	12	4	8
8-13	DAMAGE STATE* 0	89	100	75	75	46	94	76	63	86	13	16	6	36	27	44	23	21	25	25	0	50
	1	11	0	25	18	36	6	16	25	9	12	12	13	31	33	31	21	22	22	0	0	0
	2	0	0	0	7	18	0	8	12	5	37	28	53	20	32	6	29	30	29	25	0	50
	3	0	0	0	0	0	0	0	0	0	17	14	16	12	8	16	15	11	17	0	0	0
	4	0	0	0	0	0	0	0	0	0	11	21	0	1	0	3	7	11	1	0	0	0
	5	0	0	0	0	0	0	0	0	0	8	7	9	0	0	0	4	4	5	40	100	0
	6	0	0	0	0	0	0	0	0	0	3	2	3	0	0	0	1	1	1	0	0	0
	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	M.D.R.	0.03	0.01	0.07	0.07	0.15	0101	0.08	0.11	0.05	2.56	2.68	2.51	0.47	0.43	0.52	1.50	1.62	1.51	4.94	9.63	0.25
	ST DEVIATION	0.08	0.01	0.11	0.15	0.21	0.04	0.17	0.19	0.15	4.62	4.14	5.35	0.81	0.65	0.96	3.53	3.25	3.94	4.76	1.15	0.25
	NO OF BLDGS	9	5	4	28	11	17	38	16	22	78	43	32	70	37	32	150	81	65	4	2	2
14-18	DAMAGE STATE* 0	0	0	0	75	67	83	75	67	83	0	0	0	53	80	43	48	80	38	0	0	0
	1	0	0	0	25	33	17	25	33	17	50	0	50	37	20	43	38	20	44	0	0	0
	2	0	0	0	0	0	0	0	0	0	50	0	50	0	0	0	5	0	6	0	0	0
	3	0	0	0	0	0	0	0	0	0	0	0	0	10	0	14	9	0	12	0	0	0
	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	M.D.R.	0	0	0	0.06	0.08	0.05	0.06	0.08	0.05	0.45		0.45	0.33	0.06	0.43	0.34	0.06	0.43			
	ST DEVIATION	0	0	0	0.11	0.11	0.10	0.11	0.11	0.10	0.16		0.16	0.84	0.09	0.96	0.80	0.90	0.90			
	NO OF BLDGS	0	0	0	12	6	6	12	6	6	2	2	19	5	14	21	5	16				
19+	DAMAGE STATE* 0	0	0	0	100	100	100	100	100	100	0	0	0	27	100	21	26	100	20	0	0	0
	1	0	0	0	0	0	0	0	0	0	0	0	0	50	0	54	48	0	52	0	0	0
	2	0	0	0	0	0	0	0	0	0	0	0	0	23	0	25	22	0	24	0	0	0
	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	5	0	0	0	0	0	0	0	0	0	100	0	100	0	0	0	4	0	4	0	0	0
	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	M.D.R				0.02	0.04	0.01	0.02	0.04	0.01	8.67		8.67	0.22	0.01	0.24	0.53	0.01	0.57			
	ST DEVIATION				0.02	0	0.01	0.02	0	0.01	0		0	0.26	0.01	0.26	1.62	0.01	1.67			
	NO OF BLDGS				3	1	2	3	1	2	1		1	26	2	24	27	2	25			
All	DAMAGE STATE* 0	90	100	78	79	64	91	80	73	86	14	16	9	33	32	33	25	23	25	6	0	10
	1	10	0	22	18	28	9	16	21	12	12	13	13	34	29	39	25	21	30	19	0	30
	2	0	0	0	3	8	0	4	6	2	35	27	50	20	24	18	26	26	28	44	17	60
	3	0	0	0	0	0	0	0	0	0	18	16	17	10	13	9	14	15	12	13	33	0
	4	0	0	0	0	0	0	0	0	0	11	18	0	3	3	2	6	10	1	6	17	0
	5	0	0	0	0	0	0	0	0	0	6	5	9	0	0	0	3	2	3	12	33	0
	6	0	0	0	0	0	0	0	0	0	4	5	2	0	0	0	1	2	1	0	0	0
	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	M.D.R.	0.03	0	0.06	0.05	0.09	0.02	0.06	0.07	0.05	2.75	3.22	2.20	0.5	0.5	0.46	1.44	1.87	1.04	2.74	5.0	0.39
	ST DEVIATION	0.08	0.01	0.11	0.12	0.16	0.06	0.14	0.15	0.13	5.64	6.44	4.64	0.93	0.92	0.95	3.86	4.72	2.90	4.17	9.37	0.24
	NO OF BLDGS	19	8	9	57	25	32	77	33	42	114	62	46	156	63	91	275	128	139	16	6	10

FIGURE 3.1. SUMMARY OF DAMAGE MATRICES, REPLACEMENT COST VERSION

* Data for damage states expressed in percentage

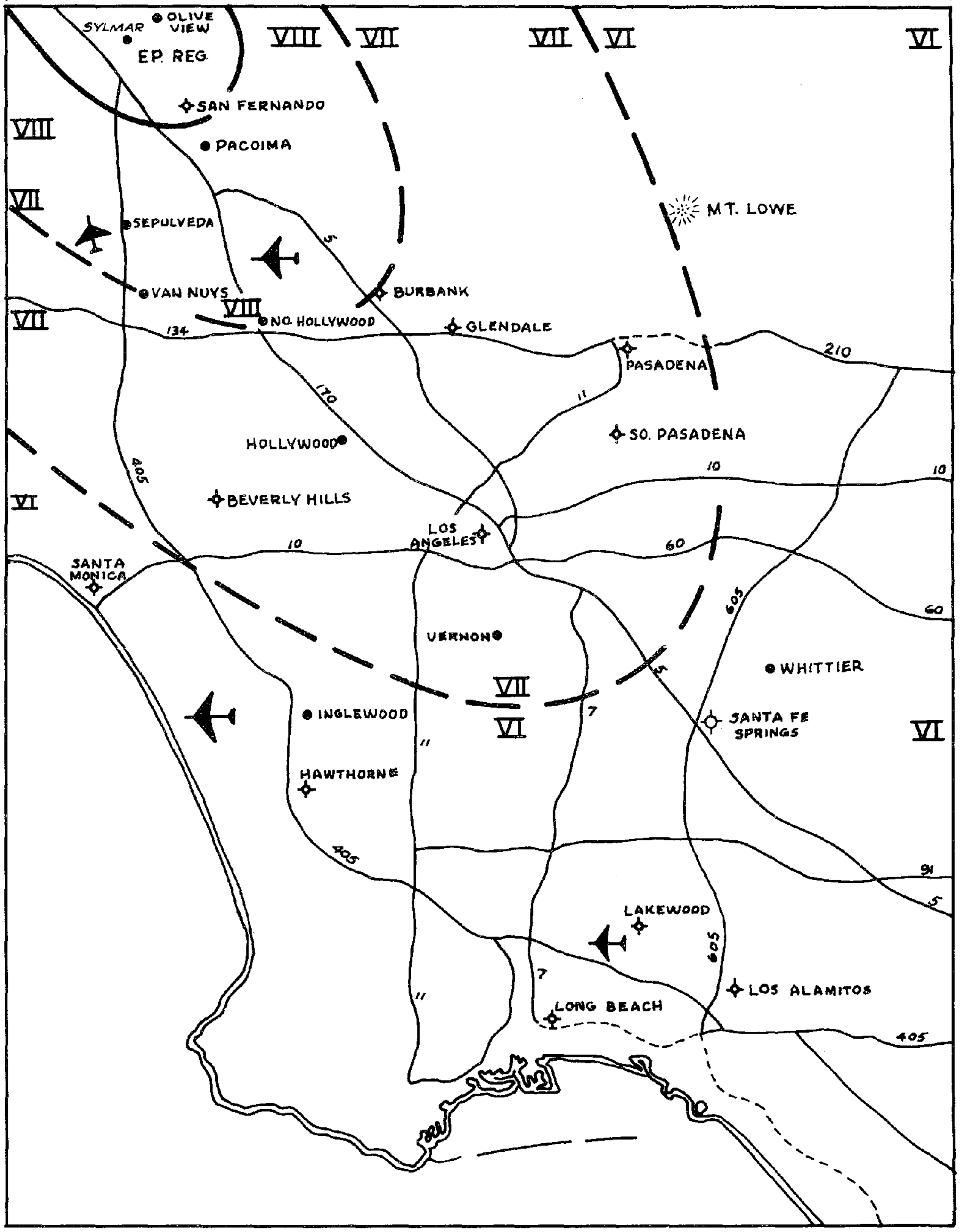


FIGURE 3.2 GEOGRAPHICAL AREA OF STUDY WITH ZONES OF MODIFIED MERCALLI INTENSITY

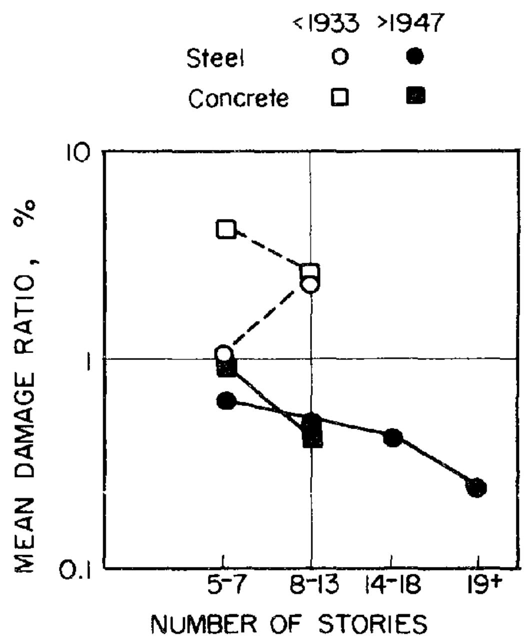


FIGURE 3.3 VARIATION OF DAMAGE WITH BUILDING HEIGHT FOR INTENSITY VII FROM SAN FERNANDO EARTHQUAKE

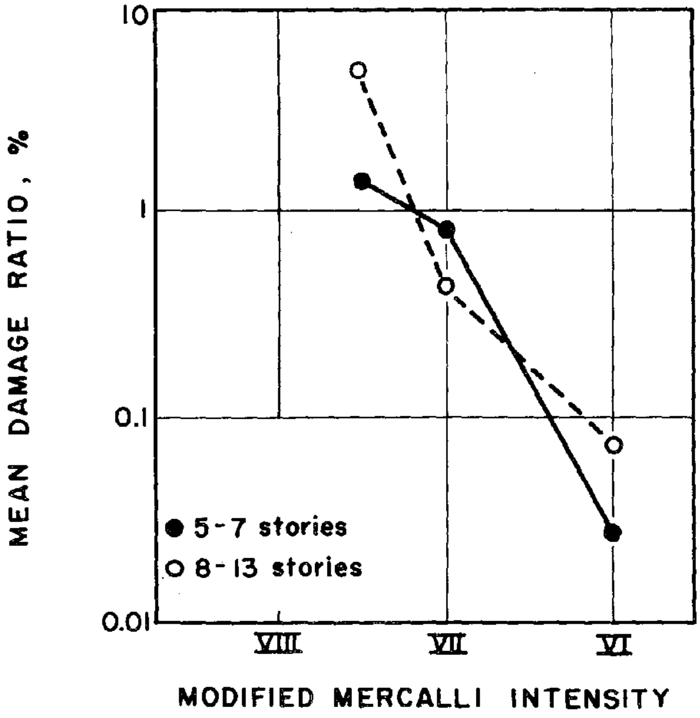


FIGURE 3.4 VARIATION OF DAMAGE WITH INTENSITY FOR POST-1947 BUILDINGS

CARACAS 1967

ZONE 2 DESIGN

DS	VI	VII	VIII	IX	X
0		51	66		
1					
2		42	20		
3					
4					
5		7	9		
6					
7		0	2		
8		0	3		
n_I		1090	127		

JAPAN 1968

ZONE S* DESIGN

DS	VI	VII	VIII	IX	X
0		100		20	
1				40	
2				20	
3				20	
4				0	
5				0	
6				0	
7				0	
8				0	
n_I		100+		5	

ANCHORAGE 1964

ZONE 2 DESIGN

DS	VI	VII	VIII	IX	X
0				0	
1				0	
2				0	
3				0	
4				0	
5				0	
6				50	
7				0	
8				50	
n_I				4	

ZONE 3 DESIGN

DS	VI	VII	VIII	IX	X
0				0	
1				0	
2				20	
3				10	
4				10	
5				10	
6				20	
7				10	
8				20	
n_I				10	

SAN FRANCISCO 1957

ZONE 2 DESIGN

DS	VI	VII	VIII	IX	X
0	98				
1	1.6				
2	0.3				
3	0				
4	0				
5	0				
6	0				
7	0				
8	0				
n_I	1124				

SEATTLE 1965

ZONE 0 DESIGN

DS	VI	VII	VIII	IX	X
0		70			
1		14			
2					
3		9			
4					
5					
6		7			
7					
8		0			
n_I		286			

ZONE 2 DESIGN

DS	VI	VII	VIII	IX	X
0		88			
1		8			
2					
3		2			
4					
5					
6		2			
7					
8		0			
n_I		57			

SAN FERNANDO 1971

ZONE 0 DESIGN

DS	VI	VII	VIII	IX	X
0	90	14			
1	10	12			
2	0	35			
3	0	18			
4	0	11			
5	0	6			
6	0	4			
7	0	0			
8	0	0			
n_I	19	114			

ZONE 3 DESIGN

DS	VI	VII	VII.5	IX	X
0	79	33	6	0	
1	18	34	19	0	
2	3	20	44	0	
3	0	10	13	0	
4	0	3	6	0	
5	0	0	12	0	
6	0	0	0	25	
7	0	0	0	75	
8	0	0	0	0	
n_I	57	156	16	4	

ZONE 0 DESIGN | ZONE 1 DESIGN | ZONE 2 DESIGN | ZONE 3 DESIGN | ZONE S* DESIGN

FIGURE 3.5 DAMAGE PROBABILITY MATRICES FROM HISTORICAL EARTHQUAKES

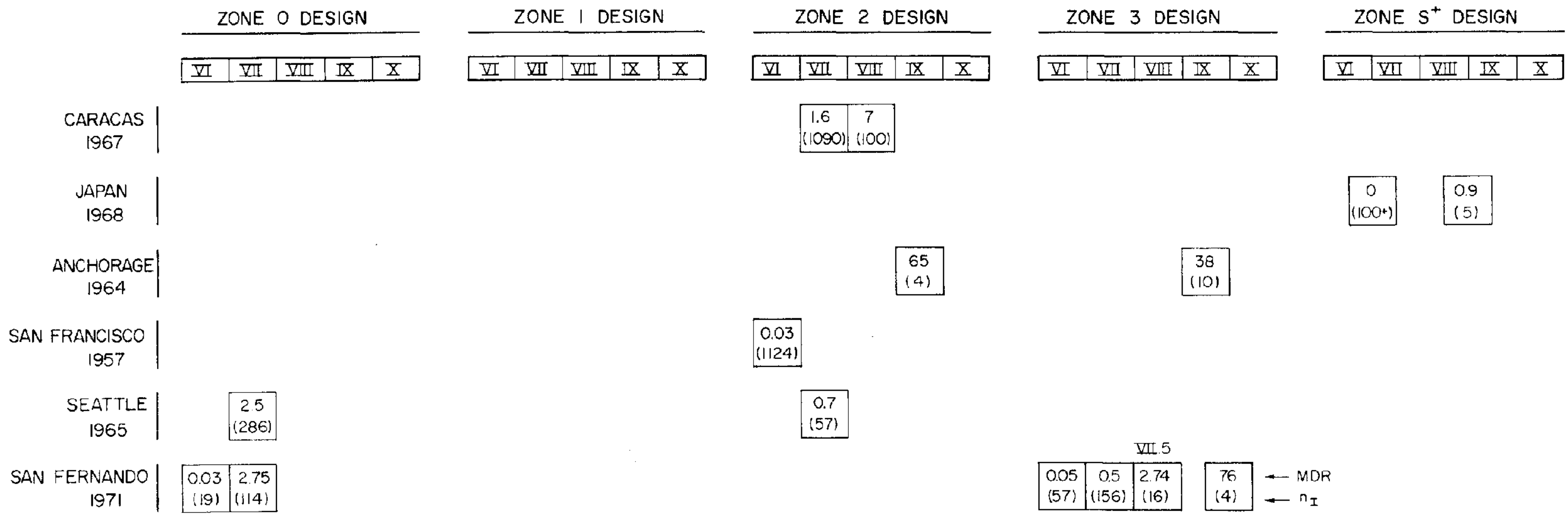


FIGURE 3.6 MEAN DAMAGE RATIOS FROM HISTORICAL EARTHQUAKES

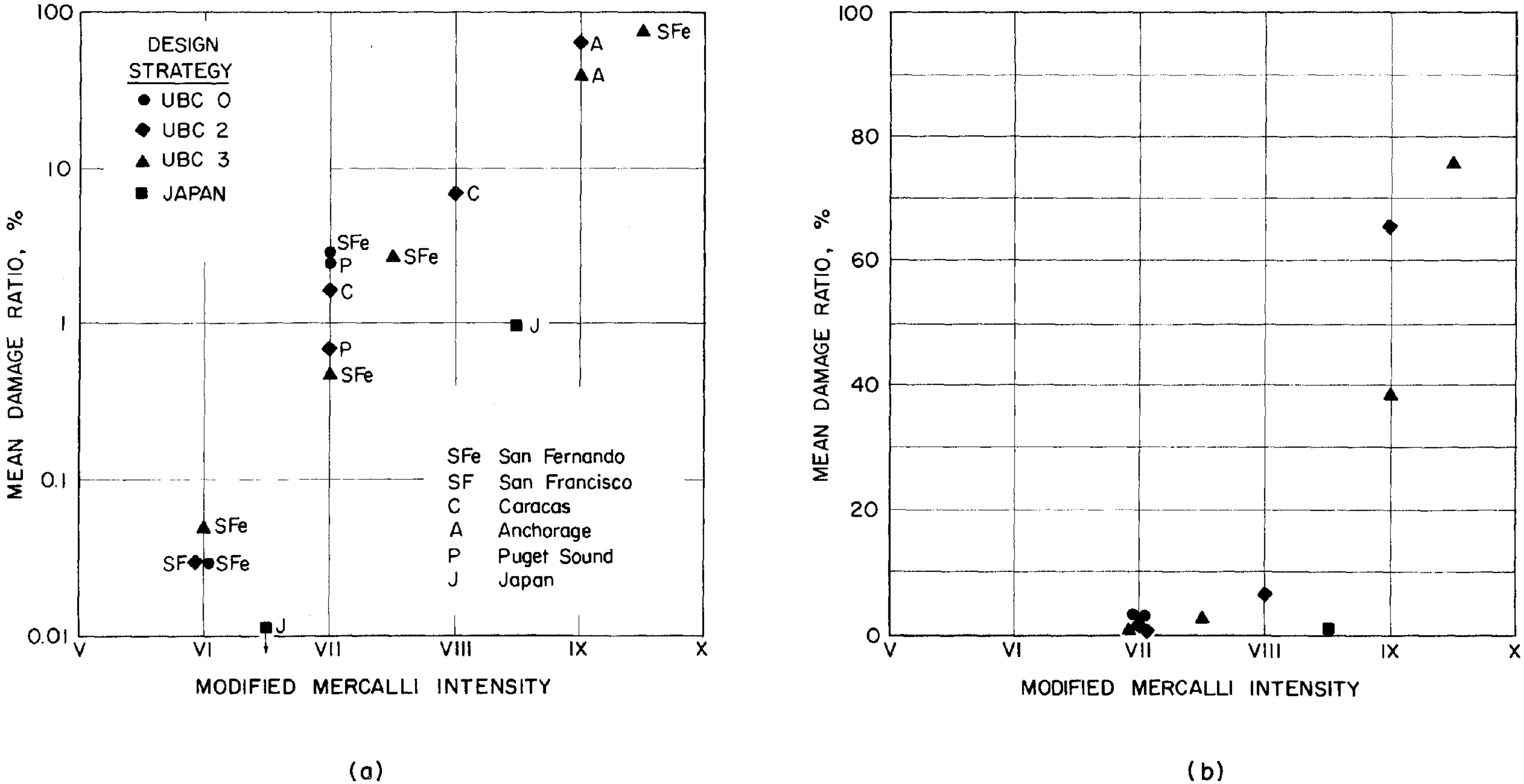


FIGURE 3.7 MEAN DAMAGE RATIOS FROM HISTORICAL EARTHQUAKES

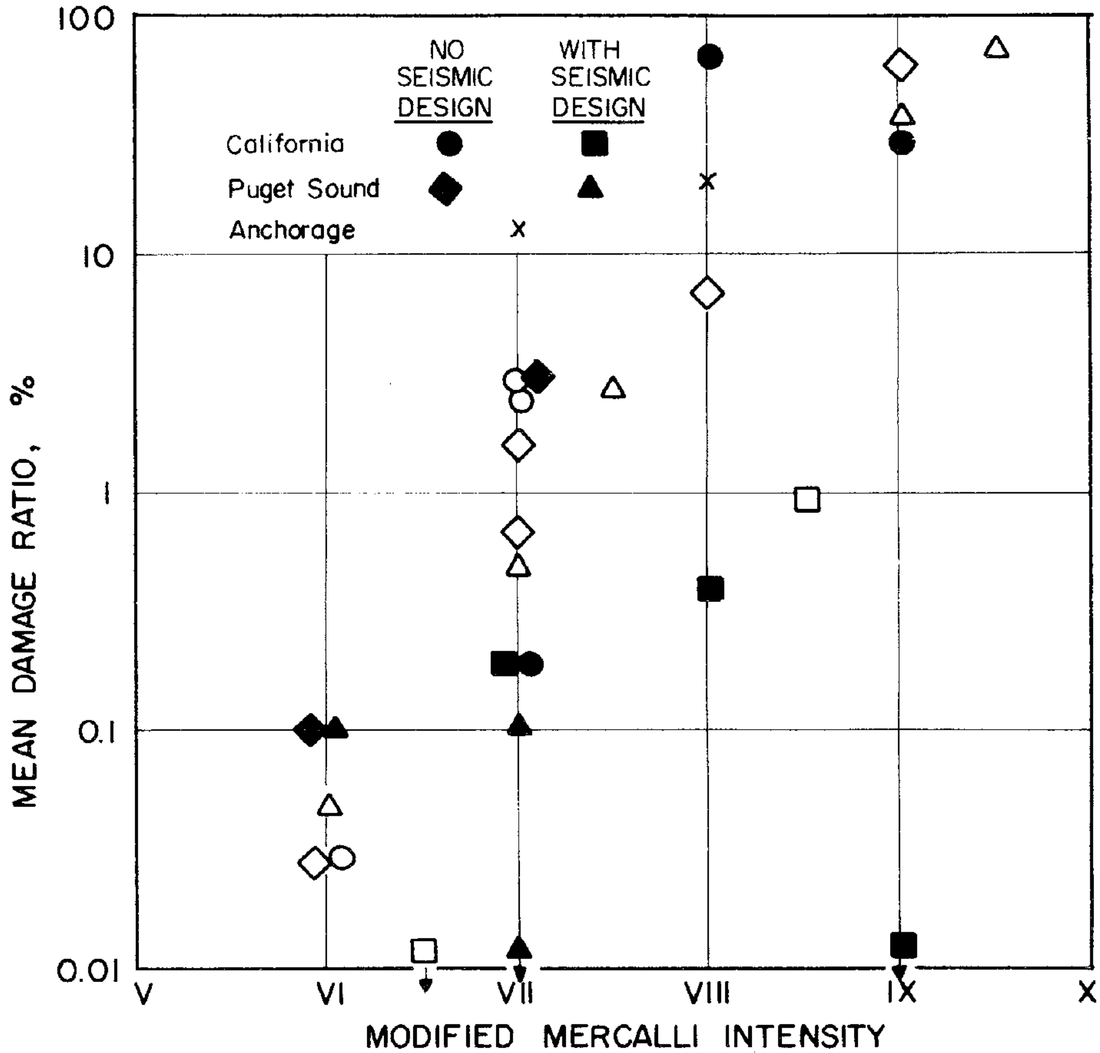


FIGURE 3.8 MEAN DAMAGE RATIOS FOR SCHOOL BUILDINGS
(From Crumlish and Wirth, 1967)

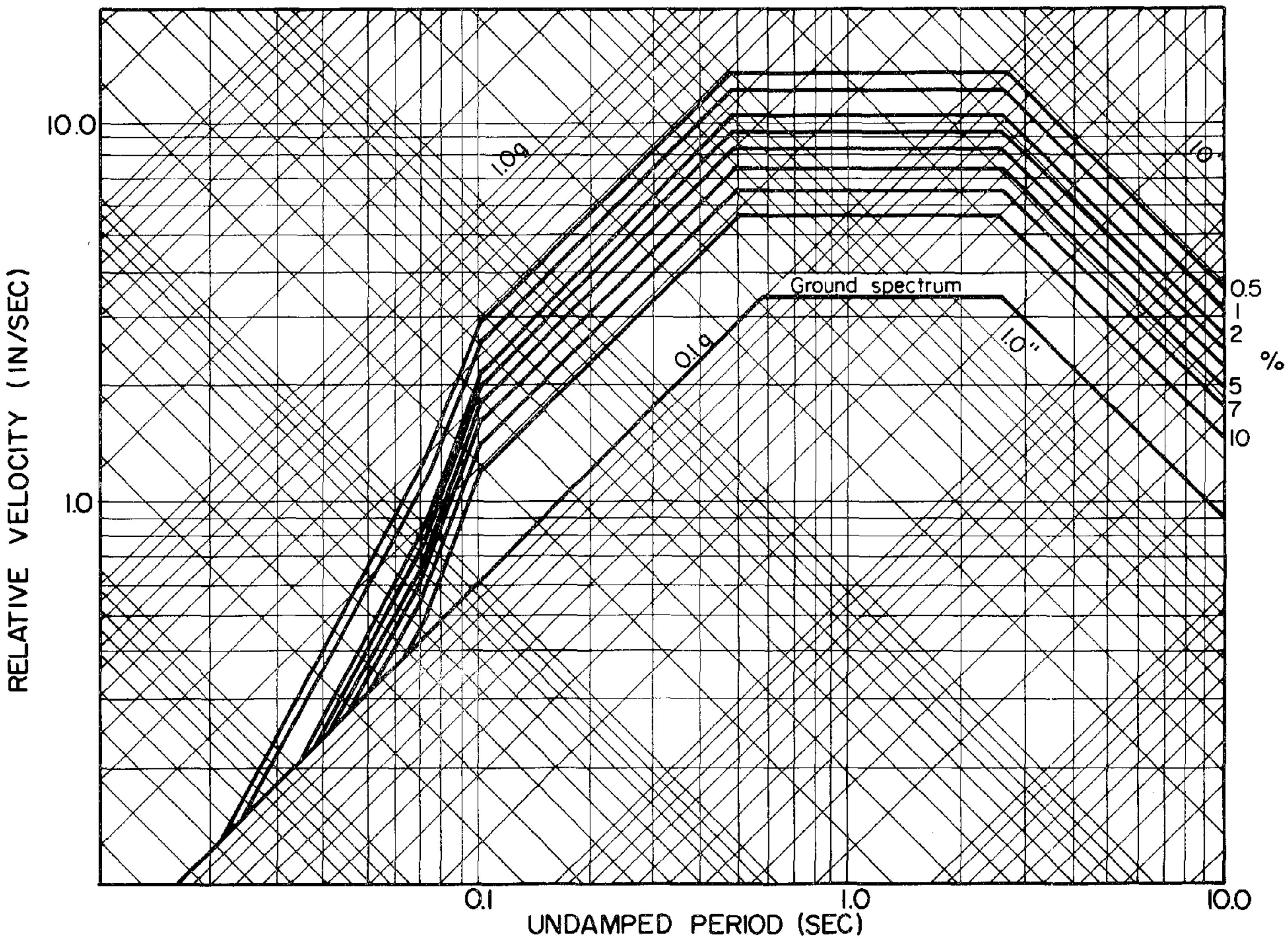


FIGURE 4.1 EXPECTED RESPONSE SPECTRA (Normalized to 0.1 g)

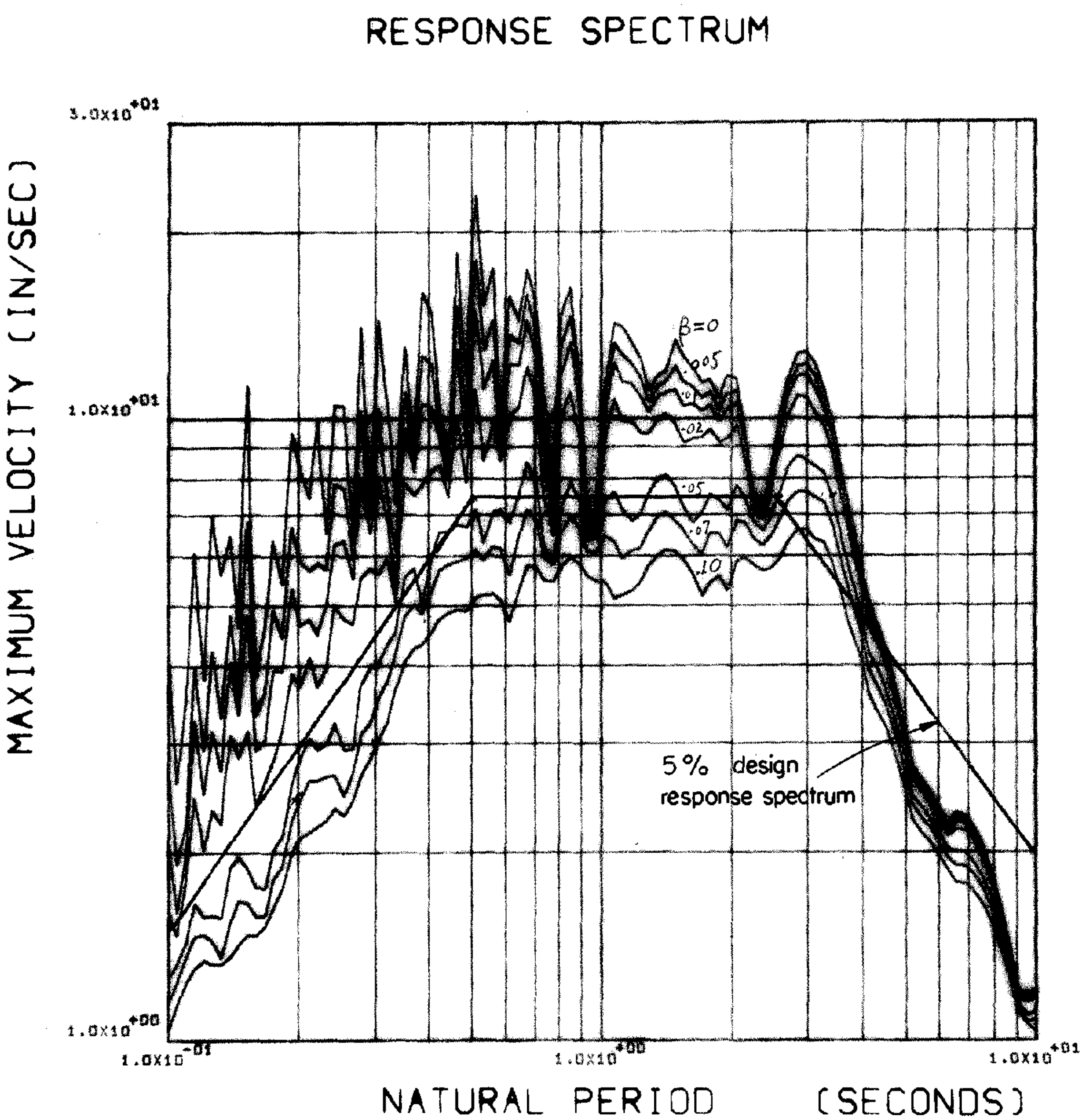
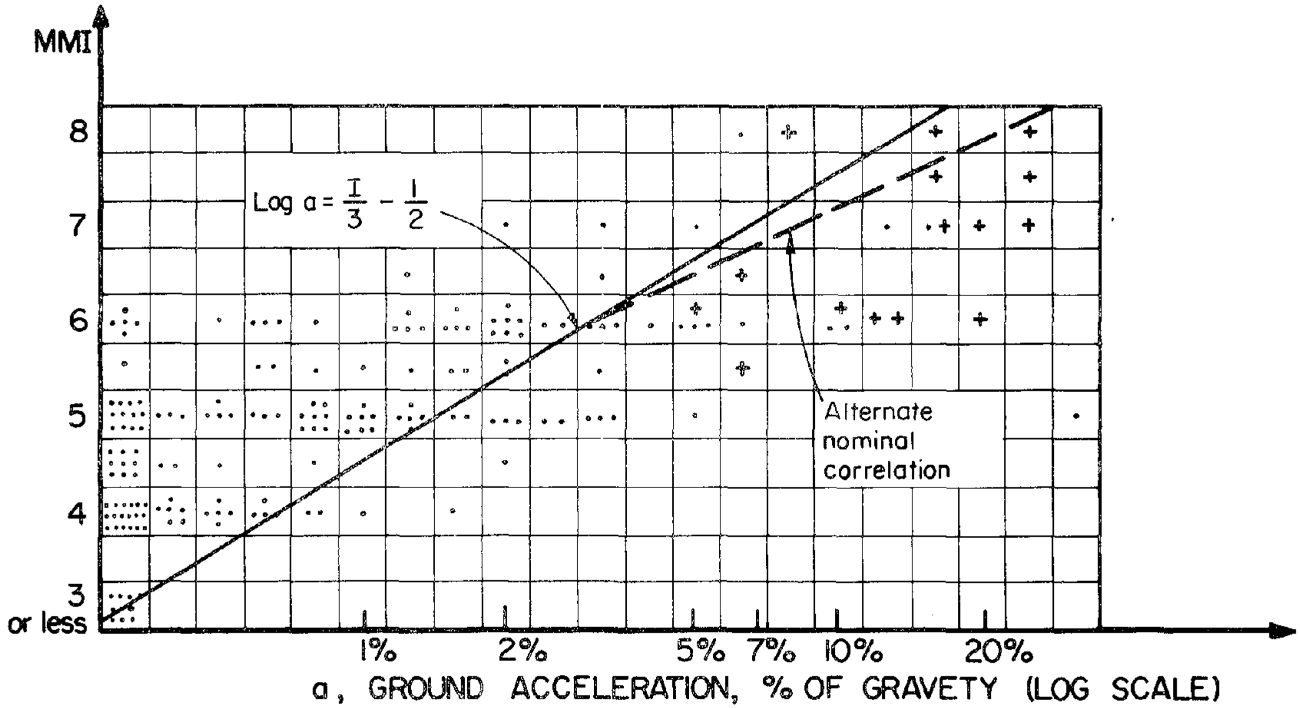


FIGURE 4.2 RESPONSE SPECTRA OF THE ARTIFICIAL EARTHQUAKE (Normalized to 0.1 g)



From (+) Arturo Arias "A measure of earthquake intensity" in Seismic Design for Nuclear power plants, M.I.T. press, 1969. Ed. R. J. Hansen

(·) Gutenberg, B. and Richter, C. "Earthquake magnitude, intensity, energy and acceleration", BSSA, vol. 46 pp. 105 - 143 (1956)

FIGURE 4.3 ACCELERATION VS. INTENSITY

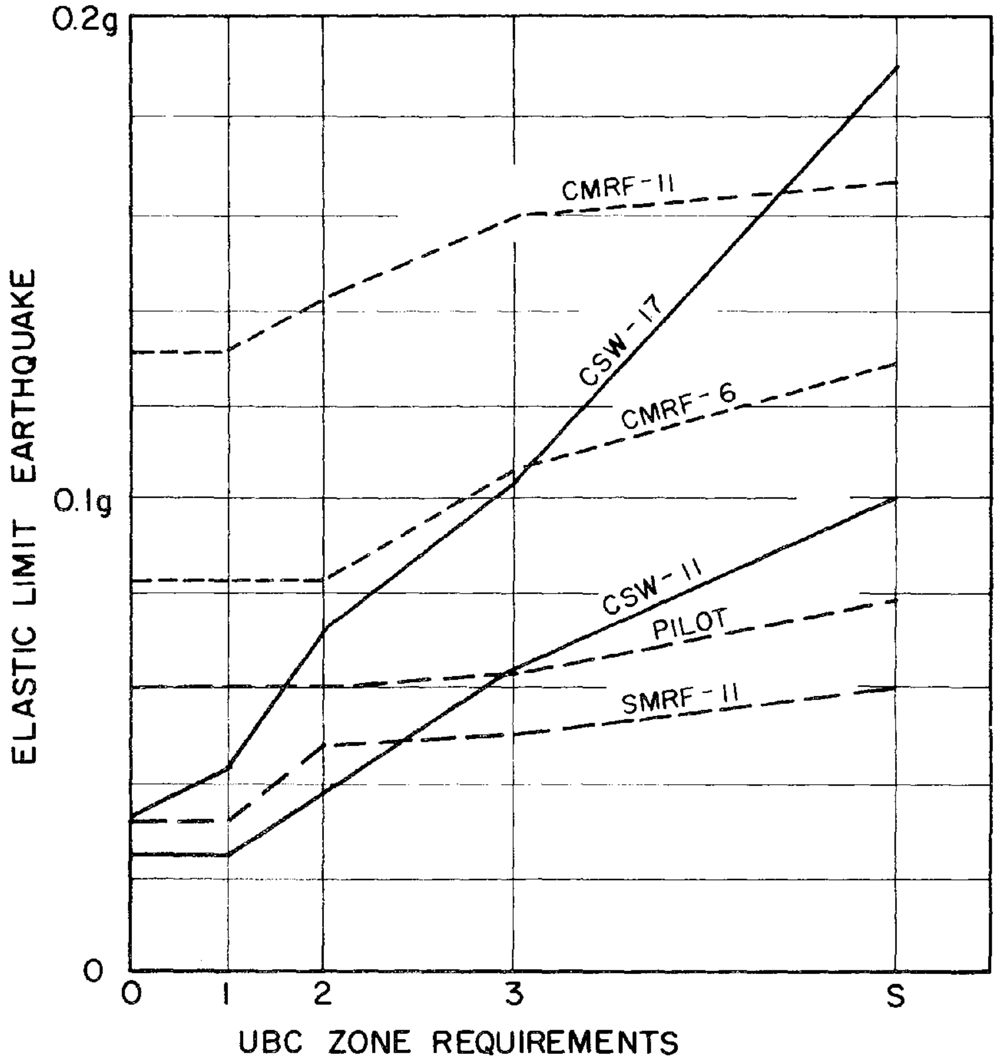


FIGURE 4.4 EFFECT OF DESIGN STRATEGY UPON INTENSITY OF EARTHQUAKE FIRST CAUSING YIELD

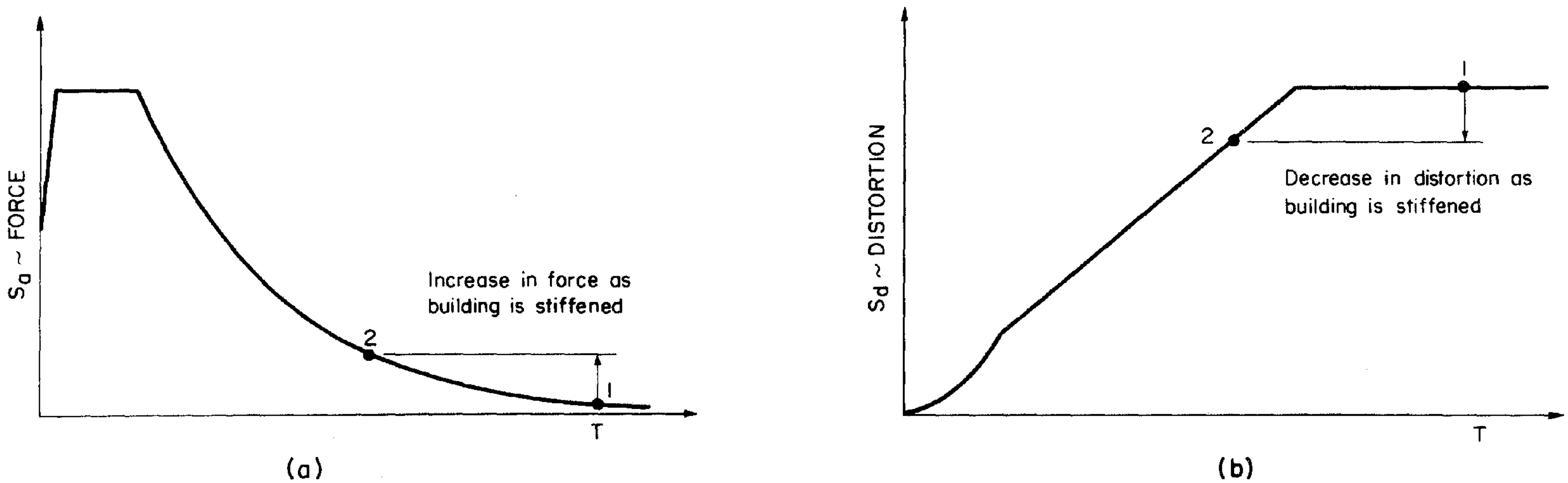


FIGURE 4.5 EFFECT OF STIFFENING UPON FORCES AND DISTORTIONS CAUSED BY EARTHQUAKE

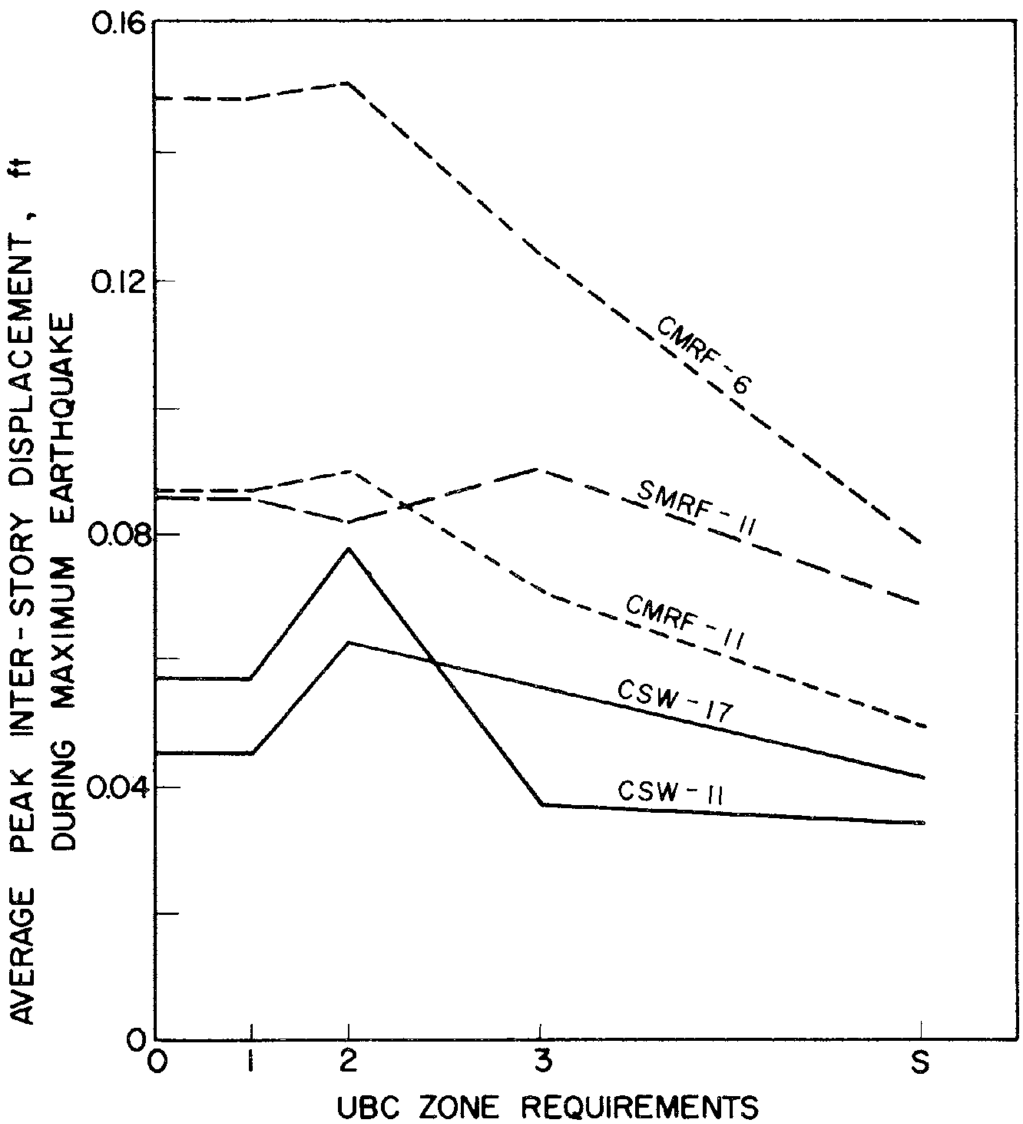


FIGURE 4.6 EFFECT OF DESIGN STRATEGY UPON AVERAGE INTERSTORY DISPLACEMENT DURING 0.27g EARTHQUAKE

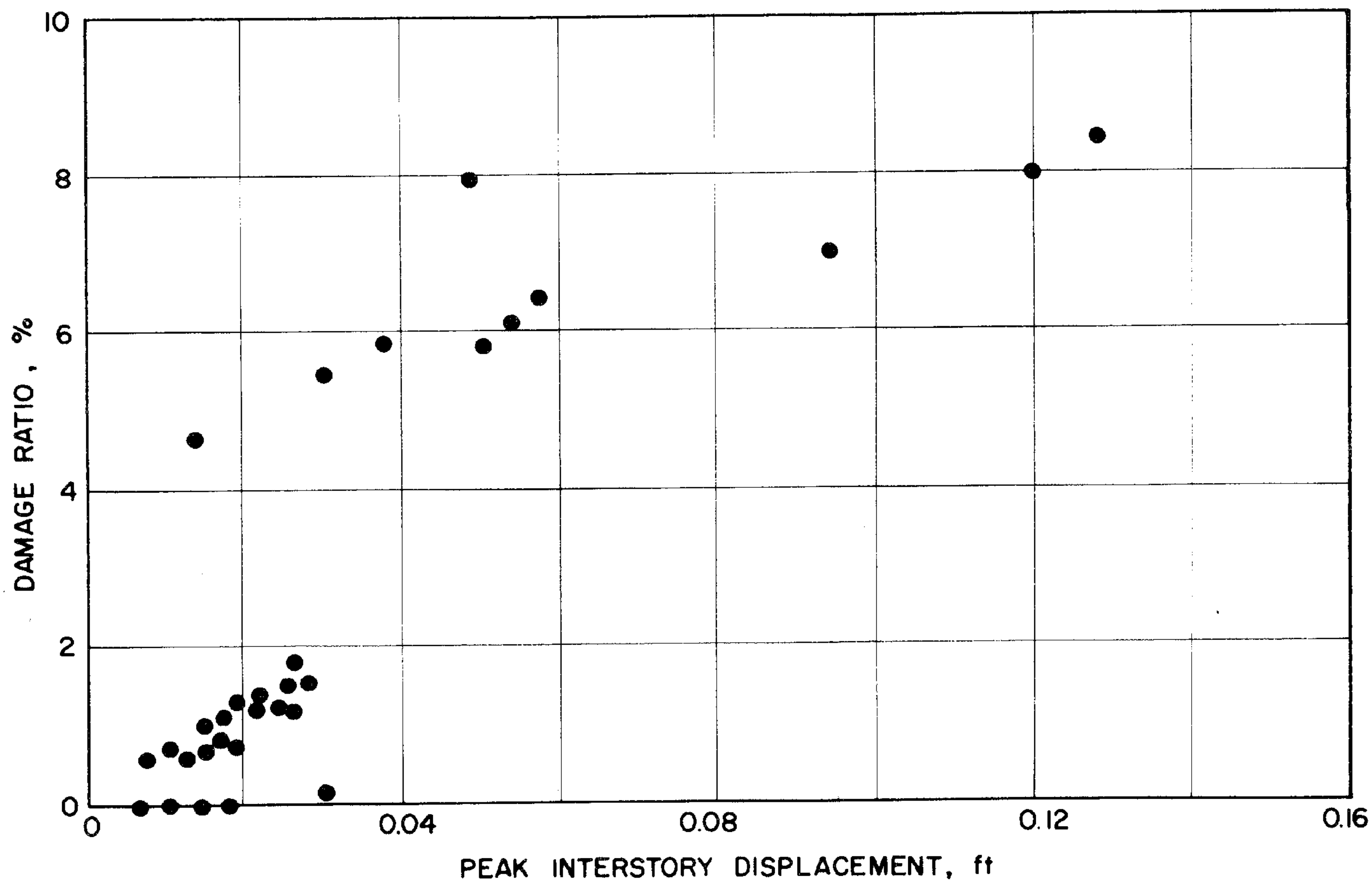


FIGURE 4.7 CORRELATION BETWEEN DAMAGE AND INTERSTORY DISPLACEMENT; DATA FROM SAN FERNANDO EARTHQUAKE

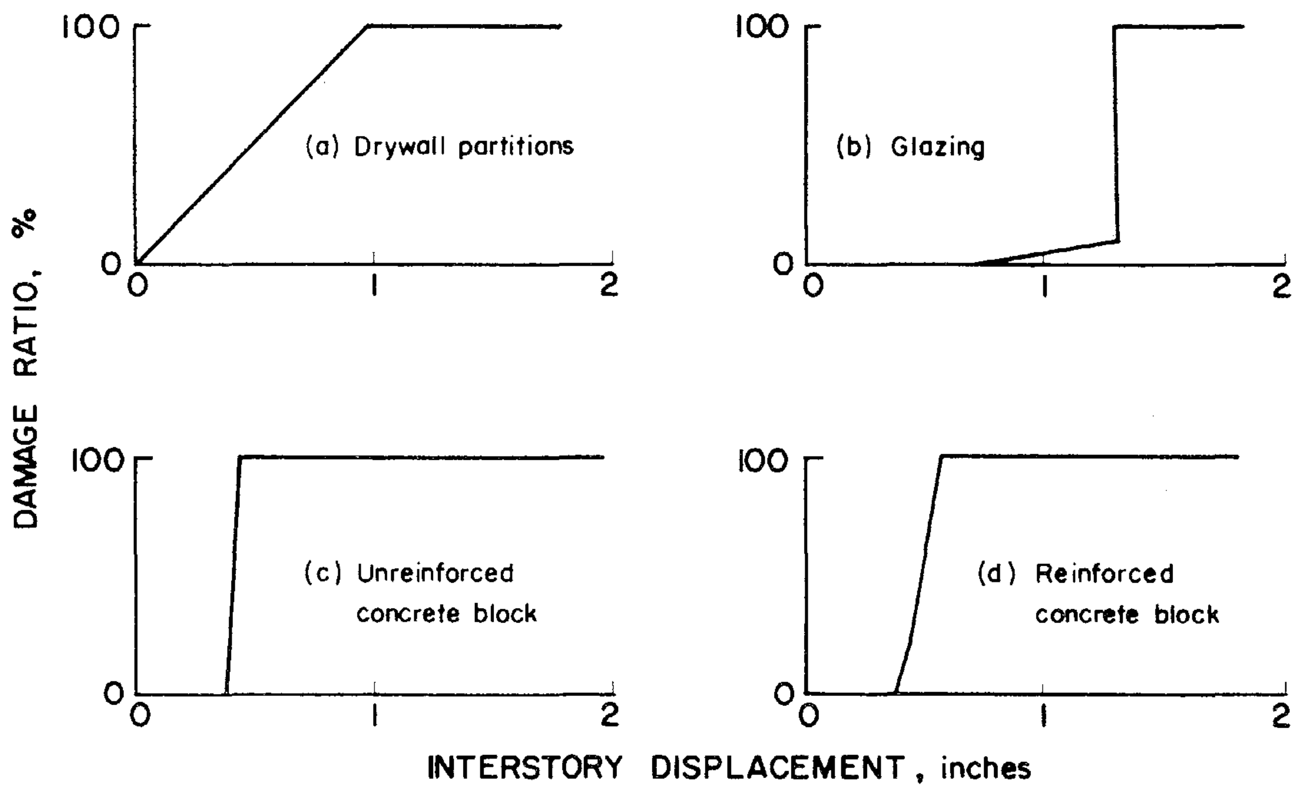


FIGURE 4.8 PRELIMINARY DAMAGE MODELS FOR NON-STRUCTURAL COMPONENTS

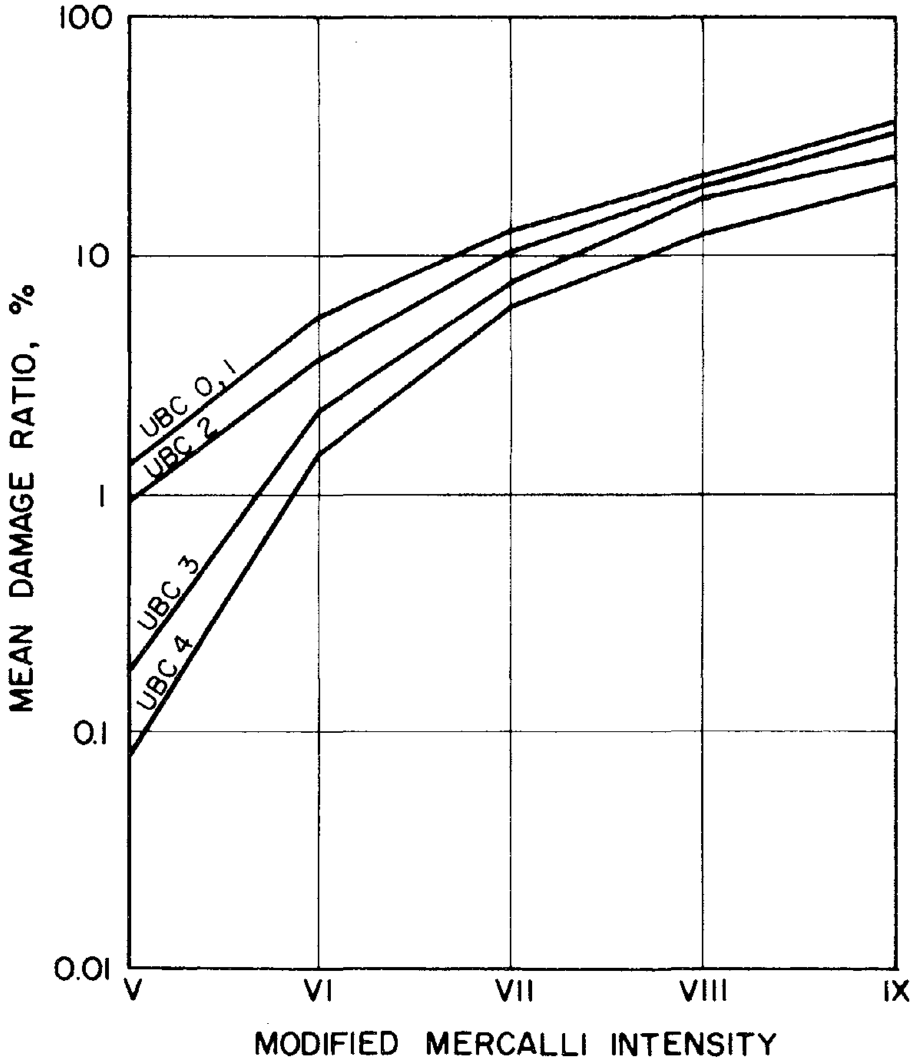


FIGURE 4.9 THEORETICAL DAMAGE ESTIMATES FOR CONCRETE MOMENT RESISTING FRAME BUILDINGS

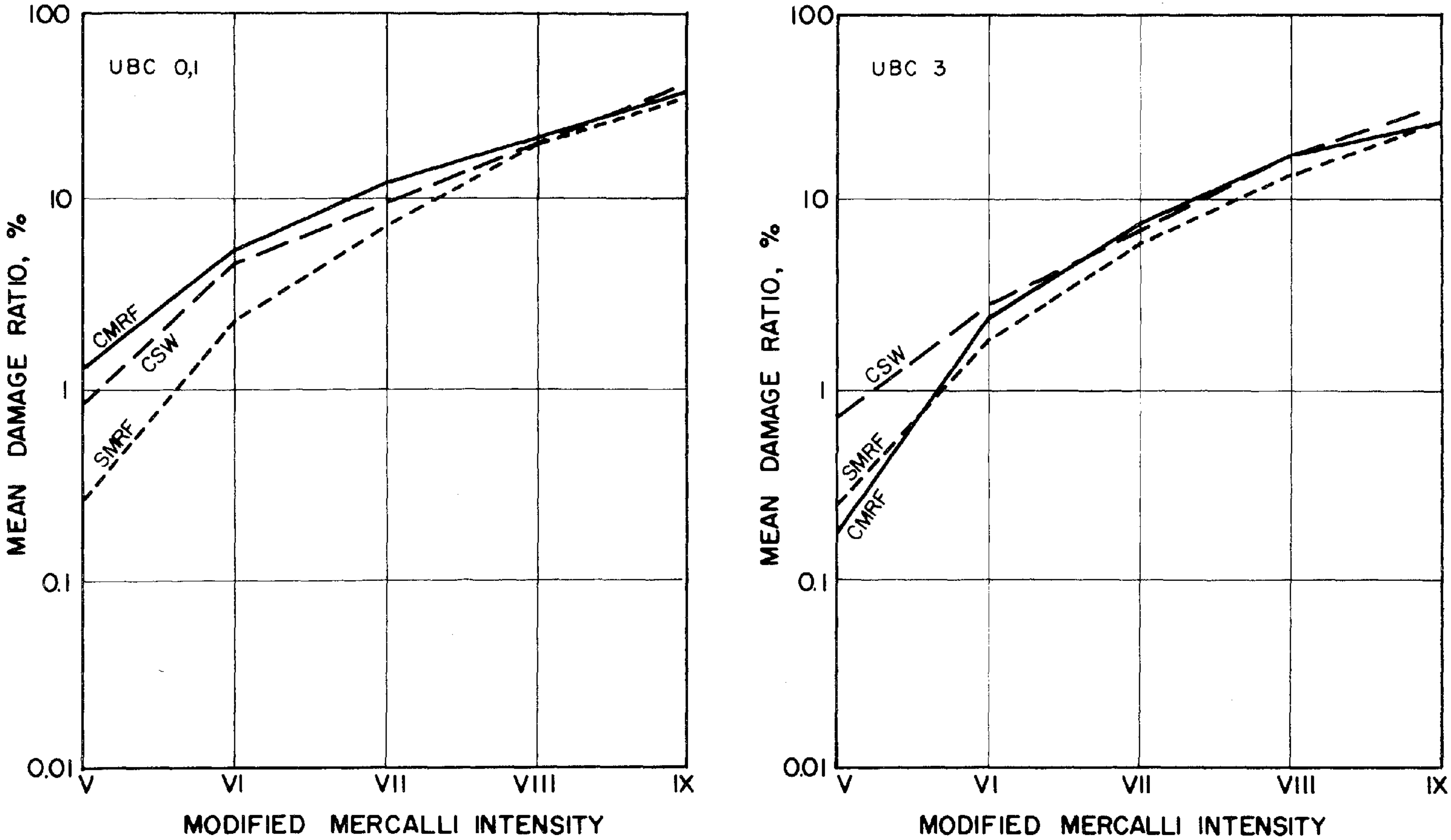


FIGURE 4.10 THEORETICAL DAMAGE ESTIMATES FOR PROTOTYPE BUILDINGS

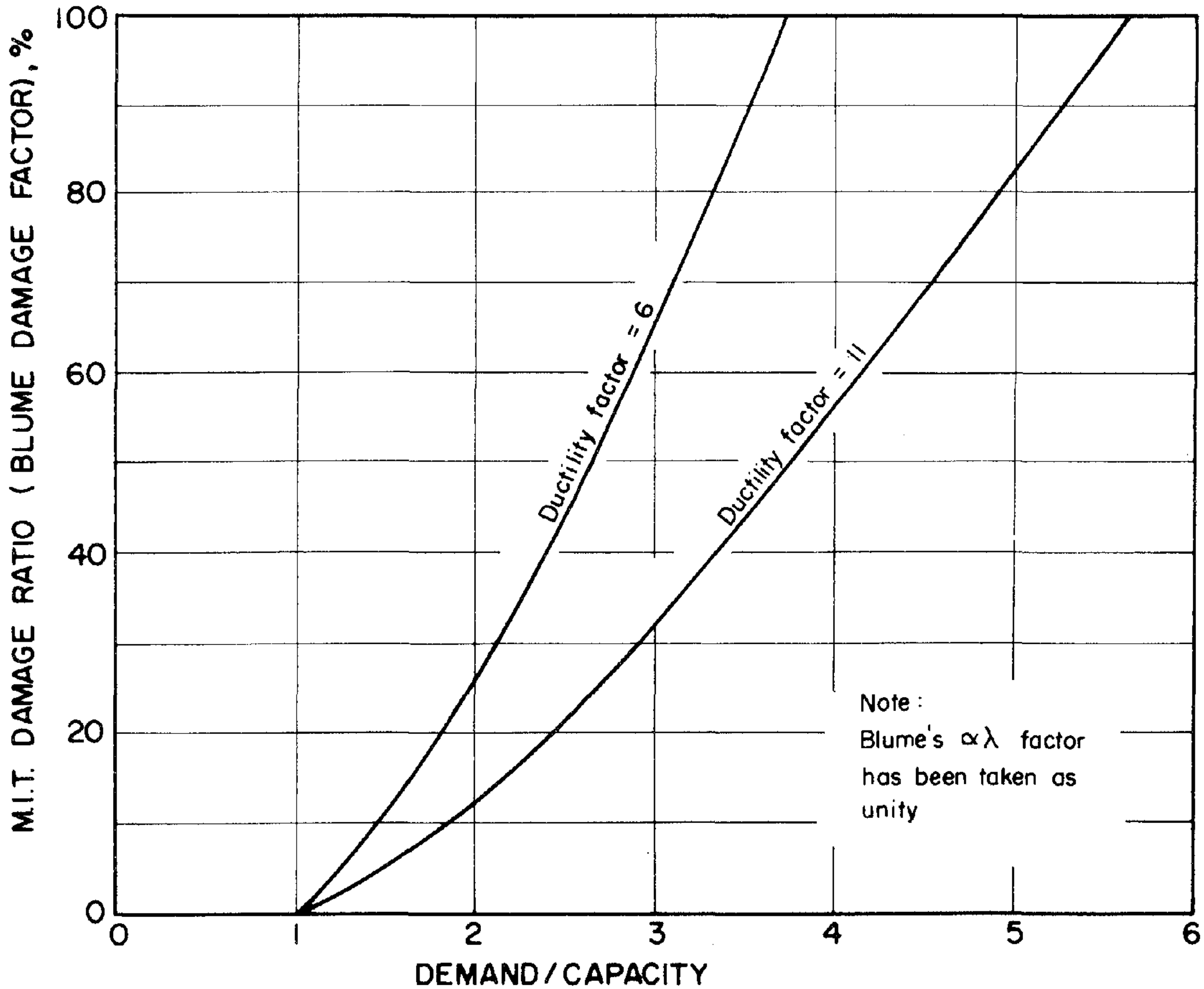


FIGURE 4.11 DAMAGE VS. DEMAND/CAPACITY RATIO (FROM BLUME & MUNROE, 1971)

74

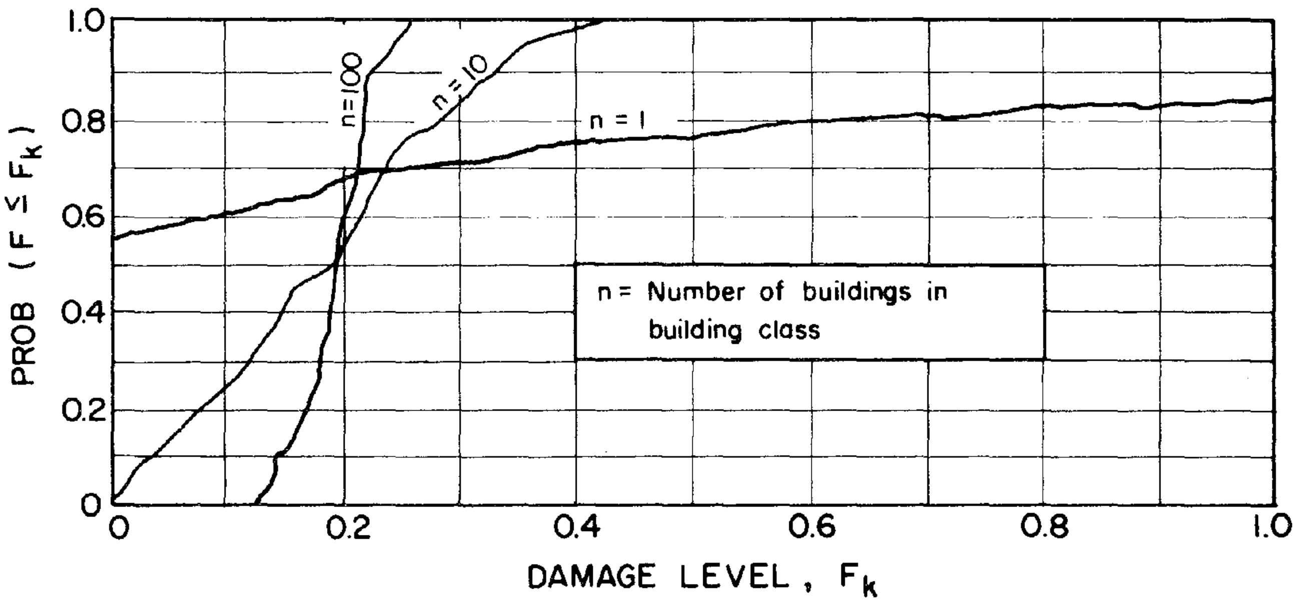


FIGURE 4.12 PROBABILITY THAT DAMAGE FACTOR (M.I.T.'S MEAN DAMAGE RATIO) EXCEEDS VARIOUS DAMAGE LEVELS (M.I.T.'S DAMAGE STATES) (FROM BLUME AND MUNROE, 1971)

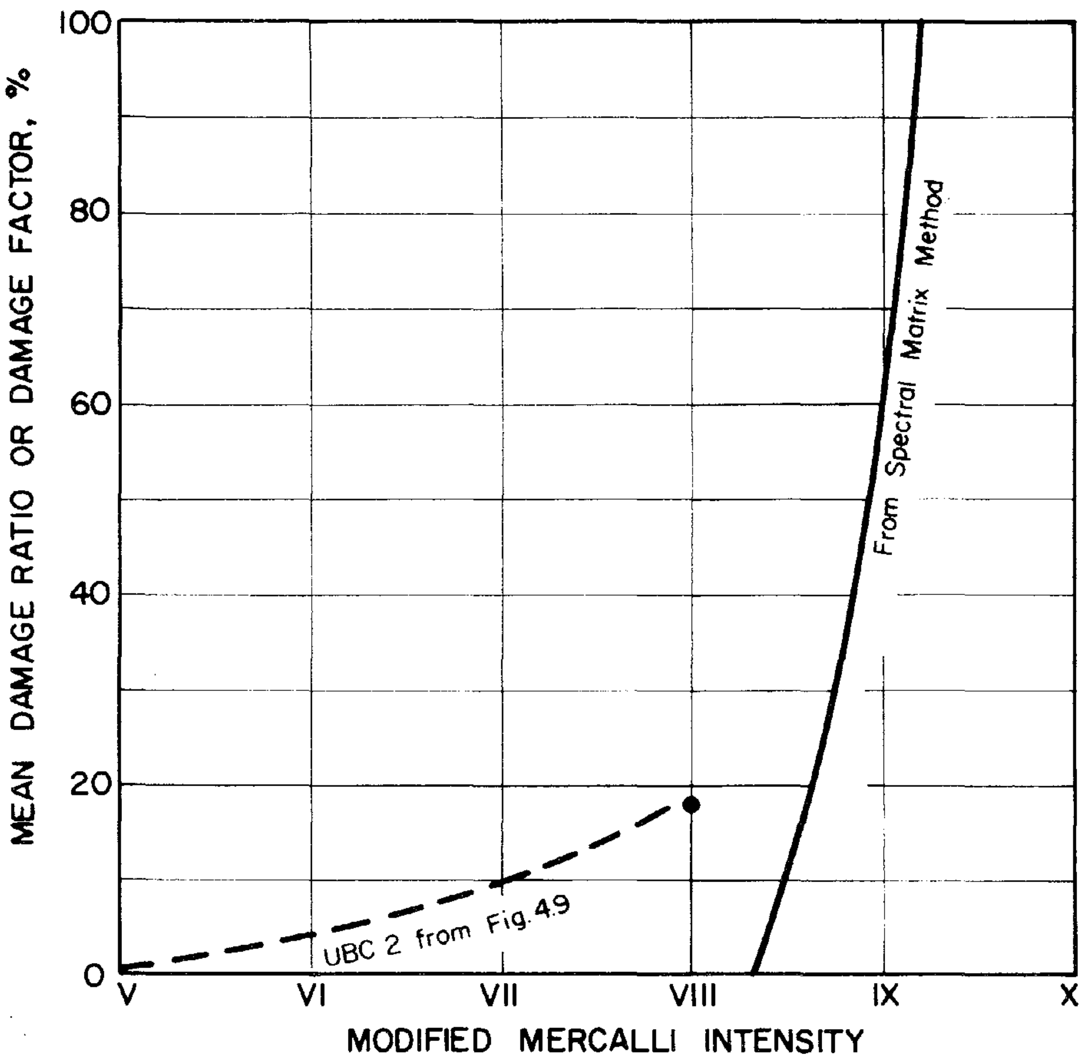


FIGURE 4.13 MEAN DAMAGE RATIO BY SPECTRAL MATRIX METHOD USING AVERAGE DEMAND AND CAPACITY

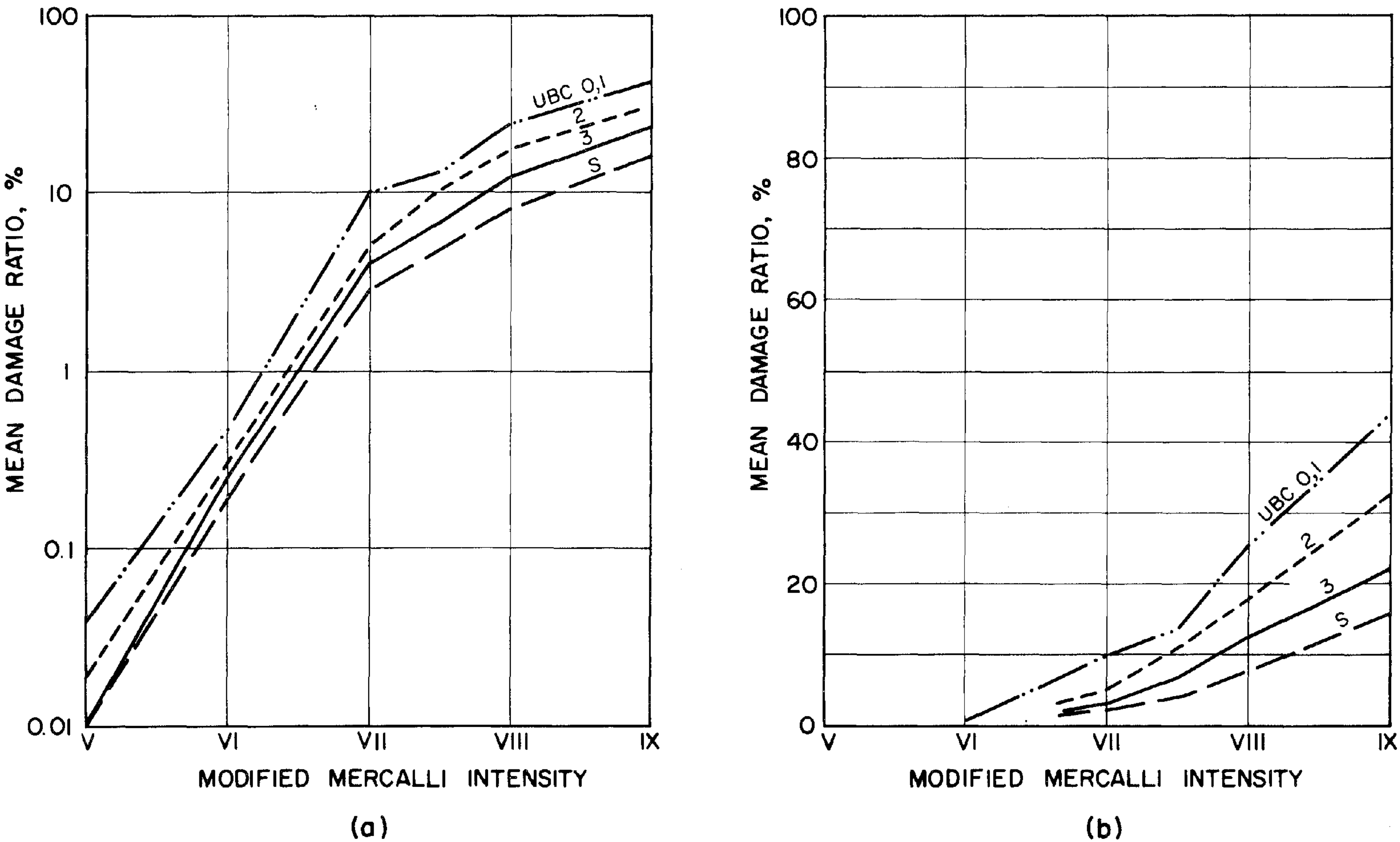
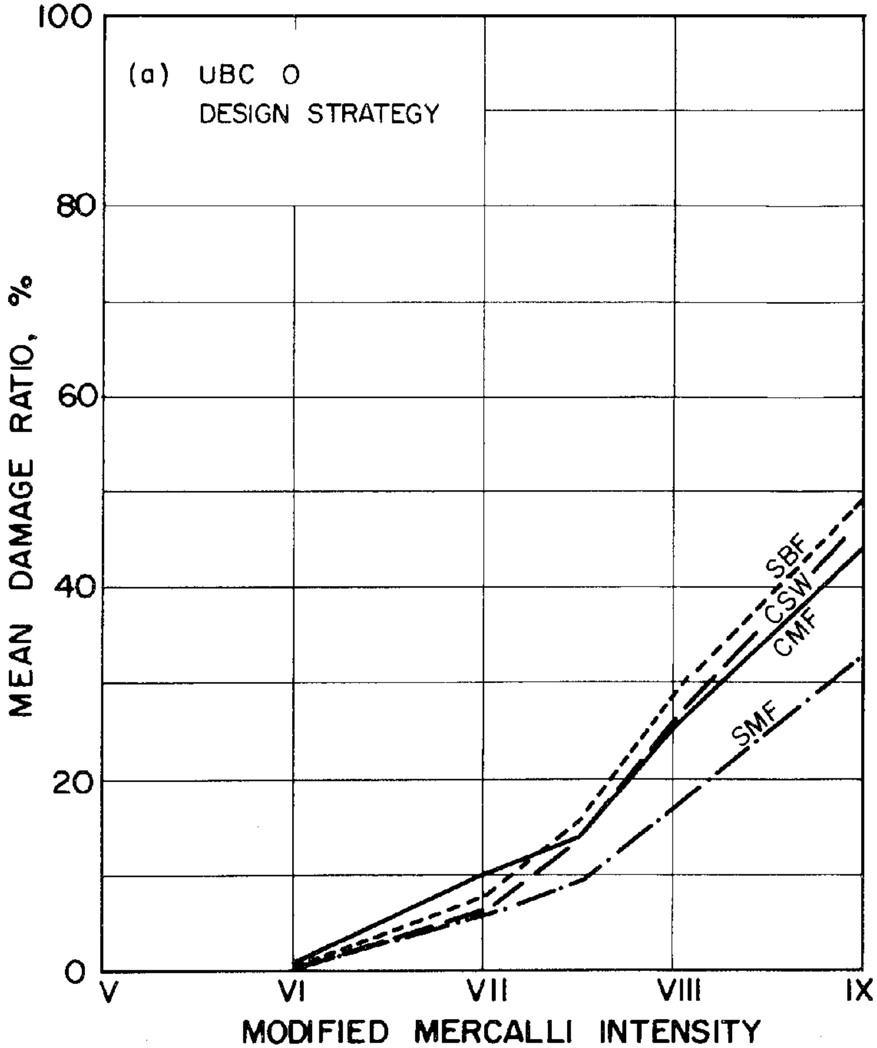


FIGURE 5.1 SUBJECTIVE MEAN DAMAGE RATIOS FOR CONCRETE FRAME BUILDINGS

77



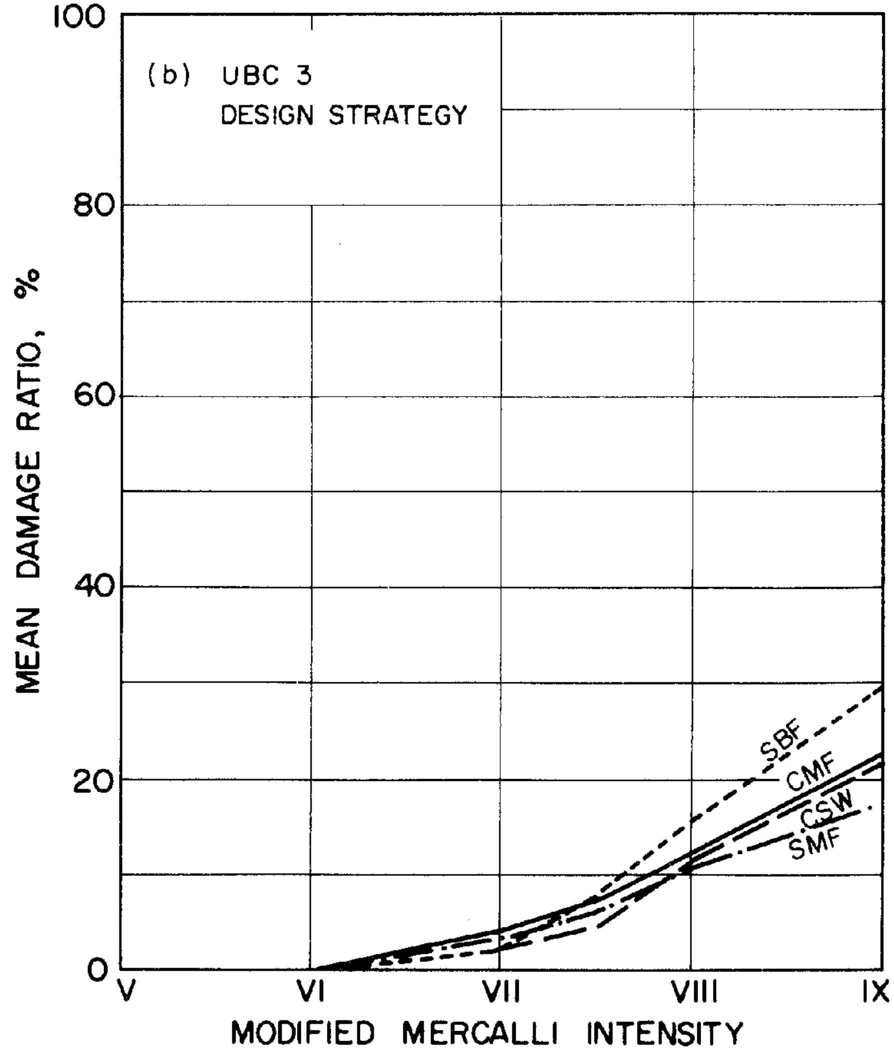


FIGURE 5.2 SUBJECTIVE MEAN DAMAGE RATIOS FOR VARIOUS STRUCTURAL SYSTEMS

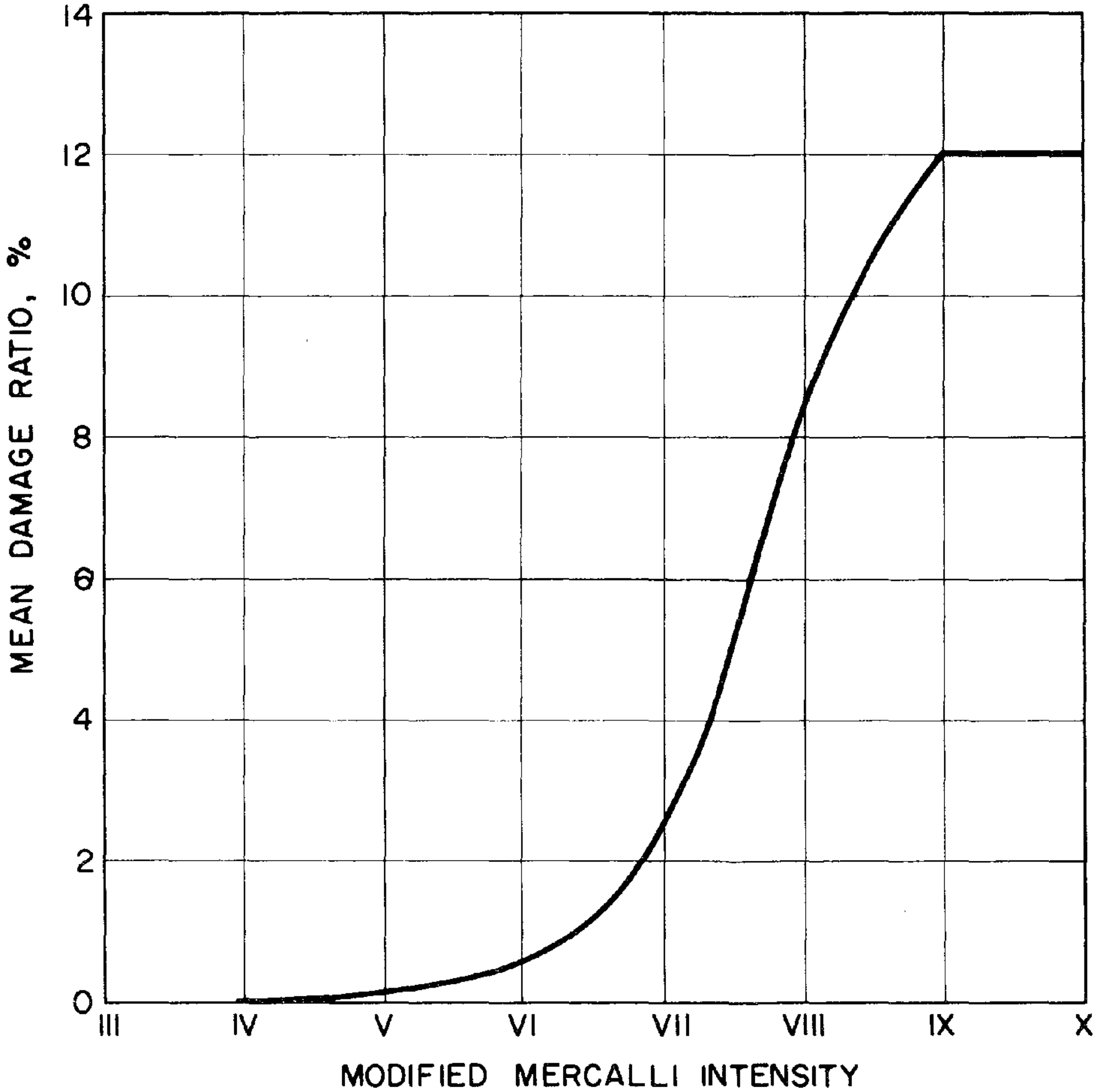


FIGURE 5.3 MEAN DAMAGE RATIO FOR WOODEN DWELLINGS (FROM USCGS, 1969)

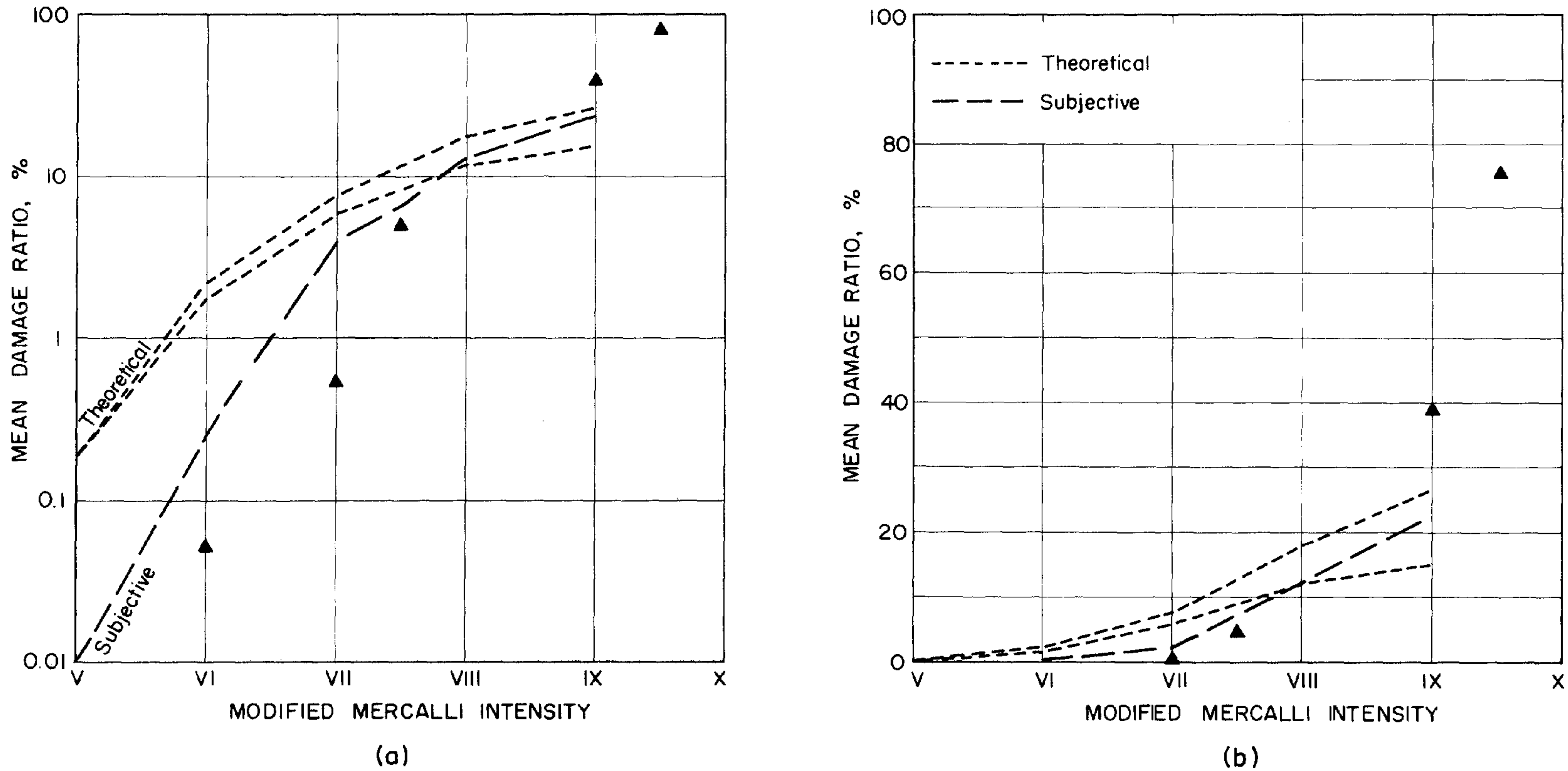


FIGURE 6.1 COMPARISON OF EMPIRICAL, THEORETICAL AND SUBJECTIVE MEAN DAMAGE RATIOS FOR CONCRETE FRAMES DESIGNED TO UBC 3

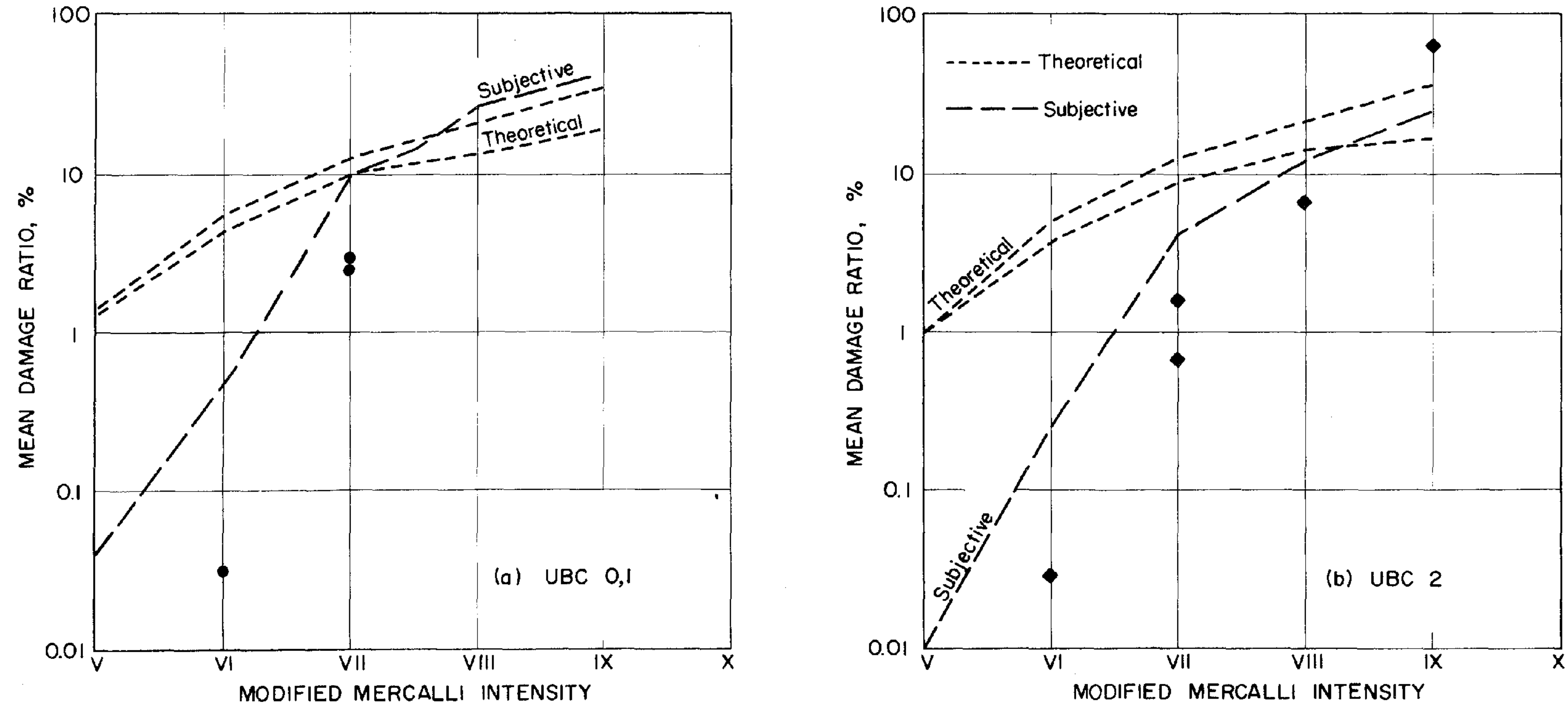


FIGURE 6.2 COMPARISON OF EMPIRICAL, THEORETICAL AND SUBJECTIVE MEAN DAMAGE RATIOS FOR CONCRETE FRAMES DESIGNED TO UBC 0 AND 2

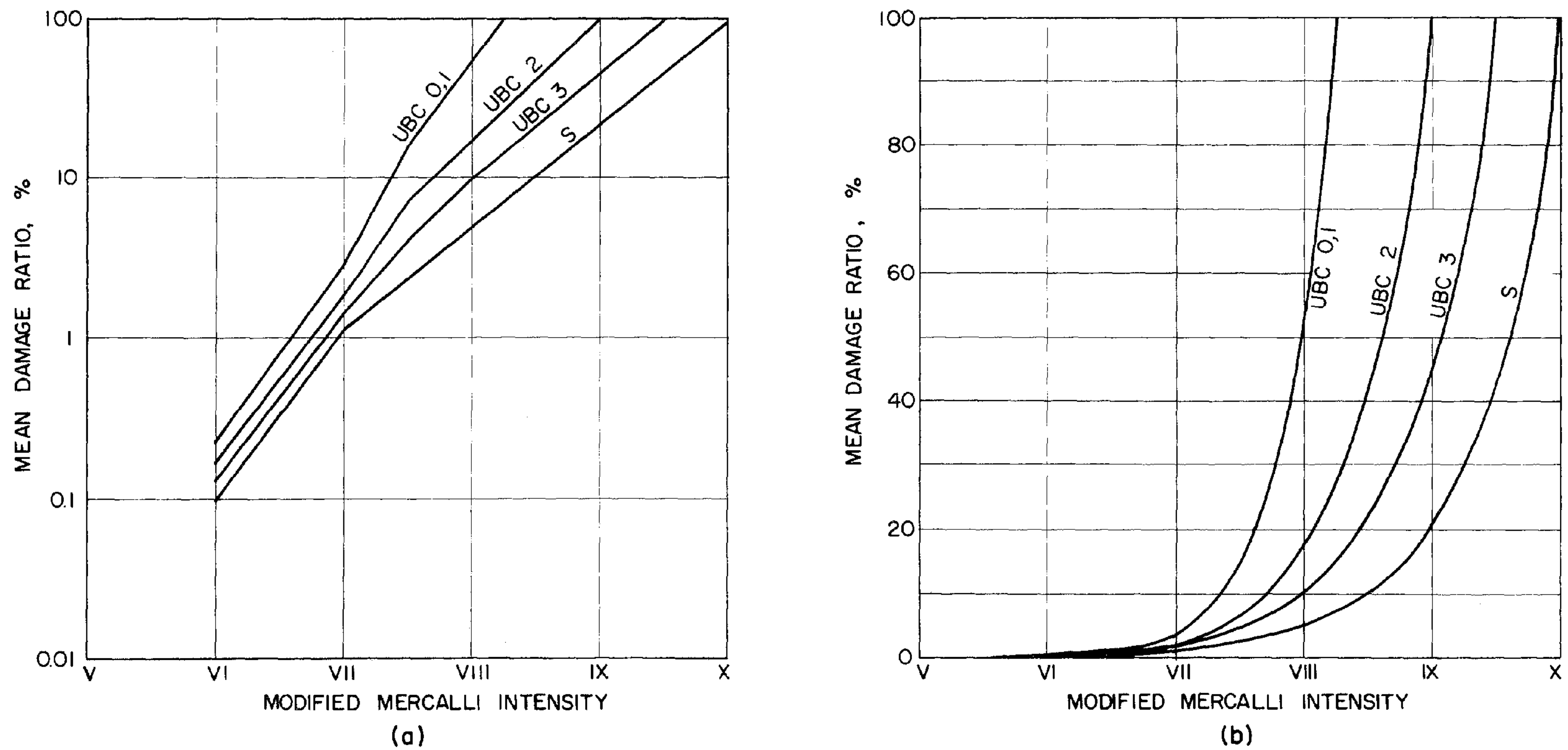


FIGURE 6.3 MEAN DAMAGE RATIOS FOR PILOT APPLICATION OF SEISMIC DESIGN DECISION ANALYSIS

82

APPENDIX A.

SYMBOLS USED IN CONNECTION WITH DAMAGE PROBABILITY

DPM: Damage probability matrix

DS: Damage state (see Table 2.1)

I: Intensity of ground motion shaking

MMI: Modified Mercalli intensity

P_{DSI}: Probability that a given building will experience damage state DS when subjected to ground motion intensity I

DR: Damage ratio (see Figure 2.2 or 2.3)

CDR: Central damage ratio (see Figure 2.2 or 2.3)

MDR: Mean damage ratio

APPENDIX B

DETERMINATION OF MEAN DAMAGE RATIO FROM THEORETICAL DYNAMIC RESPONSE

In this appendix is described the procedure used to generate damage probability matrices (DPM) and compute mean damage ratios (MDR) based upon theoretical analyses of the prototype buildings (See Chapter 4). Even though the results are very crude, a definite procedure was utilized in order to make the results consistent within themselves.

Two dynamic analyses were employed: One assumed purely elastic response, and the other computed inelastic response for an input level of 0.27g peak ground acceleration. Damage estimates were made for these cases and, in addition, for the "elastic limit" earthquake which was obtained by extrapolation of the elastic analysis results. The same time history input was used for both analyses, scaled to the selected ground acceleration. These analyses are described in Report 4 (Biggs and Grace, 1973).

Summary of Procedure

The procedure consists of the following steps:

1. Each analysis is related to the MMI using the Gutenberg-Richter correlation shown in Table 4.1. Thus the associated intensities are,

Peak Acceleration	MMI
0.27g	IX
0.05g	VI.5
Elastic Limit	Varies with structure

   Results are also obtained for the "alternate" correlation shown in Table 4.1.

2. The interstory displacements computed in the analyses are used to estimate damage.
3. By extrapolation, damages are estimated for other MMI of interest.
4. A probable distribution of damage for a given MMI is assumed.
5. A probable distribution of peak ground acceleration for a given MMI is assumed.

6. Based upon the above, a DPM is generated for each building.
7. The MDR is computed for each MMI. These steps are described in more detail below.

Estimation of Damage from Interstory Displacements

The procedure used to obtain the damage estimates, which are summarized here, is more fully explained in Report No. 5 (Czarnecki, 1973). In essence, the components of the total building are segregated into categories (structure, partitions, etc.) and an estimate is made of the damage to each category expressed as a percentage of the value of the components in that category. For this purpose, the damage has been related to interstory displacements by functions such as those shown in Fig. 4.8.

In order to obtain the total damage to the building, a weighted average of the damage in the various categories is computed. The weighting factors, i.e., the fractions of total building value, are assumed as follows:

Structure	0.25
Partitions	0.05
Glazing	0.05
Brick Walls	0.05
Concrete masonry walls	0.05
Other	0.55

The "other" category includes all electrical and mechanical equipment, elevators, ceilings, etc.

The resulting damage estimates for the five prototype buildings are given in Tables B. 1 (0.05g motion - elastic response), B.2 (elastic limit motion), and B.3 (0.27g motion-inelastic response). The input levels corresponding to the elastic limits are given in Table B.4. A summary of all total damage estimates is given in Table B.5. These values are taken to be the median of the expected damage for the given peak ground acceleration.

For simplicity, the damages for the two heights of CMRF and CSW buildings have been combined. Thus the DPM's for these two building types are based upon the average damage for the two heights analyzed.

Generation of DPM's

The generation of a typical DPM is illustrated in Fig. B.1. Basically, two assumptions are made:

1. For each MMI, a probability distribution for damages above and below the median previously established is assumed. This is intended to reflect variations in building properties which affect dynamic response and the relation between damage and response.
2. It is further assumed that there is a 50% probability that the peak ground acceleration for a given MMI would in fact be the value given in Table 4.1, and a 25% probability that the acceleration would be that associated with the next higher or next lower MMI.

The steps in the DPM generation are the following:

1. Plot the three computed median damage ratios on the matrix. For this purpose the matrix is considered to be a continuous plot of damage ratio vs. MMI. The plotting involves relating ground acceleration to intensity and damage ratio to damage state, using the ranges for each state shown in Fig. 2.1. The three points are shown in Fig. B.1(a).
2. Based upon these three points, a curve is drawn over the range of MMI from IV to X. Obviously, the curve is very approximate at the lower intensities.
3. Distribute a total of 50% vertically in each MMI column. 50% is used because of the assumption of 50% probability that the acceleration would actually be that associated with this MMI. If the median curve is near the center of the damage state, 25% is assigned to that state and 12.5% to each of the adjacent states. If the curve intersects the boundary between states the assumed distribution is 5-20-20-5. For intermediate cases distributions of 10-25-15 or 15-25-10 are used. The resulting probabilities are shown in Fig. B.1 (a).

4. Based upon the assumed 25% probability that the acceleration would be that associated with the next higher or next lower MMI, one-half of each number in Fig. B.1 (a) is carried over horizontally to the adjacent columns. The resulting summation in each box is shown in Fig. B.1 (b). Each column now totals 100%. Note that intersities IV and X were included in B.1 (a) only to permit this operation.
5. The probabilities are now rounded off to the nearest 5%. The final result is shown in Fig. B.1 (c).
6. The mean damage ratio (MDR) for each MMI is computed by

$$MDR_I = \sum_{DS} \left(P_{DSI}\right) \left(CDR_{DS}\right)$$

where P_{DSI} = probability that the damage lies in state DS and CDR_{DS} = central damage ratio for state DS as given in Table 2.2. The result for the example case is shown in Fig. B.1 (c).

A summary of the MDR's computed for all the prototype buildings is given in Table B.6.

<u>Alternate Acceleration - MMI Correlation</u>

Because of the uncertainties discussed in Section 4.1.2, results were also obtained for the alternate acceleration - MMI correlation shown in Table 4.1. The resulting MDR's,obtained by the procedure described above, are given in Table B.7. The predicted damages for this correlation are of course higher for the larger intensities.

<u>Comments</u>

The theoretical damage estimates made here are obviously very crude. There are many uncertainties involved: The relation between MMI and ground motion, the characterization of the motion, the actual response of a building to that motion, the relation between damage and response, the variability in building properties, etc. The intent has been to develop a theoretical approach to the problem in the hope that future studies will reduce the range of uncertainties and increase the reliability of the prediction.

TABLE B.1

DAMAGE TO TYPICAL BUILDINGS - 0.05g EARTHQUAKE

Building - - - Zone	Structural Damage	Partition Damage	Glass Damage	Unreinf. Masonry Brick Wall Damage	Reinforced Masonry Brick Wall Damage	Unreinf. Concrete Brick Wall Damage	Reinforced Concrete Brick Wall Damage	Other Damages	Total Damage
11-CMRF									
0,1	0.0	23.6	0.0	0.0	0.0	0.0	0.0	0.0	1.2
2	0.0	23.5	0.0	0.0	0.0	0.0	0.0	0.0	1.2
3	0.1	17.9	0.0	0.0	0.0	0.0	0.0	0.0	0.9
4	0.0	12.7	0.0	0.0	0.0	0.0	0.0	0.0	0.6
6-CMRF									
0,1	0.4	45.5	0.0	78.1	66.2	75.9	60.1	0.0	11.1
2	0.4	45.5	0.0	78.1	66.2	75.9	60.1	0.0	11.1
3	0.1	33.1	0.0	31.4	22.6	30.2	19.3	0.0	3.1
4	0.0	21.4	0.0	0.0	0.0	0.0	0.0	0.0	1.1
17-CSW									
0	7.5	17.7	0.0	0.0	0.0	0.0	0.0	0.0	2.8
1	7.5	17.7	0.0	0.0	0.0	0.0	0.0	0.0	2.8
2	13.0	16.7	0.0	0.0	0.0	0.0	0.0	0.0	4.0
3	16.0	14.5	0.0	0.0	0.0	0.0	0.0	0.0	4.7
4	18.0	13.0	0.0	0.0	0.0	0.0	0.0	0.0	5.1
11-CSW									
0,1.2	9.1	17.1	0.0	0.0	0.0	0.0	0.0	0.0	3.2
3	13.8	12.6	0.0	0.0	0.0	0.0	0.0	0.0	4.1
4	16.5	10.2	0.0	0.0	0.0	0.0	0.0	0.0	4.6
11-SMRF									
0,1	0.0	33.9	0.0	15.7	10.6	14.4	9.1	0.0	3.5
2	0.0	32.9	0.0	14.8	10.0	13.6	8.5	0.0	3.0
3	0.0	29.0	0.0	7.3	4.9	6.7	4.2	0.0	2.0
4	0.0	21.8	0.0	0.0	0.0	0.0	0.0	0.0	1.1

TABLE B.2

DAMAGES TO TYPICAL BUILDINGS FOR THE ELASTIC LIMIT EARTHQUAKE

Building --- Zone	Structural Damage	Partition Damage	Glass Damage	Unreinf. Masonry Brick Wall Damage	Reinforced Masonry Brick Wall Damage	Unreinf. Concrete Brick Wall Damage	Reinforced Concrete Brick Wall Damage	Other Damages	Total Damage
11-CMRF									
0,1	1.1	61.7	0.7	89.0	84.0	88.4	80.3	0.1	12.3
2	1.3	66.4	0.1	90.2	86.9	89.5	83.6	0.1	12.8
3	1.4	56.8	0.4	81.8	76.6	81.3	73.3	0.0	10.7
4	1.0	42.4	0.0	61.8	54.4	60.5	49.8	0.0	7.6
6-CMRF									
0,1	1.5	74.2	1.4	100.0	95.3	99.5	93.6	0.0	14.1
2	1.5	74.2	1.4	100.0	95.3	99.5	93.6	0.1	14.1
3	1.3	70.2	1.1	97.6	93.0	96.4	91.6	0.1	13.1
4	0.8	58.1	0.5	83.3	80.0	83.3	76.4	0.1	10.9
17-CSW									
0	7.5	11.7	0.0	0.0	0.0	0.0	0.0	0.0	2.5
1	7.5	15.2	0.0	0.0	0.0	0.0	0.0	0.0	2.1
2	13.1	23.7	0.0	0.0	0.0	0.0	0.0	0.0	4.5
3	16.1	30.2	0.0	30.1	20.4	27.7	17.4	0.0	7.8
4	18.3	49.2	0.0	75.3	71.1	74.7	68.2	0.0	13.9
11-CSW									
0,1,2	9.1	13.2	0.0	0.0	0.0	0.0	0.0	0.0	2.9
3	13.8	15.9	0.0	0.0	0.0	0.0	0.0	0.0	4.3
4	16.6	20.2	0.0	0.0	0.0	0.0	0.0	0.0	5.2
11-SMRF									
0,1	0.0	21.0	0.0	0.0	0.0	0.0	0.0	0.0	1.0
2	0.0	31.0	0.0	0.0	0.0	0.0	0.0	0.0	1.5
3	0.0	29.0	0.0	0.0	0.0	0.0	0.0	0.0	1.4
4	0.0	26.1	0.0	16.0	10.9	14.7	9.3	0.0	2.5

TABLE B.3

DAMAGES TO TYPICAL BUILDINGS FOR 0.27g EARTHQUAKES

Building - - - Zone	Structural Damage	Partition Damage	Glass Damage	Unreinf. Masonry Brick Wall Damage	Reinforced Masonry Brick Wall Damage	Unreinf. Concrete Brick Wall Damage	Reinforced Concrete Brick Wall Damage	Other Damages	Total Damage
11-CMRF									
0,1	2.5	79.0	12.0	81.8	81.8	81.8	81.8	0.1	13.5
2	3.1	88.6	21.2	90.9	90.9	90.9	90.9	0.1	15.1
3	2.7	80.3	3.0	90.9	88.7	90.9	87.7	0.1	13.9
4	1.8	60.8	0.5	81.8	79.4	81.6	78.4	0.1	11.5
6-CMRF									
0,1	5.6	95.9	52.4	100.0	100.0	100.0	100.0	0.2	19.0
2	5.7	95.9	67.9	100.0	100.0	100.0	100.0	0.2	19.8
3	4.5	96.9	36.2	100.0	100.0	100.0	100.0	0.2	17.9
4	2.3	81.0	3.8	83.3	83.3	83.3	83.3	0.1	13.1
17-CSW									
0	11.3	55.4	0.2	99.8	87.4	99.2	79.9	0.0	15.4
1	10.9	56.4	0.1	93.6	89.2	93.2	84.4	0.0	14.6
2	21.0	69.0	2.9	75.3	72.2	74.8	71.1	0.1	16.9
3	30.0	67.6	1.5	82.1	79.6	81.6	77.9	0.1	19.5
4	27.3	50.7	0.2	70.1	66.1	69.6	63.5	0.0	16.0
11-CSW									
0,1,2	15.7	76.8	20.2	95.6	88.2	93.8	85.6	0.1	18.3
3	18.7	44.2	0.0	63.6	63.0	63.6	59.4	0.0	12.3
4	22.5	37.3	0.0	54.5	49.4	54.5	42.5	0.0	12.1
11-SMRF									
0,1	<0.1	78.5	11.4	100.0	100.0	100.0	100.0	0.1	14.5
2	<0.1	81.6	28.1	100.0	100.0	100.0	98.8	0.1	15.4
3	<0.1	85.2	12.7	100.0	98.9	100.0	97.8	0.1	14.7
4	<0.1	69.2	10.7	98.4	88.1	96.5	84.5	0.1	13.7

TABLE B.4

PEAK ACCELERATION OF THE ELASTIC LIMIT EARTHQUAKE

Building Type Design Zone	Peak Acceleration of Elastic Limit Earthquake
11-CMRF	
0,1	0.131g
2	0.141g
3	0.159g
4	0.167g
6-CMRF	
0,1	0.082g
2	0.082g
3	0.106g
4	0.136g
17-CSW	
0	0.0033g
1	0.043g
2	0.071g
3	0.104g
4	0.190g
11-CSW	
0,1,2	0.0386g
3	0.0633g
4	0.0995g
11-SMRF	
0,1	0.031g
2	0.047g
3	0.050g
4	0.060g

TABLE B.5

SUMMARY OF DAMAGES TO TYPICAL BUILDINGS
FOR THREE EARTHQUAKES

Building Type Design Zone	Total Damage 0.05g Earthquake	Total Damage Elastic Limit Earthquake	Total Damage 0.27g Earthquake
11-CMRF			
0,1	1.2	12.3	13.5
2	1.2	12.8	15.1
3	0.9	10.7	13.9
4	0.6	7.6	11.5
6 CMRF			
0,1	11.1	14.1	19.0
2	11.1	14.1	19.8
3	3.1	13.1	17.9
4	1.1	10.9	13.1
17-CSW			
0	2.8	2.5	15.4
1	2.8	2.1	14.6
2	4.0	4.5	16.9
3	4.7	7.8	19.5
4	5.1	13.9	16.0
11-CSW			
0,1,2	3.2	2.9	18.3
3	4.1	4.3	12.3
4	4.6	5.2	12.1
11-SMRF			
0,1	3.5	1.0	14.5
2	3.0	1.5	15.4
3	2.0	1.4	14.7
4	1.1	2.5	13.7

TABLE B.6

SUMMARY OR RESULTS

MEAN DAMAGE RATIOS

GUTENBERG-RICHTER ACCELERATION-INTENSITY CORRELATION

		INTENSITY (MODIFIED MERCALLI)				
BLDG. TYPE	ZONE	5	6	7	8	9
	0, 1, 2	0.88	2.61	6.53	12.1	20.7
CSW	3	0.68	2.46	6.30	10.9	15.0
	4	0.56	2.46	6.30	10.8	13.7
	0, 1	1.18	4.24	10.2	13.8	19.5
CMRF	2	0.93	3.58	8.57	13.4	16.3
	3	0.18	1.70	5.96	12.1	15.0
	4	0.06	0.68	4.12	10.2	12.5
	0, 1	0.27	1.83	5.98	10.8	19.5
SMRF	2	0.26	1.68	5.81	9.2	19.1
	3	0.26	1.60	5.18	8.58	19.1
	4	0.21	0.85	3.30	6.53	18.4

TABLE B.7

SUMMARY OF RESULTS

MEAN DAMAGE RATIOS

ALTERNATE ACCELERATION - INTENSITY CORRELATION

		INTENSITY (MODIFIED MERCALLI)				
BLDG. TYPE	ZONE	5	6	7	8	9
	0, 1, 2	0.85	4.56	9.58	19.1	40.3
CSW	3	0.71	2.71	6.93	17.9	30.8
	4	0.58	2.47	6.53	16.6	26.5
	0, 1	1.33	5.46	12.1	20.8	36.8
CMRF	2	0.93	4.83	10.3	19.5	32.2
	3	0.18	2.19	7.86	17.9	26.5
	4	0.08	1.42	6.11	12.1	19.5
	0, 1	0.27	2.32	7.38	19.1	36.8
SMRF	2	0.26	1.92	6.21	17.9	32.3
	3	0.26	1.82	5.98	13.1	26.5
	4	0.21	0.97	4.86	10.2	20.8

B12

MMI

DAMAGE STATE	IV	V	VI	VII	VIII	IX	X
0	35	5					
1	15	20					
2		20	5				
3		5	20	5			
4			20	20	10	10	5
5			5	20	25	25	20
6				5	15	15	20
7							5
8							

0.05g

Elastic limit - 0.11g

0.27g

(a)

DAMAGE STATE	V	VI	VII	VIII	IX
0	22.5	2.5			
1	27.5	10			
2	22.5	15	2.5		
3	15	25	15	2.5	
4	10	30	35	25	17.5
5	2.5	15	35	47.5	47.5
6		2.5	12.5	25	32.5
7					2.5
8					

(b)

DAMAGE STATE	V	VI	VII	VIII	IX
0	25	5			
1	30	10			
2	20	15	5		
3	15	25	15	5	
4	10	30	35	25	15
5		15	35	45	50
6			10	25	35
7					
8					
MDR	0.9	3.6	8.6	13.4	16.3 %

(c)

FIGURE B.1 GENERATION OF DAMAGE PROBABILITY MATRIX
REINFORCED CONCRETE FRAME - ZONE 2

APPENDIX C

SUBJECTIVE DAMAGE PROBABILITIES

Fig. C.1 shows one of the DPM provided by S.B. Barnes & Associates, based upon subjective judgement. Two different engineers independently prepared a matrix for each case, and both estimates are shown in Fig. C.1 for comparison.

Table C.1 gives the mean damage ratios for each of the structural systems and story heights. These were computed from the DPM using Eq. 2.2.

Table C.2 gives the damage probabilities P_8 and P_{7+8}. Here the numbers are averages of these estimated by the two engineers.

Table C.1

MEAN DAMAGE RATIOS (%) FROM SUBJECTIVE DAMAGE PROBABILITY MATRICES

Each pair of lines in the following table gives the values from the two different engineers.

Building Type	Design Strategy	Modified Mercalli Intensity V	VI	VII	VII.5	VIII	IX
CMRF-6	UBC 0,1	0.02	0.22	10.3	14.8	21.5	47.2
		0.06	0.78	7.9	8.4	31.6	45.8
	UBC 2	0.02	0.25	5.3	9.6	16.2	33.2
		0.02	0.28	4.2	7.7	16.4	33.0
	UBC 3	0.02	0.18	4.0	7.2	11.0	22.8
		0.01	0.24	3.7	6.1	12.9	19.6
	S	0.02	0.18	3.3	5.6	7.4	15.6
		0.01	0.16	2.6	3.6	7.6	15.7
CMRF-11	UBC 0,1	0.02	0.25	15.0	20.0	28.7	51.4
		0.06	0.88	5.5	7.3	22.6	38.8
	UBC 2	0.02	0.25	5.9	11.7	22.3	35.3
		0.02	0.62	4.4	6.9	16.8	32.4
	UBC 3	0.02	0.25	4.4	8.5	13.3	24.3
		0.01	0.38	3.8	5.7	11.6	19.5
	S	0.02	0.25	3.4	6.0	8.7	17.0
		0.01	0.27	2.8	3.8	7.5	15.4
CMRF-17	UBC 0,1	0.02	0.40	17.7	22.7	32.5	56.8
		0.03	0.32	3.7	8.5	16.2	24.1
	UBC 2	0.02	0.39	6.9	12.8	25.0	39.5
		0.02	0.13	3.6	18.7	13.0	23.3
	UBC 3	0.02	0.39	4.6	9.1	15.0	27.5
		0.01	0.14	3.6	7.3	11.9	18.2
	S	0.02	0.39	3.6	6.4	8.1	18.3
		0.01	0.05	2.3	3.6	9.0	13.9

Building Type	Design Strategy	Modified Mercalli Intensity V	VI	VII	VII.5	VIII	IX
SMRF-6	UBC 0,1	0.02	0.29	7.1	12.0	18.0	37.3
		0.05	0.23	7.3	8.8	20.4	38.9
	UBC 2	0.02	0.21	5.8	10.7	14.5	36.6
		0.02	0.38	3.8	6.5	16.1	28.3
	UBC 3	0.02	0.15	3.9	8.4	10.6	21.0
		0.02	0.24	3.5	5.1	12.8	18.3
	S	0.02	0.12	3.1	7.5	10.1	16.5
		0.01	0.12	2.5	3.3	7.5	14.9
SMRF-11	UBC 0,1	0.02	0.25	5.8	11.5	16.6	34.8
		0.06	2.6	6.7	8.5	19.6	37.7
	UBC 2	0.02	0.17	5.2	8.9	14.0	31.0
		0.03	1.8	4.1	6.2	15.5	30.3
	UBC 3	0.02	0.15	3.1	6.9	9.1	18.0
		0.02	0.42	3.6	5.5	11.2	16.1
	S	0.02	0.12	3.1	6.5	8.2	13.6
		0.01	0.30	2.7	3.7	7.8	14.2
SMRF-17	UBC 0,1	0.02	0.18	5.5	8.9	14.5	30.4
		0.03	0.23	2.7	7.9	13.4	23.5
	UBC 2	0.02	0.15	4.1	8.1	12.7	25.7
		0.03	0.15	4.0	7.3	12.9	18.4
	UBC 3	0.02	0.15	2.9	6.4	8.1	16.4
		0.02	0.12	3.6	6.7	11.6	16.0
	S	0.02	0.12	3.1	5.4	7.8	12.5
		0.01	0.05	2.0	3.3	7.6	12.2
CSW-6	UBC 0,1	0.01	0.06	6.8	14.3	25.2	46.1
		0.04	0.37	9.2	9.0	21.7	47.8
	UBC 2	0.01	0.05	2.0	4.3	12.4	27.8
		0.01	0.45	2.6	6.3	17.5	34.2
	UBC 3	0.01	0.03	0.7	1.9	5.9	12.5
		0.01	0.14	2.6	5.0	14.3	26.7
	S	0.01	0.02	0.5	1.2	2.9	6.4
		0.01	0.05	1.8	2.9	6.4	18.5

Building Type	Design Strategy	Modified Mercalli Intensity V	VI	VII	VII.5	VIII	IX
CSW-11	UBC 0,1	0.01	0.06	7.0	15.9	27.7	48.3
		0.04	0.63	5.5	12.1	26.4	44.2
	UBC 2	0.01	0.05	2.2	5.9	13.8	30.1
		0.02	0.30	5.7	11.6	25.4	45.4
	UBC 3	0.01	0.03	1.0	2.5	7.7	14.8
		0.01	0.18	4.0	7.3	14.7	29.3
	S	0.01	0.02	0.5	1.4	3.4	7.0
		0.01	0.08	2.1	5.2	15.4	23.4
CSW-17	UBC 0,1	0.04	0.08	7.4	16.6	29.3	50.7
		0.02	0.22	5.8	11.8	27.1	48.4
	UBC 2	0.01	0.07	2.6	6.3	17.5	32.4
		0.02	0.25	5.8	11.9	27.2	45.4
	UBC 3	0.01	0.05	1.2	2.9	9.2	16.2
		0.01	0.18	4.4	8.7	17.3	32.1
	S	0.01	0.03	0.7	1.8	3.7	9.7
		0.01	0.05	2.3	6.1	14.3	22.7
SBF-6	UBC 0,1	0.01	0.16	7.2	16.7	28.8	47.3
		0.04	0.28	8.8	11.4	22.7	48.2
	UBC 2	0.01	0.06	2.3	10.4	16.0	37.4
		0.01	0.32	5.5	10.2	21.1	38.9
	UBC 3	0.01	0.03	1.0	7.0	11.1	22.5
		0.02	0.14	3.5	7.0	16.1	29.7
	S	0.01	0.02	0.5	4.0	7.0	14.2
		0.01	0.06	1.9	5.4	9.4	19.9
SBF-11	UBC 0,1	0.01	0.16	9.3	19.2	33.0	51.9
		0.02	0.13	6.1	11.1	26.5	46.2
	UBC 2	0.01	0.06	3.4	14.3	21.1	43.0
		0.02	0.32	5.5	11.2	25.0	41.0
	UBC 3	0.01	0.03	1.4	7.6	12.9	25.2
		0.02	0.14	2.7	8.6	19.8	35.0
	S	0.01	0.02	0.7	4.7	8.8	16.7
		0.01	0.05	2.3	5.1	11.9	21.9

Building Type	Design Strategy	Modified Mercalli Intensity V	VI	VII	VII.5	VIII	IX
SBF-17	UBC 0,1	0.01	0.16	9.7	22.8	36.8	57.7
		0.02	0.13	5.9	13.4	27.3	46.7
	UBC 2	0.01	0.06	4.8	16.9	23.8	47.2
		0.02	0.17	5.9	12.2	27.4	45.1
	UBC 3	0.01	0.03	1.9	9.5	15.6	27.7
		0.02	0.14	4.9	10.4	21.7	39.1
	S	0.01	0.02	0.8	5.2	10.6	20.2
		0.01	0.05	2.4	6.2	13.1	22.0

Table C.2

SUBJECTIVE PROBABILITIES (%) FOR

DAMAGE STATE (7+8) AND DAMAGE STATE B

Design Strategy	Building Type	Damage State DS 8				Damage State DS 7+8			
		VII	VII.5	VIII	IX	VII	VII.5	VIII	IX
UBC 0,1	CMRF	2	2	4	10	4	6	12	32
	CSW	0	2	5	12	2	6	16	37
	SMRF	0	0	1	4	0	2	7	22
	SBF	0	2	6	13	2	6	19	38
UBC 2	CMRF	0	0	1	4	0	3	7	19
	CSW	0	0	4	9	0	3	11	26
	SMRF	0	0	0	2	0	1	6	16
	SBF	0	2	5	11	0	6	14	32
UBC 3	CMRF	0	0	0	2	0	1.5	4	8
	CSW	0	0	1	2	0	1	5	12
	SMRF	0	0	0	0	0	1	3	6
	SBF	0	0	2	6	0	3	9	20
S	CMRF	0	0	0	0	0	0	1	4
	CSW	0	0	0	1	0	0	3	6
	SMRF	0	0	0	0	0	0	2	3
	SBF	0	0	1	3	0	1	4	10

DAMAGE STATE	INTENSITY (MODIFIED MERCALLI)					
	V	VI	VII	VII.5	VIII	IX
0	92 / 89	50 / 60	0 / 7	0 / 3	0 / 0	0 / 0
1	6 / 6	27 / 16	1 / 8	0 / 6	0 / 1	0 / 0
2	2 / 3	18 / 8	8 / 12	4 / 14	1 / 9	0 / 1
3	0 / 2	4 / 6	36 / 17	18 / 18	8 / 10	3 / 6
4	0 / 0	1 / 8	32 / 26	34 / 30	24 / 16	10 / 9
5	0 / 0	0 / 2	12 / 17	27 / 18	35 / 30	20 / 22
6	0 / 0	0 / 0	6 / 12	10 / 9	22 / 22	32 / 27
7	0 / 0	0 / 0	3 / 1	4 / 2	6 / 9	23 / 26
8	0 / 0	0 / 0	2 / 0	3 / 0	4 / 3	12 / 9

FIGURE C.1 SUBJECTIVE DAMAGE PROBABILITY MATRICES FOR CMRF-6 DESIGNED FOR UBC 0, BY TWO DIFFERENT ENGINEERS

Appendix D

DETERMINATION OF DAMAGE PROBABILITY MATRICES FOR PILOT SEISMIC DESIGN DECISION ANALYSIS

The damage probabilities in Table 6.1 were developed in part through use of the data assembled in Fig. 3.5, in part from the subjective probability matrices, and in part by using Eq. 2.2 to ensure that the damage probabilities are consistent with the mean damage ratios in Table 6.1.

MMI VI: The data indicate the P = 0 for all damage states except 0 and L, while the subjective matrices suggest small non-damage probabilities for damage state M. Since the assumed MDR are intermediate between the empirical and subjective results, it was decided to try keeping P = 0 except for damage states 0 and L. Then P_L may be calculated from

$$P_L \times CDR_L = MDR$$

or

$$P_L = MDR \div 0.3$$

where CDR and MDR are expressed as percentages and P_L as a decimal. For design strategy 2, for example, MDR = o.16% and

$$P_L = 0.16 \div 0.3 = 0.53 \text{ or } 53\%$$

MMI VII: Now the data suggest the following probabilities for damage states O and H:

	P_O	P_H
UBC 0	15%	4%
UBC 2	20%	1%
UBC 3	25%	0%

With regard to the arriving at P_0 for UBC 3, recall that the MDR was taken somewhat larger than the result from the San Fernando earthquake and hence the damage probabilities must be smaller in the lower damage states than those from the San Fernando earthquake. By extension of the values in the table, P_0 for the superzone S was taken as 30%.

Now the remaining P may be calculated by application of Eq. 2.2. For example, for UBC 2:

$$\Sigma \text{ CDR} \times P = \text{MDR}: \quad 0.3P_L + 5P_H + 30(0.01) = 1.9$$

$$\Sigma P = 1: \quad 0.20 + P_L + P_H + 0.01 = 1.0$$

Solving these two equations yields $P_L = 0.50$ and $P_H = 0.29$

MMI VII.5: The data suggest the following probabilities for damage states 0 and T:

	P_0	P_T
UBC 0	0	5%
UBC 2	0	1%
UBC 3	5%	0
S	10%	0

This information does not completely determine the remaining damage probabilities, and a trial-and-error approach must be adopted. For UBC 2, for example, the available results indicate that P_H might be about 10%. Now two equations can be formulated and solved for P_L and P_M. Several such trials were made.

P_H	P_M	P_L
10%	58%	31%
11%	52%	36%
12%	46%	41%

While there is not much basis for choosing among these results, those corresponding to P_H = 11% seem to give a somewhat better pattern of probabilities throughout the matrices.

<u>MMI VIII:</u> For UBC 0, the MDR is now very large. Assuming $P_0 = P_L = 0$ and $P_M = 20\%$, two equations can be solved to give P_H and $P_{T + C}$.

$$5(0.2) + 30P_H + 100\, P_{T + C} = 52$$

$$0.2 + P_H + P_{T + C} = 1$$

The result is $P_H = 0.41$ and $P_{T + C} = 0.39$. The value of $P_{T + C}$ seems rather large, but is the result of assuming a conservatively large value of MDR.

For the remaining design strategies, the various results suggest $P_0 = 0$ and

	P_L	$P_{T + C}$
UBC 2	15%	5%
UBC 3	25%	1%
S	35%	0

With this input, it is possible to solve for P_M and P_H. After making these solutions, it appeared that the results for UBC 2 would be more reasonable if the solution started with $P_L = 10\%$ and $P_{T + C} = 6\%$.

<u>MMI IX:</u> In this case, assumptions were made concerning P_L and P_M, and P_H and P_{T+C} were then computed.

<u>Breakdown of $P_{T + C}$:</u> This breakdown was guided by the few available results plus a large dose of intuition.

Appendix E.

REFERENCES

Anagnostopoulas, S.A., J.M. Roesset, and J.M. Biggs, 1972: "Non-Linear Dynamic Responses and Ductility Requirements of Building Structures Subjected to Earthquakes," M.I.T. Department Civil Engineering Research Report R72-54; Project Report No. 3.

Blume, J.A. and R.F. Munroe, 1971: "The Spectral Matrix Method of Predicting Damage from Ground Motion," Report by John A. Blume & Associates Research Division to Navada Operations Office, U.S. Atomic Energy Commission.

Cornell, C.A., 1970: "Design Seismic Inputs," Seismic Design for Nuclear Power Plants, M.I.T. Press, Cambridge, Mass. R.J. Hansen, ed.

Crumlish, J.D. and G.F. Wirth, 1967: "A Preliminary Study of Engineering Seismology Benefits," U.S. Dept. of Commerce, Coast and Geodetic Survey.

EERI/NOAA, 1973: Forthcoming report on San Fernando earthquake by Earthquake Engineering Research Institute under contract to National Oceanic and Atmospheric Administration.

Garcia, F. and J.M. Roesset, 1970: "Influence of Damping on Response Spectra, " M.I.T. Department Civil Engineering Report R70-4.

Gutenberg, B. and C. Richter, 1956: "Earthquake Magnitude, Intensity, Energy and Acceleration," Bull. Seismological Soc. America, Vol. 46, pp. 105-143.

Hou, S.N.,1968: "Earthquake Simulation Models and Their Applications," M.I.T. Department of Civil Engineering Report R68-17.

Jennings, P.C. (editor), 1971: "Engineering Features of the San Fernando Earthquake, February 9, 1971." California Institute of Technology, Pasadena.

Leslie, S.K. and J.M. Biggs, 1972: "Earthquake Code Evolution and the Effect of Seismic Design on the Cost of Buildings," M.I.T. Department Civil Engineering Research Report R72-20; Project Report No. 2.

NBS, 1971: "Engineering Aspects of the 1971 San Fernando Earthquake," U.S. Department of Commerce, National Bureau of Standards, Building Science Series 40.

Newmark, N.M. and W.J. Hall, 1972: "Procedure and Criteria for Earthquake Resistant Design," Building Practices for Disaster Mitigation, U.S. Dept. of Commerce, National Bureau of Standards, Building Science Series 46, pp. 209-236.

Seed, B., R.V. Whitman, H. Dezfulian, R. Dobry, and I.M. Idriss, 1972: "Soil Conditions and Building Damage in 1967 Caracas Earthquake," Journal of the Soil Mechanics and Foundation Division, ASCE, August, pp. 787-806.

Steinbrugge, D.V., 1970: "Earthquake Damage and Structural Performance in the United States," Earthquake Engineering, Prentice-Hall, R.L. Wiegel, ed., Chapter 9, pp. 167-226.

Steinbrugge, K.V., E.E. Schader, H.C. Bigglestone and C.A. Weers, 1971: "San Fernando Earthquake, February 9, 1971, "Pacific Fire Rating Bureau (now Insurance Services Office), San Francisco.

USCGS, 1964: Earthquake Investigations in the Western United States, 1931-1964, U.S. Dept. of Commerce, Coast and Geodetic Survey, Publication 41-2, D.S. Carder, ed.

USCGS, 1967: "Studies in Seismicity and Earthquake Damage Statistics - Part 1" U.S. Dept. of Commerce, ESSA, Coast and Geodetic Survey, 345 pp.

USCGS, 1969: "Studies in Seismicity and Earthquake Damage Statistics, 1969," U.S. Dept. of Commerce, Environmental Science Services Administration, Coast and Geodetic Survey. There is a main body with summary and recommendations, Appendix

A. by Steinbrugge, McClure and Snow describing damage evolution and insurance practice, and Appendix B by Algermissen, Stopp, Rinehart and Arnold describing seismic risk evaluation.

USGS/NOAA, 1971: "The San Fernando, California Earthquake of February 9, 1971," joint report by U.S. Geological Survey and National Oceanic and Atmospheric Administration, USGS Professional Paper 733.

Vanmarcke, E.H. and C.A. Cornell, 1972: "Seismic Risk and Design Response Spectra," ASCE Specialty Conf. on Safety and Reliability of Metal Structures; Pittsburgh, Pa.

Whitman, R.V., 1969: "Effect of Soil Conditions Upon Damage to Structures; Caracas Earthquake of 29 July 1967," Report to the Presidential Commission for Study of the Earthquake, Caracas, Venezuela.

Whitman,R.V.,S.-T. Hong, and J.W. Reed, 1973: "Damage Statistics for High-Rise Buildings in the Vicinity of the San Fernando Earthquake," Structures Publication No. 363, Dept. of Civil Engineering, M.I.T.; Project Report No. 7.

Whitman, R.V., J.W. Reed, and S.-T. Hong, 1973: "Earthquake Damage Probability Matrices," Proceedings of the Fifth World Conference on Earthquake Engineering.