OPTIMUM SEISMIC PROTECTION AND
BUILDING DAMAGE STATISTICS

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EARTHQUAKE DAMAGE TO TALL BUILDINGS

by

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PREVIOUS REPORTS

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

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

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ABSTRACT

A method for the estimation of earthquake damage to tall buildings based upon the building's response is developed. Damage to typical building components is estimated by comparing the amount of energy absorbed by a particular component to the maximum energy absorbtion capacity of that component. The method is applied to some of the buildings which were damaged during the 1971 San Fernando earthquake. It is concluded that the method for the estimation of damage can predict general trends of damage but might be in considerable error in any specific case. The method is also applied to some typical buildings as part of an optimum seismic protection study. It is concluded that changing the seismic design zone does not have a drastic effect on the total estimated damage.

PREFACE

This is the fifth report prepared under the National Science Foundation grants GK-27955 and GI-29936. This report is identical with a thesis written by Robert M. Czarnecki in partial fulfillment of the requirements for the degree of Master of Science. The research was supervised by J. M. Biggs, professor of Civil Engineering at the Massachusetts Institute of Technology. The author wishes to thank Professor Robert Whitman (MIT) and Doctor John Reed for their advice and assistance in the preparation of this report.

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LeMessurier Associates Inc. (Cambridge, Mass.) - for providing the design data for typical buildings.

Ayres, Cohen, and Hayakawa Inc. (Los Angeles) - for preparing damage studies for buildings in Los Angeles.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

A_R	Reduced bending energy capacity for shear walls
A_T	Maximum inelastic energy absorbtion capacity
A_X	Amount of inelastic energy absorbed to develop E_X
A_Y	Elastic energy absorbtion capacity
C_i	Structural damage at story i
D	Structural damage ratio
D_C	Structural damage ratio for just the columns of a braced frame
E	Modulus of elasticity
E_{SH}	Bending strain at strain hardening for steel
E_U	Ultimate bending for steel
E_X	Bending strain in column for a given lateral deflection
E_Y	Bending strain at yield for steel
E_{20}	Strain at $0.2F_c'$ for concrete
E_{50}	Strain at $0.5F_c'$ for concrete
F_c'	Compressive strength for concrete
F_Y	Yield strength for steel
F_U	Ultimate strength for steel
g	Acceleration of gravity
H	Story height
I	Modified Mercalli Intensity
I	Moment of inertia
K	Number of columns or shear walls per story

LIST OF SYMBOLS (Continued)

L	Bracing factor. Defined in Chapter 3
M	Bending moment
R	Perameter of the stress - strain diagram for steel
S	Parameter of the stress - strain diagram for steel
T	Depth of column or shear wall
v	Ground velocity in cm/sec
W	Width of column
W	Intensity of lateral load
X_i	Distance from top of building to story i
Y	Peak interstory displacement
Y_{CR}	Interstory displacement of a brick panel at which major cracking occurs
Y_U	Ultimate interstory displacement of a brick panel
Z	Slope of the falling portion of the stress - strain diagram for concrete

CHAPTER I. INTRODUCTION

1.1 Background

In 1971 the Massachusetts Institute of Technology initiated a major research effort in the field of Earthquake Engineering. In addition to topics in structural dynamics, seismic risk, and soil-structure interaction, it is the aim of this project to determine the economic and social implications of an earthquake on a major city. Specifically, it is the goal of this project to determine the optimum seismic protection for new buildings constructed in Metropolitan Boston.

The general methodology used to perform the optimization study is as follows:

1. Design a number of typical buildings for various levels of seismic protection.
2. Subject these typical buildings to artificial earthquakes of various magnitudes.
3. Determine, from the building response, the amount of earthquake-induced damage suffered by these buildings.
4. Determine the optimum trade-off between increased initial construction cost and decreased earthquake damage for various levels of seismic protection.

It is the third step of this process which is the subject of this report.

1.2. Scope

This report demonstrates the development for determining the amount of damage which might be suffered by typical buildings during earthquakes. For the purpose of this study, a typical building is defined in the following way. Its height is greater than five stories, and it has been designed according to the seismic provisions of the uniform building code. It is a flexible building whose first mode period is greater than or equal to the U.B.C. approximation: $T_1 = 0.1N$, N = number of stories. The structural system may be a steel moment-resisting frame, a concrete moment-resisting frame, a steel-braced frame, a concrete shear wall, or a combination of these. The buildings are rectangular in plan, and the plan dimensions are not greatly different from those used in real buildings. It is expected that the typical buildings may have any number of nonstructural elements which are often found in real apartment and office buildings.

In the development of a method to compute earthquake damage, it was desired that the method could be applied to a building without a detailed knowledge of exactly how the building is constructed or exactly how the building responds to an earthquake. Due to these constraints, it is necessary to limit the parameters which might be used as input to a method for the computation of earthquake damage. It is expected that a user of the suggested method for the computation of earthquake damage have a knowledge of the kind of structural system to be tested, the geometry of the structural system, the materials of construction, the kinds of nonstructural systems used in the buildings, and the average or maximum response of the building to a particular earthquake. It is with these parameters that the mathematical models for earthquake damage are formulated.

CHAPTER 2. NATURE OF THE PROBLEM

2.1 General Comments on Earthquakes and the Effects on Buildings

From the structural engineering standpoint, the occurrence of an earthquake means that the structures in the vicinity of the causative fault will experience dynamic forces in excess of those present under normal service load conditions. These forces result from a transient, cyclical motion of the base of the structure which will undoubtedly vary in magnitude, frequency and direction during the length of the shaking. The details of this base motion are related in a very complex way to the nature of the faulting and the geological features between the epicenter of the earthquake and the recording station. Due to this interaction between the stress waves generated by the faulting and the geology of the region, it is impossible to predict with any great certainty the details of ground motion which might be observed at a given site during a given earthquake. In fact, there have been cases during actual earthquakes when ground motions recorded at two sites separated by only city blocks have been quite dissimilar.

Faced with the uncertainties inherent to the input, one might think that it would be very difficult to design a building which would successfully resist earthquake loading. However, engineers have been able to design buildings which have withstood very severe shaking. It is true that buildings which were severely shaken by earthquakes have experienced considerable damage. However, in most cases, this damage was largely to nonstructural elements and not to the structural system. This is partially due to conservatism in the building codes which govern earthquake resistant design, but, more importantly, the successful design of a building for earthquake effects depends on the engineer's understanding of the way the structure responds to dynamic loading. The general nature of this response is that the base of the structure undergoes a series of distortions which are felt to some extent throughout the entire structural system. It is these distortions which create additional axial loads, bending moments and shear forces which must be resisted by individual members. The available resistance of individual members is also subject to some uncertainty. For example, the amount of dead and live load carried by the structure at the time of the earthquake, possible local weakness due to understrength material, and the approximations made in determining the dynamic characteristics of the building may cause individual members to behave quite differently than the design engineer originally envisioned.

The above-mentioned sources of uncertainty are just a few reasons why some people choose to treat the earthquake problem in the context of probability. Although the work presented in the subsequent chapters does not use probabilistic methods, it is intended that the method developed be understood as an approximate technique to compute damage. Any results obtained through utilization of this method are better described as mean values rather than deterministic quantities.

2.2 Damage Considerations in the Design of Buildings

The Uniform Building Code, which governs earthquake-resistant design in the United States, adopts a general philosophy which does have some implications on the amount of damage a building might suffer during an earthquake.* A loose interpretation of this philosophy would say that the structure should respond elastically during relatively minor earthquakes, and during major earthquakes the structure should exhibit considerable ductility and not fail. Elastic response implies that damage to the structure would be minor during small earthquakes. However, ductility requirements during large earthquakes will cause considerable inelastic action by the structure, and the damage to the structure may be so severe that it cannot be repaired. Aside from this general philosophy the code contains no regulation concerning permissible values of damage during earthquakes or how values of damage might be computed. It is therefore not surprising that in the design of buildings a computation of expected earthquake damage is not included. Certainly, steps are taken during the design process to limit the expected damage. Careful attention to structural details, attempts to isolate brittle elements such as partitions and glass panels, and other techniques have become more or less standard practice in earthquake-

* The intent of the seismic provisions of the Uniform Building Code is to produce structures which will not endanger human lives during an earthquake. The implications of the code on damage are understood through the Code's philosophy on the protection of human life.

resistant design. Many times it is the attention to these details rather than just the strength of the structural frame which determines how much damage a building will receive as a result of an earthquake. However, even when these techniques are well planned and carefully implemented, the design engineer will have only a vague idea of the amount of damage a particular building might experience during an earthquake.

For the design of typical buildings the considerations given to expected damage as described above are probably adequate. This statement is substantiated by the fact that the majority of buildings designed by this procedure suffered only minor damage during the 1971 San Frenando earthquake. However, there may be circumstances in which the amount of expected damage needs to be computed with greater accuracy. For example, it is conceivable that an engineer might find it desirable to provide seismic protection for a particular building in excess of that required by the Code. Undoubtedly, this extra protection will increase the cost of the building, but it will, hopefully, decrease the amount of damage which could be expected during an earthquake. If it were necessary to find the optimum balance point between increased initial cost and decreased earthquake damage, one would need a more refined estimate of earthquake damage than is obtained through the use of the building code and current design procedures.

The method developed in Chapter 3 for estimating earthquake damage arises from an optimization study similar to the one described above. Quite early in the study, it was realized that in the wake of a major earthquake there are associated costs which may be much larger than the physical damage to buildings. These associated costs are quite difficult to evaluate, since they include many intangible items such as loss of human life. It should be emphasized that the method developed in chapter 3 accounts only for physical damage to buildings, and, therefore, it is only a part of the total picture.

CHAPTER 3. CORRELATION OF BUILDING RESPONSE AND DAMAGE

3.1 Introduction

This chapter presents the development of a mathematical model for estimating earthquake damage to a building based upon the building's dynamic response. In the process of investigating the earthquake damage problem, many approaches were considered and subsequently discarded in favor of the method developed in section 3.4. For the sake of brevity only those concepts which directly influenced the final models are presented.

3.2 An Existing Procedure for the Estimation of Earthquake Damage

The Modified Mercalli Intensity Scale, shown in Fig. 3.1, was developed by Mercalli in 1902 and later updated by Wood and Neumann. This scale gives a brief description of the kind of damage that will be associated with an earthquake having a given intensity number. After an earthquake, trained observers throughout the region of shaking give their estimate of the modified Mercalli intensity felt in their particular area. By collecting these estimates from a number of earthquakes it has become possible to correlate Modified Mercalli Intensity with ground motion. One correlation suggested by Neumann is[1]:

$$I \text{ (Modified Mercalli Intensity)} = \frac{\text{Log } 14v}{\text{Log } 2}$$

where v is the ground velocity in cm/sec. Use of this relationship along with the Mercalli scale will give a gross estimate of how much damage is associated with a particular level of ground motion.

I	Not felt except by a very few under especially favorable circumstances.
II	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.
IV	During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
V	Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles and other tall objects sometimes noticed. Pendulum clocks may stop.
VI	Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
VII	Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.
VIII	Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motor cars disturbed.
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations, ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.

FIGURE 3.1 ABRIDGED MODIFIED MERCALLI INTENSITY SCALE

Unfortunately, the damage descriptions given in the Mercalli scale are far too general for use in this study; however, relationships such as the one suggested by Neumann do have an interesting implication. These relationships suggest that it might be possible to correlate gross damage statistics to average values of building response. It was this possibility which lead to study of the damage to buildings during past earthquakes.

3.3 Historical Earthquake Investigation

3.3.1 Purpose of Study

The general purpose of this study was to gather information about the performance of buildings during past earthquakes. It was hoped that by determining gross damage statistics and average motion statistics for many buildings during several earthquakes, that a meaningful correlation between damage and motion could be achieved. In addition, by looking at several buildings in great detail, it was hoped thatthe key parameters for computing damage could be established. These parameters would then be used in a more precise formulation of the earthquake damage problem.

3.3.2 San Fernando Earthquake

This study was begun less than a year after the 1971 San Fernando earthquake. Since this information was so recent, and hopefully undistorted, it was decided to investigate the performance of buildings during this earthquake first. The Uniform Building Code in section 2314 requires that all buildings constructed in seismic zone 3, having more than 6 stories and 60,000 square feet, or more than 10 stories, be equipped with strong motion instruments (California is in seismic zone 3). Most of these instruments functioned properly during the earthquake, and as a result numerous strong motion records were collected. These records have been digitized and used as input for the dynamic analysis for many buildings (2,3,4,5,6). Dynamic analyses have been performed for many buildings in Los Angeles. These buildings range in height from six to forty-two stories. All buildings that were analyzed were constructed within the last twenty years and were designed according to the Uniform Building Code.

The dynamic analysis of these buildings was performed by assuming a lumped mass model with estimated elastic spring constants and viscous damping coefficients. The time history of ground motion recorded during the earthquake at the base of each building was used as input. An elastic dynamic analysis was performed on the assumed model using the input motion, and the time history of response of each floor was computed. For the floors on which the time history of response had been recorded during the earthquake, the computed response was compared to the recorded response. The elastic dynamic analysis was repeated with adjusted spring constants and damping coefficients until there was good agreement between recorded and computed responses. The results of these analyses were reported to M.I.T. either in the form of complete time histories of floor acceleration and interstory displacement at each floor, or in the form of a summary of peak floor accelerations and interstory displacements at each floor.

The collection of damage statistics is a more tedious and uncertain process. Ayres, Cohen and Hayakawa, a Los Angeles consulting firm, was retained to perform detailed damage studies for 18 buildings in the Los Angeles Area. The kind of information that was required for the damage motion correlation was the initial construction cost, the total repair cost, the repair cost for various building components, and a floor-by-floor breakdown of the cost. Unfortunately, Ayres, Cohen and Hayakawa encountered considerable difficulty in the collection of this data. Some building owners would not cooperate because they feared that divulging the amount of damage suffered by their building would impair its rentability. When owners would cooperate they frequently did not have exact information desired. For example, some owners could provide exact information about the total repair cost, but could only estimate the amount of money spent to repair the various building components. In addition, there were only two cases in which the building owner was able to give more than just a vague description of how the damage was distributed throughout the building.

Due to the uncertainties in the data collected by Ayres, Cohen and Hayakawa, it became necessary to verify the reported damage with less precise descriptions of damage given in other sources (2,3,4, 5,6). The primary function of these sources was to report the results of dynamic analyses performed for individual buildings; however, these reports typically contained a good description of the earthquake damage. By combining Ayres' damage reports and the descriptions given in the other sources mentioned, it became possible to verify Ayres' data and to determine a crude estimate of how damage was distributed throughout the building. In obtaining this distribution it was first necessary to read a description of how the damage varied from floor to floor and then attempt to assign a portion of the total repair cost to each floor. An example of this procedure is shown below.

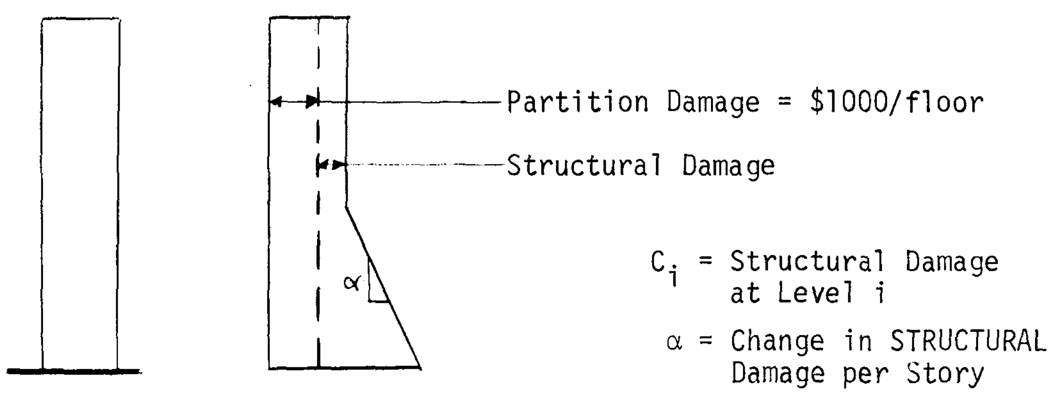
DISTRIBUTION OF DAMAGE BUILDING X

Ayres Report: Structural Damage \$32,000; Partition Damge, \$12,000.

From Written Summary of Damage:

1) Partition damage assumed uniform throughout building
2) Structural damage assumed uniform for floors 6-12 and varies linearly with height for floors 1-6.

Damage Model



$$7C_6 + C_5 + C_4 + C_3 + C_2 + C_1 = 32{,}000$$

$$7C_6 + C_6 + \alpha + C_6 + 2\alpha + C_6 + 3\alpha + C_6 + 4\alpha + C_6 + 5\alpha = 32{,}000; \qquad 12\,C_6 + 15\alpha = 32{,}000.$$

From the damage description reported at story 6, Assume $C_6 = 2000$.

$$24{,}000 + 15\alpha = 32{,}000$$

$$\alpha = \$533/\text{floor}$$

DAMAGE SUMMARY BUILDING X

	Component Damage		
STORY	PARTITION	STRUCTURE	TOTAL DAMAGE
12	$1000	$2000	$3000
11	1000	2000	3000
10	1000	2000	3000
9	1000	2000	3000
8	1000	2000	3000
7	1000	2000	3000
6	1000	2000	3000
5	1000	2533	3533
4	1000	3066	4066
3	1000	3599	4599
2	1000	4132	5132
1	1000	4665	5665

As demonstrated in this example, any attempt to convert total damage values to discrete values for every story introduces additional uncertainty. For each building, it was also necessary to determine the initial value of each story. This number was usually obtained by dividing the initial construction cost by the total number of stories.

As the process of data collection continued, important trends of damage were noticed. For example, Figs. 3.2, 3.3, 3.4, and 3.5

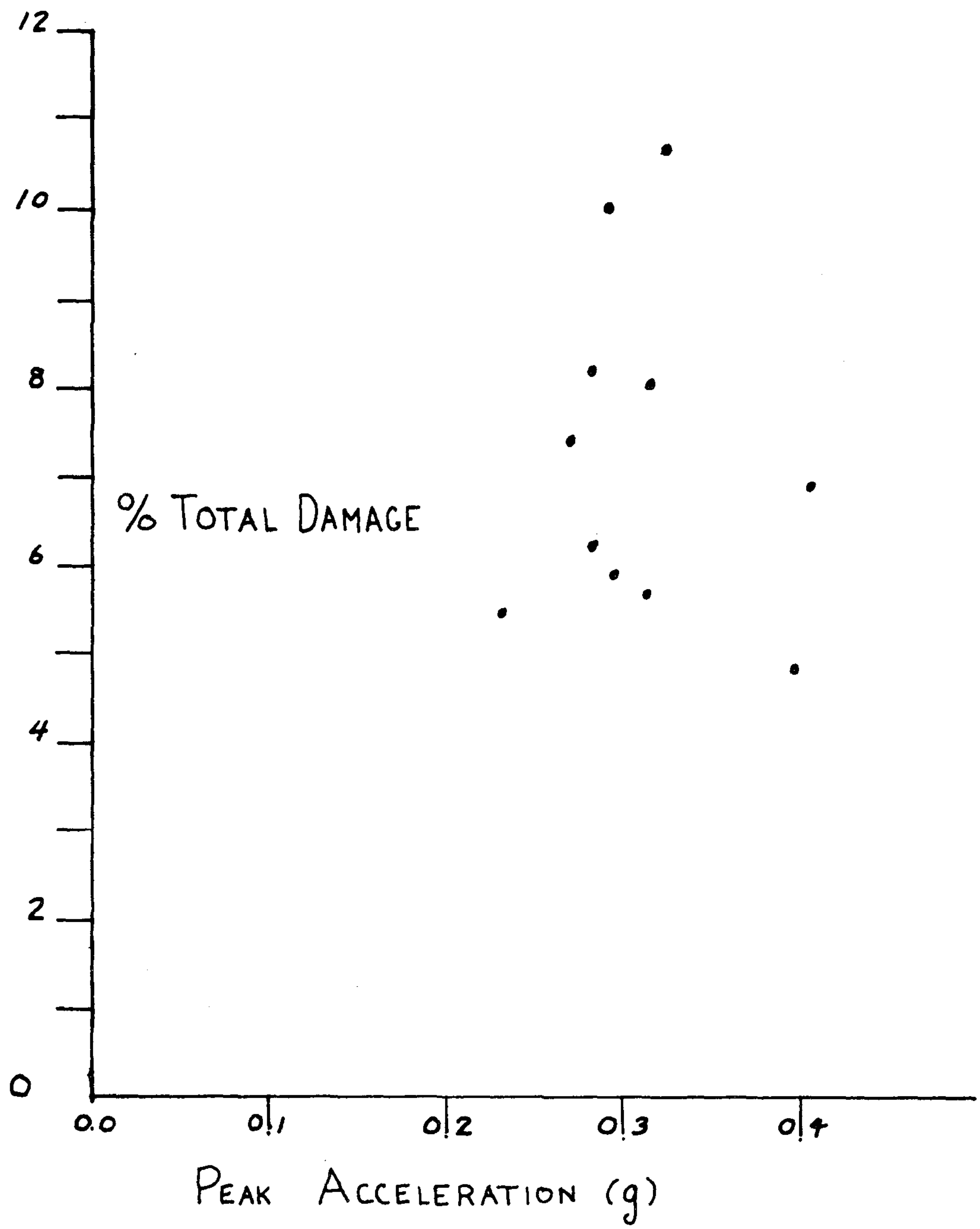


FIGURE 3.2 % TOTAL DAMAGE vs. PEAK ACCELERATION

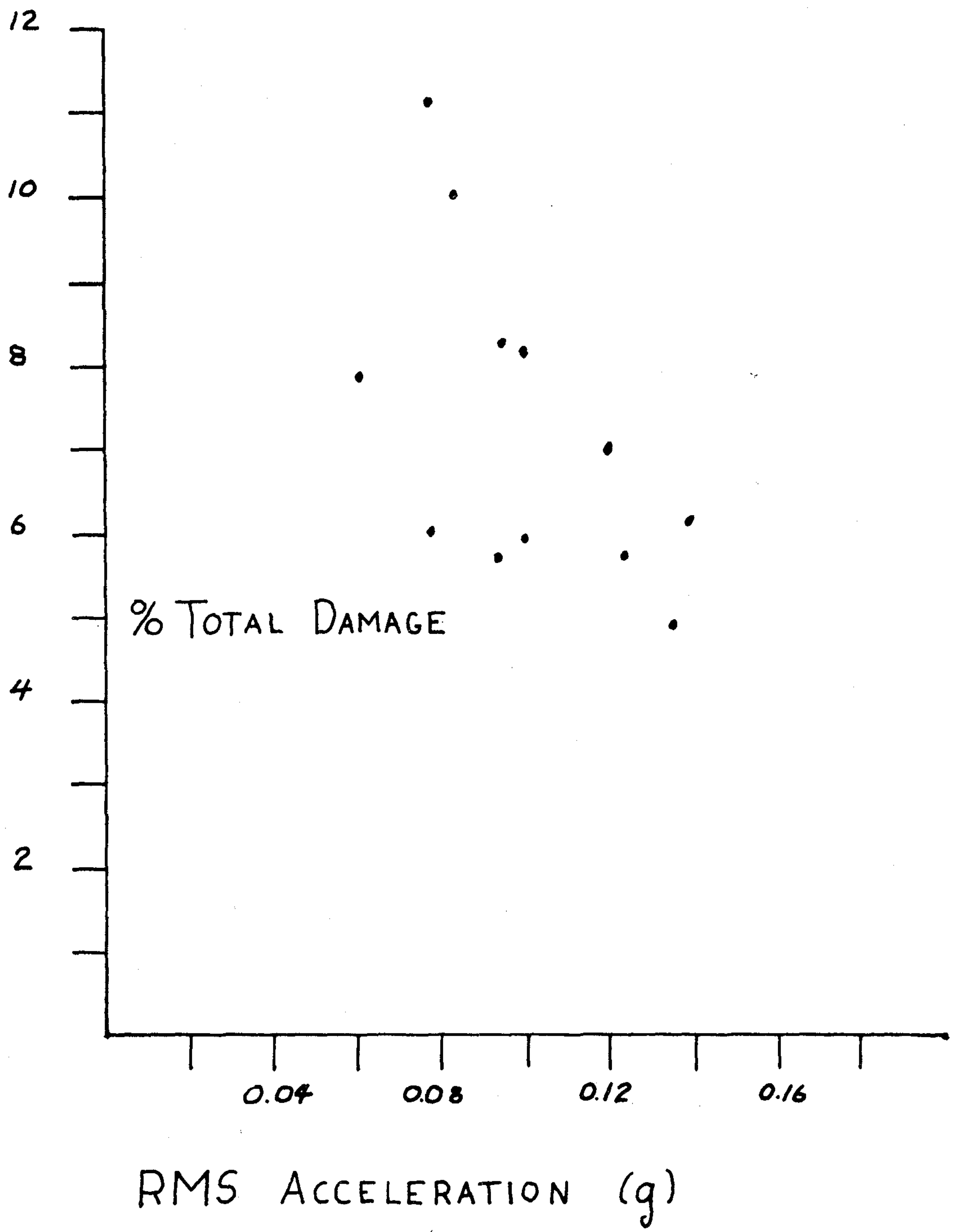


FIGURE 3.3 % TOTAL DAMAGE vs. RMS ACCELERATION

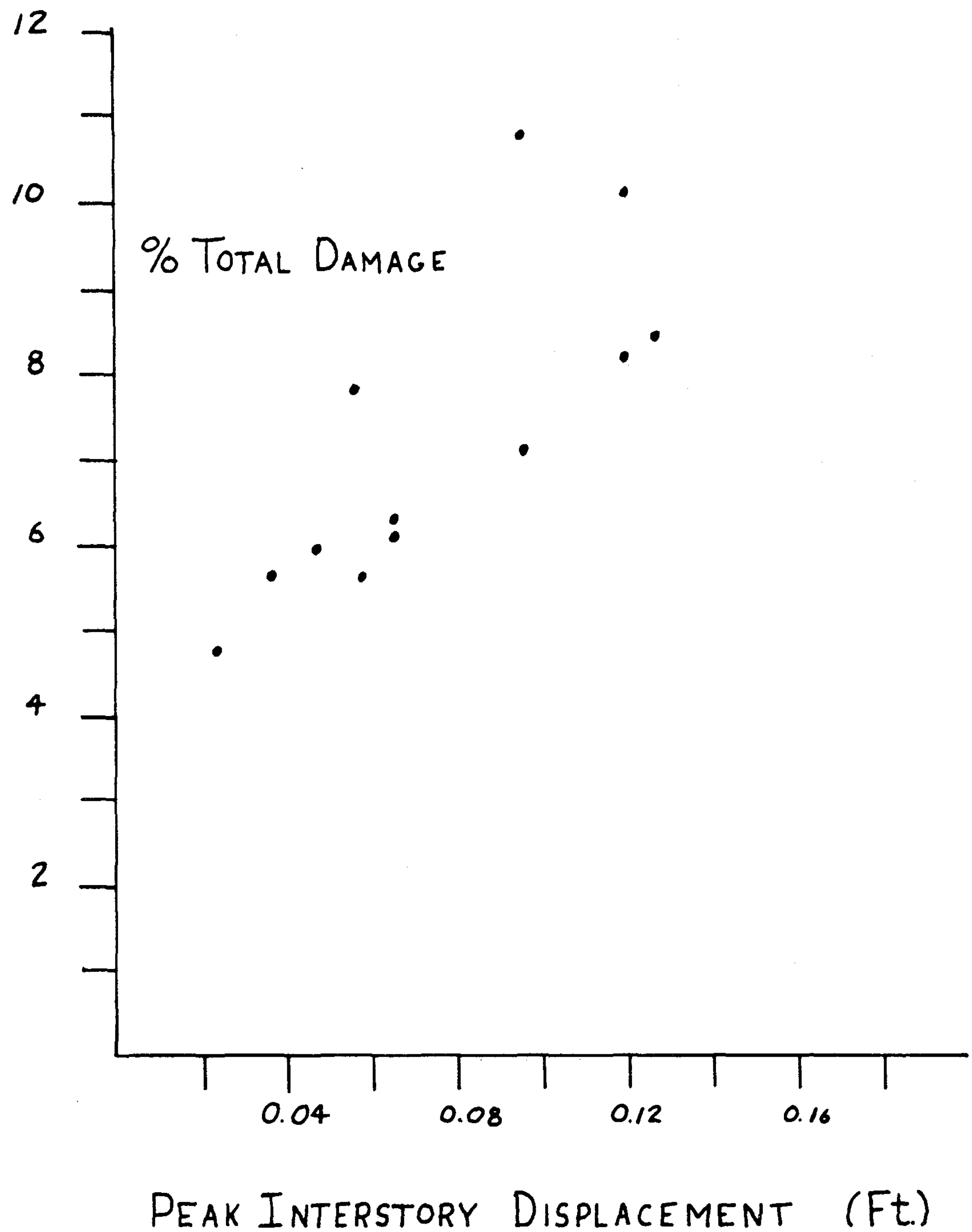


FIGURE 3.4 % TOTAL DAMAGE vs. PEAK INTERSTORY DISPLACEMENT

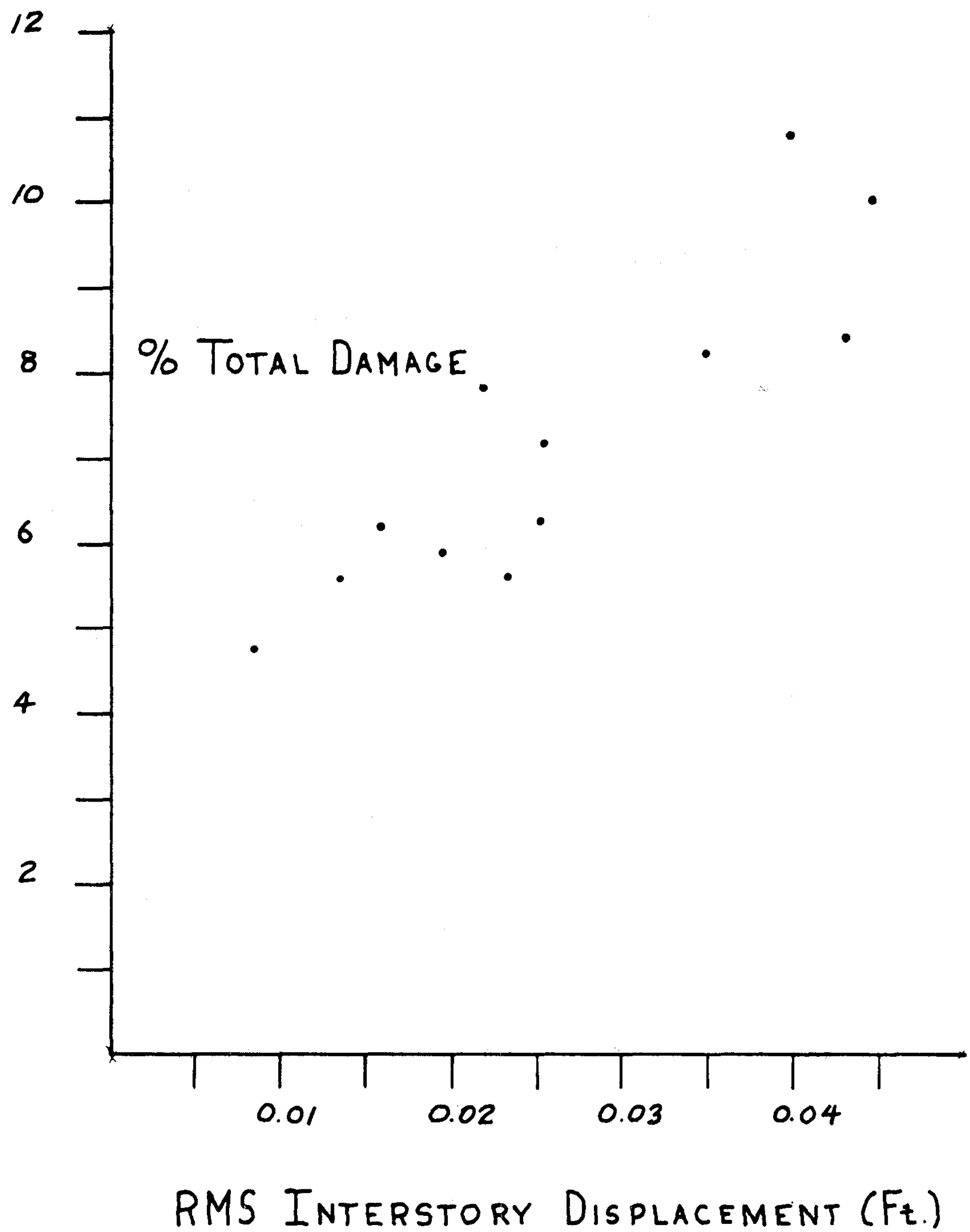


FIGURE 3.5 % TOTAL DAMAGE vs. RMS INTERSTORY DISPLACEMENTS

show the results of a preliminary study to determine which summary of building response would best correlate with damage. For each plot the vertical scale is the total repair cost per story, divided by the initial value of that story. The horizontal scale is peak acceleration (fig. 3.2), root mean square acceleration (fig. 3.3), peak interstory displacement (fig. 3.4), and root mean square industry displacement (fig. 3.5). The data from which these plots are constructed comes from two buildings in Los Angeles in which the earthquake damage and the buildings' response are very well documented (3,4).

Figures 3.2 and 3.3 show that the total damage on a given floor is almost completely uncorrelated with peak acceleration or root mean square acceleration. On the other hand, figs. 3.4 and 3.5 show a definite trend of increasing total damage with increasing peak interstory displacement and root mean square interstory displacement. On the basis of only 2 buildings, it would be presumptuous to state that interstory displacement was the only indicator necessary to predict earthquake damage. However, as more information on more buildings was gathered, it became apparent that the best prediction of earthquake damage could be made on the basis of interstory displacement. This conclusion is drawn from the observation that, when earthquake damage occurs, the building components which are most heavily damaged are those which are orientated vertically. These components include columns, shear walls and partitions. When horizontally orientated components were heavily damaged it was often found that there was some error in the design or installation of these components. For example, where beam damage was noticed it was often found that adequate compression steel was not provided at the mid-span sections (7). Where damage to machinery was noticed it usually resulted from failure of the vibration mounts which then allowed the equipment to be tossed about (8). It was also found, and is shown in Table 3.1, that for 9 buildings studied in detail, damage to vertical elements exceeds damage to horizontal elements. Furthermore, it can be envisioned that lateral distortions would be the key cause of damage to vertical elements. The only justification for this

TABLE 3.1

COMPARISON OF DAMAGE TO HORIZONTAL AND VERTICAL ELEMENTS

Building Number	Vertical Damage	Horizontal Damage
1	$100,100	2,060
2	88,190	70
3	11,000	0
4	110,000	20,000
5	35,250	7,300
6	1,100	0
7	318,000	31,500
8	2,000	0
9	79,000	11,000

statement is a conceptual understanding that when two stories of a building are displaced by different amounts, the elements which run vertically between the stories undergo distortions, strain, and cracking, and therefore require repair. Finally, it is realized that RMS interstory displacement is undoubtedly a better indicator of the total effect of an earthquake on a building. Unfortunately, the RMS values for several of the nine buildings mentioned above were not available, and since figs. 3.4 and 3.5 show that there is not a great deal of difference between correlation attempts with peak and RMS values, it was decided to base a damage motion correlation on peak interstory displacement.

To summarize, the general trends discovered in the early stages of data collection are: a) The largest portion of earthquake damage to buildings is contributed by vertical elements, b) interstory displacements are the best indicator of earthquake damage, and c) more buildings could be included in the study if the correlation were attempted with peak rather than RMS interstory displacements.

With these general trends established, the correlation study proceeded by constructing a plot similar to fig. 3.4 for six of the nine buildings in Table 3.1. (Three buildings were not included, because it was impossible to obtain even an approximate idea of the distribution of damage on each story). In fig. 3.6, the vertical scale is the ratio of the damage to a story to the value of that story. The horizontal scale is peak interstory displacement. The general trend of the data represented by fig. 3.6 is quite clear; however, the scatter is too great to suggest any precise correlation. In the range of peak interstory displacement less than 0.03 feet and percent damage less than 2%, there is a cluster of data points

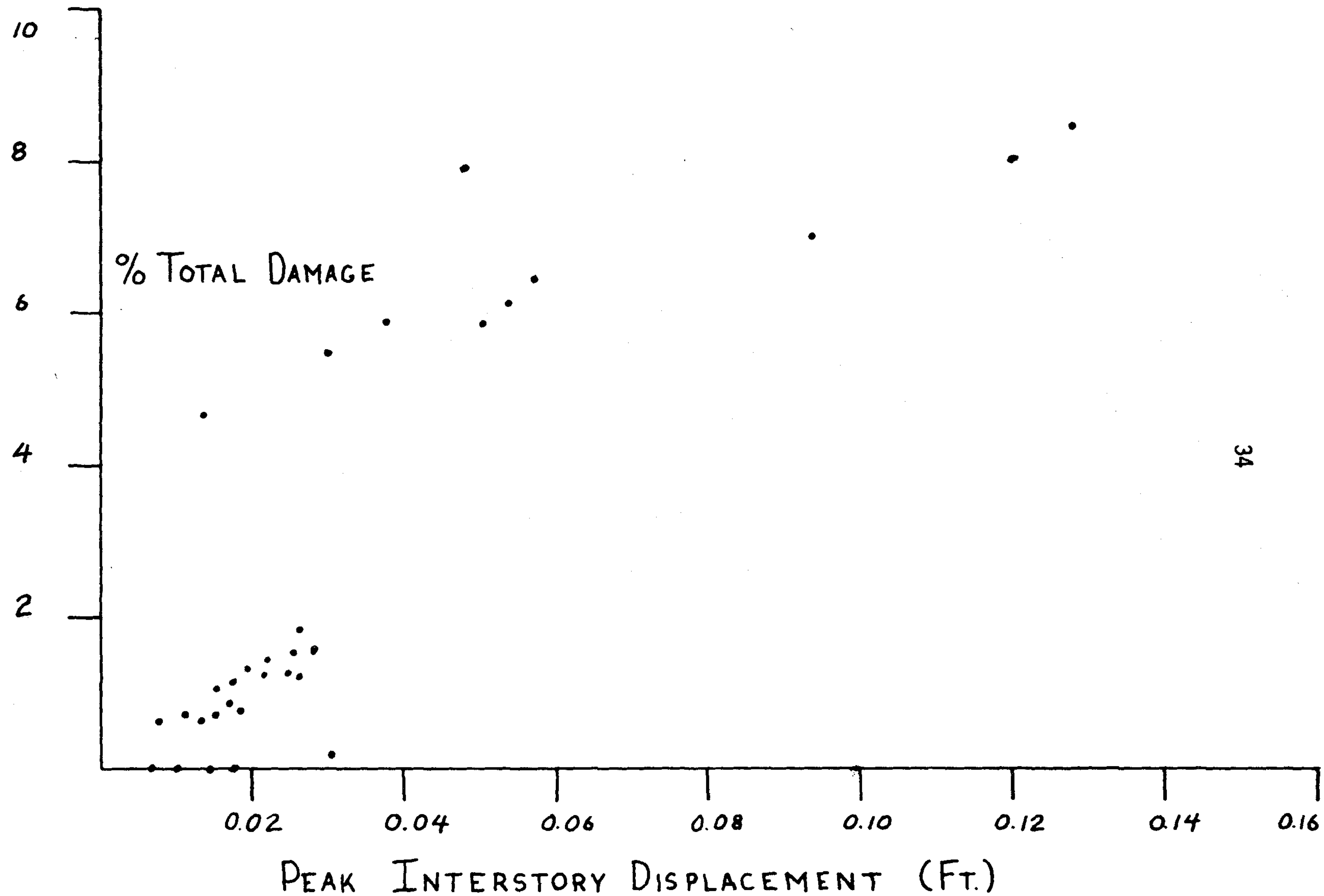


FIGURE 3.6 % TOTAL DAMAGE vs. PEAK INTERSTORY DISPLACEMENT

which seem very strongly correlated. However, data points on the rest of the plot do not follow the same pattern as those in the neighborhood of the origin. A careful investigation of the buildings which contributed data points in the range of 2 to 12 percent was conducted. It was found that these buildings were not drastically different from those in the lower damage categories. It was therefore concluded that the scatter in fig. 3.6 is a result of subtle differences in the design and construction of buildings. These differences could not be accounted for with the available information. It is also important to note that the maximum damage reported in fig. 3.6 is approximately 11 percent. Over a broader range of damage (0 to 100 percent),the scatter in fig. 3.6 might appear to be insignificant. It was hoped that this broader range of damage could be obtained through the study of the Caracas earthquake.

3.3.3 Caracas Earthquake

The 1967 earthquake caused considerable damage to the buildings in the Venezuelan capitol. More importantly, the damage is distributed over a range of values from zero, for some buildings in downtown Caracas, to 100 percent (collapse) for some buildings in the Los Palos Grande district. Reference 7 gives a good description of the damage to buildings in Venezuela, with particular attention given to the Los Palos Grande district. Although numerical values are not given in reference 7, it is not difficult to obtain an approximate percentage of damage by reading the published descriptions.

Since there were no strong motion records obtained from the 1967

Caracas earthquake, and therefore no dynamic analyses made for any of the buildings shaken by this earthquake, it became necessary to use an approximate method to obtain the dynamic response of the damaged buildings. Reference 9 gives a best estimate of the ground response spectra for locations in Caracas over different depths of soil. These spectra represent average values computed from four different time-history inputs. Three of the input records were actually recorded during Californian earthquakes, while the fourth was an artificial record, generated by the application of random vibration theory. The procedure used to compute these response spectra was to apply the input record at bedrock and to filter the response through the soil by the means of one-dimensional soil amplification theory. This procedure was repeated for the different depths of soil found in the Caracas valley.

Reference 10 gives the first mode period and the principal type of behavior (shear beam or bending beam) for 39 buildings for which the damage was reported in reference 7. All of the buildings reported in reference 10 are recently constructed concrete frame and shear wall buildings. The height of these buildings ranged from five to twenty-five stories. All buildings had been designed according to the VENEZUELA STANDARD FOR BUILDING DESIGN. The provisions for seismic forces given in the Venezuelan Regulations are approximately equivalent to the Uniform Building Code Zone 2 Regulations. To compute the dynamic response for these buildings it is necessary to make the following assumptions:

a) The shape of the first mode of response is only dependent upon the type of behavior given in reference 10 and is:

  (i) straight line for shear beam behavior
  (ii) parabola for bending beam behavior.

b) The buildings have uniformly distributed mass and stiffness.

c) Only the first mode participates in the response.

The first mode period combined with the stated assumptions and the response spectra given in reference 9 enables the computation of the maximum displacement and acceleration at the top of the building. These values can then be converted to the average drift (inches per foot of height) and the base shear.

An attempt to correlate percent damage to drift for the building in Caracas is shown in fig. 3.7. Figs. 3.8 and 3.9 show an attempt to correlate structural damage with the ratio of the actual base shear to the design base shear (C_a/C_d), and nonstructural damage to drift. The data plotted in Fig. 3.7 demonstrates the same general trend as the San Fernando data shown in fig. 3.6. Fig. 3.7 also shows that broadening the range of damage data increased rather than decreased the scatter. An attempt to divide the damage into its structural and nonstructural components as shown in figs. 3.8 and 3.9 also did not decrease the scatter.

Various attempts were made at combining the information collected from the San Fernando and Caracas earthquakes. In general, the result of this is to produce more scatter than was observed when looking at each earthquake individually. This result is not presented

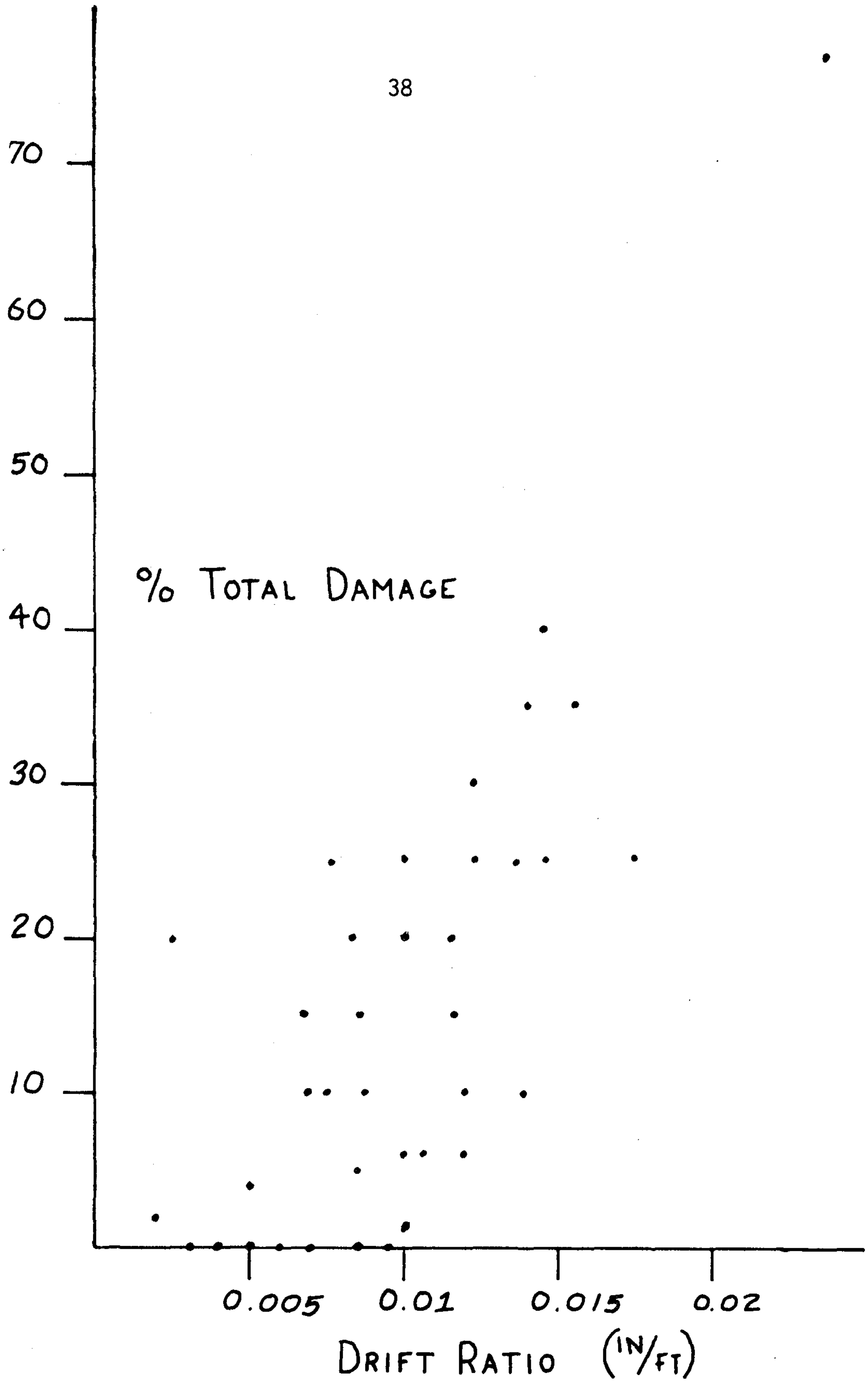


FIGURE 3.7 % TOTAL DAMAGE vs. STORY DRIFT

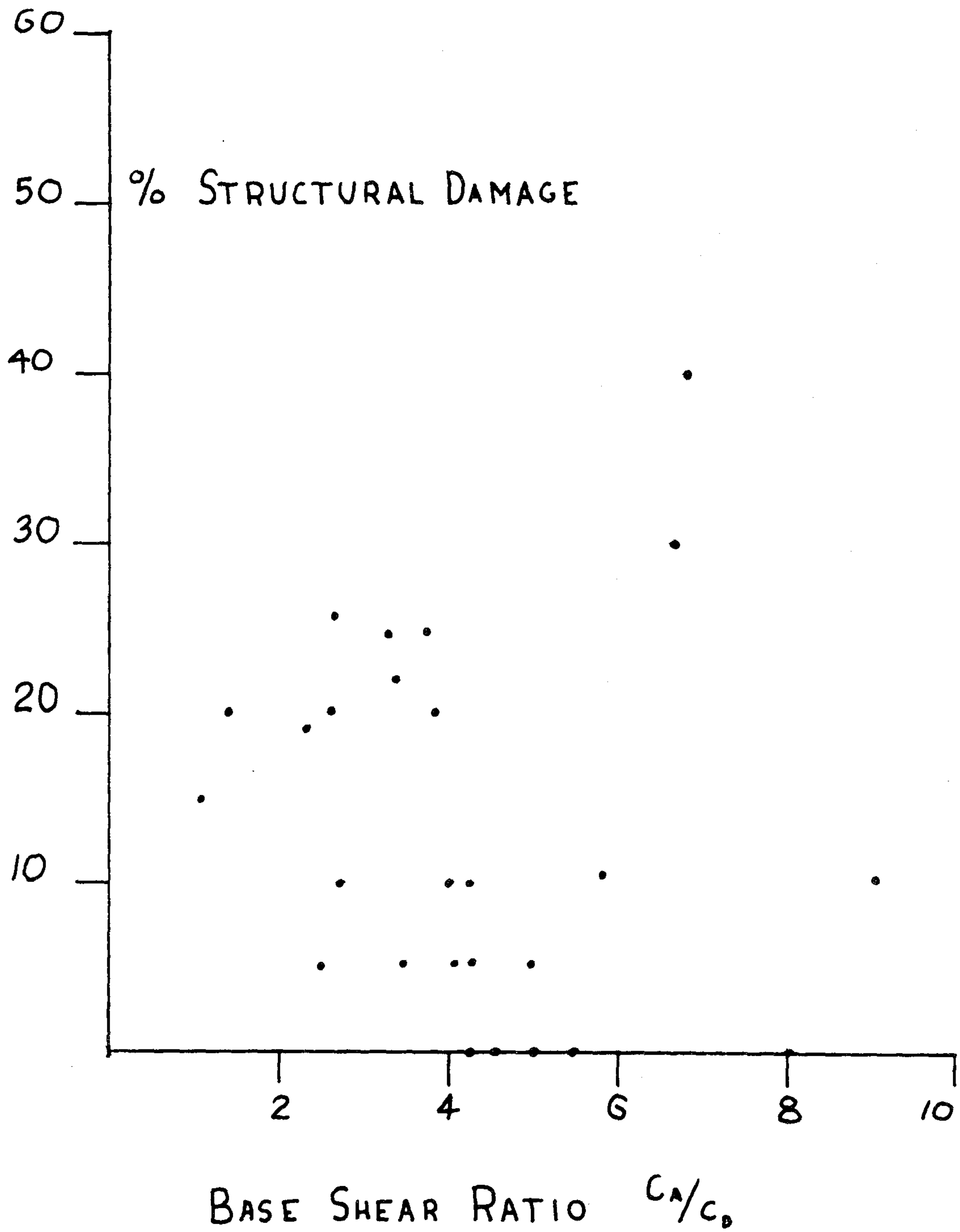


FIGURE 3.8 % STRUCTURAL DAMAGE vs. BASE SHEAR RATIO

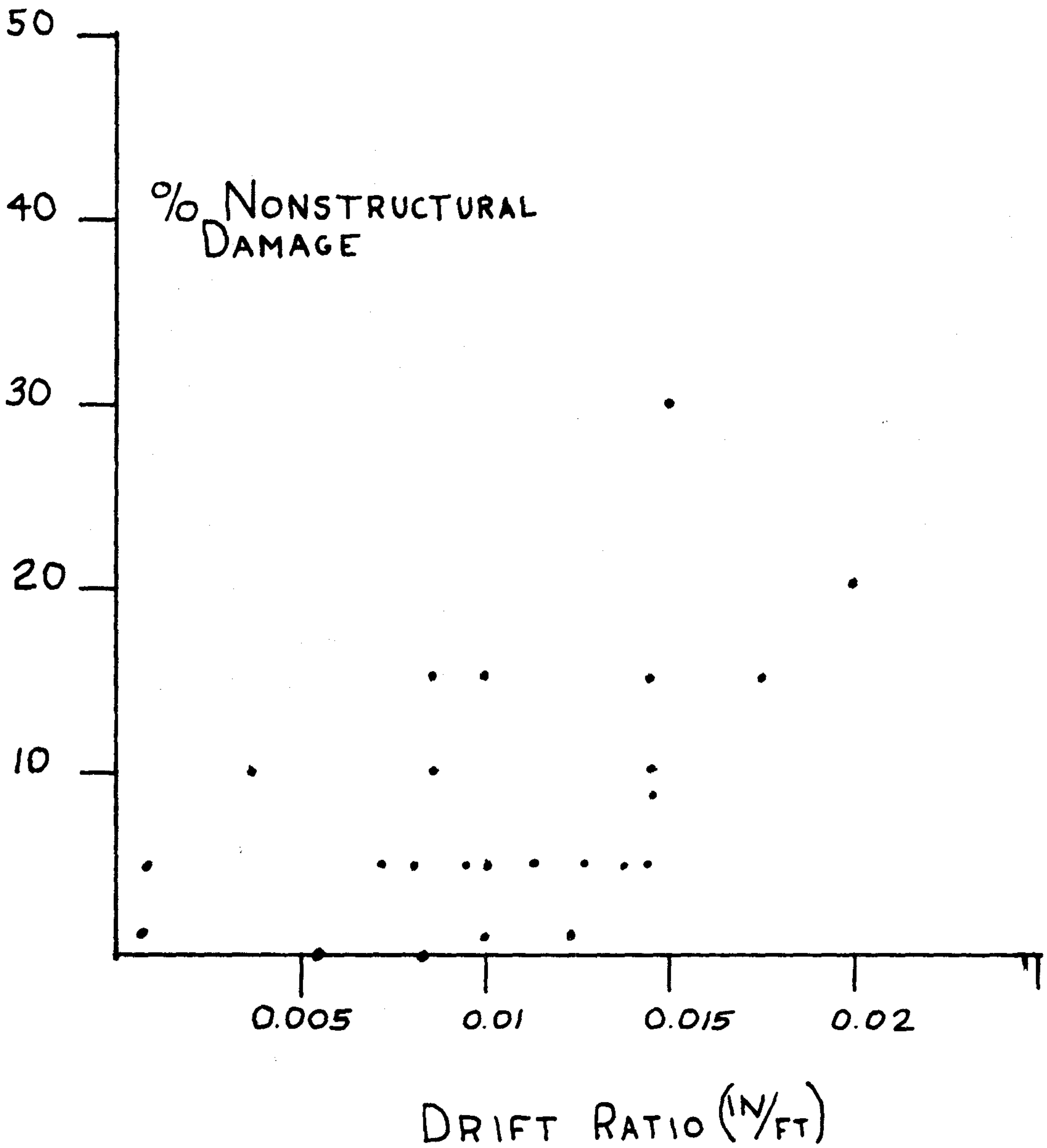


FIGURE 3.9 % NONSTRUCTURAL DAMAGE vs. DRIFT RATIO

here because it would only serve to restate conclusions already drawn from the individual study of these earthquakes.

3.3.4 Conclusions from the Investigation of Historical Earthquakes

A review of figs. 3.6 to 3.9 shows that it is impossible to predict a unique value of damage that would be associated with a particular value of interstory displacement, drift, or base shear. From these figures it is also very difficult to determine a mean value of damage and a reasonable standard deviation for a particular level of response. In fact, if one wishes to predict the amount of damage that might be associated with a particular level of response, the existing correlation as given in section 3.2 would give results as accurate as those shown in figs. 3.6 to 3.9.

Although it appears to be impossible to arrive at an acceptable correlation of earthquake damage to building response through the study of historical earthquakes, this exercise served to identify some key parameters relating to earthquake damage. These parameters are certain factors which appear to be related to damage; however, the nature of this possible relationship could not be determined. The three most important factors and why they were selected are given below. It is intended that these conclusions apply only to the buildings included in this study (see section 3.3.2 for San Fernando and section 3.3.3 for Caracas).

a) Structural system—For the buildings studied, structural damage to shear wall buildings was greater than structural damage to frame buildings.

b) Material of construction—For the buildings studied, the damage to the structural frame of concrete buildings was greater than the damage to the structural frame of steel buildings.

c) Nature of response—There is a general trend of increasing damage with increasing interstory displacements.

Any mathematical model to predict earthquake damage should include these factors.

The study of historical earthquakes also enabled the identification of the major sources of earthquake damage. As stated in section 3.3.2, the most significant contributions to earthquake damage came from vertical elements. Different kinds of vertical elements can be identified. For example, within the structural system, the damage to columns is significantly different from the damage to shear walls. Among nonstructural items, different damage patterns were observed in drywall partitions, concrete block walls, brick masonry walls, and exterior glazing. Therefore, in a mathematical model for damage it would be necessary to compute the damage to differenent elements through different models.

It is significant to note that during the San Fernando earthquake many buildings experienced substantial damage to nonstructural elements while the structural system remained undamaged. This type of damage pattern might be expected when the motions of the building are not large enough to cause individual structural members to experience any substantial yielding (steel structures) or cracking (concrete structure). However, even if the structural frame is not over-stressed, relatively weak and rigid nonstructural walls may experience deformations which are large enough to cause considerable cracking. Therefore, a mathematical model for damage should account for the possibility of large nonstructural damage even when the structural frame is only slightly damaged.

3.4 Mathematical Models for Earthquake Damage

3.4.1 Structural Damage

In the development of a method for the estimation of earthquake damage to buildings, it is desirable to include the influence of the factors mentioned in section 3.3.4. A review of current literature in the field of earthquake engineering revealed that the damage problem had not generally been treated in a comprehensive manner. Therefore, there is no existing body of knowledge upon which such an approach could be based.

When a building is shaken by an earthquake, there are energy demands upon the structure which must be satisfied, if the structure is to remain standing (11). Ground motion, in the form of stress waves traveling through the earth, possess a certain amount of energy. When a building is excited by ground motion, individual particles throughout the building are set in motion; therefore, the ground motion imparts a certain amount of its energy to the building. If the building is to remain standing after the earthquake, it must absorb this energy in some way.

Design engineers often think of the structural system as the only device in a building capable of absorbing energy. This is not quite true, since nonstructural walls also have the capacity to absorb energy; however, neglecting these elements is usually a reasonable approximation. When the structure is behaving as a device to absorb energy, there are two modes in which the required capacity can be developed. Elastic energy is absorbed when the motions of the structure are such that the stresses and strains developed in the individual members of the structural system are less than the yield point of the material. If, as a result of an input motion, the structure is caused to absorb energy in the elastic mode only, then the input motion will have no net effect on the condition of the structure (neglecting the possibility of fatigue effects). When the residual vibrations of the structure caused by the input motion have stopped, an inspection of the structural frame would reveal no permanent displacements. Levels of internal stress and strain in individual members would be the same as prior to the earthquake. If only elastic energy were absorbed by the structure during an earthquake, then the condition of the structural frame after the event would be identical to its condition before the event. Therefore, there would be no structural damage.

If the stresses and strains developed in individual members due to an earthquake are greater than the yield point of the material, then inelastic energy is absorbed. In this case, a post-earthquake inspection of the structural frame would reveal permanent displacements and levels of stress and strain in excess of those present prior to the earthquake. If the magnitude of these permanent displacements is very small, then they may go unnoticed unless careful measurements are made. However, if these permanent displacements are substantial, it would be necessary to restore the structural frame to its original condition, and this process represents a repair cost to the structure.

When a structure is excited by dynamic loading, a considerable amount of energy may be absorbed through damping. During an earthquake the structure is subjected to many cycles of repeated motions; therefore, the amount of energy which must be absorbed is constantly changing. For the purpose of developing a mathematical model for damage, it is assumed that the damage caused by the repetitive dynamic loading can be adequately modeled by single static movement to the position of maximum deformation. For a static application of the maximum deformation, the energy absorbed through damping can be neglected. For this reason, damping does not appear in the mathematical model for damage.

The two modes of energy absorption described above can be conveniently represented as the area enclosed under a stress-strain diagram. Fig. 3.10 is an idelization of the stress-strain diagram for steel. In this figure, A_i, the area under the curve up to the strain at yield E_y, is the elastic energy absorption capacity (lb-in/in^3), and A_T, the area under the curve from E_y to the strain at failure,

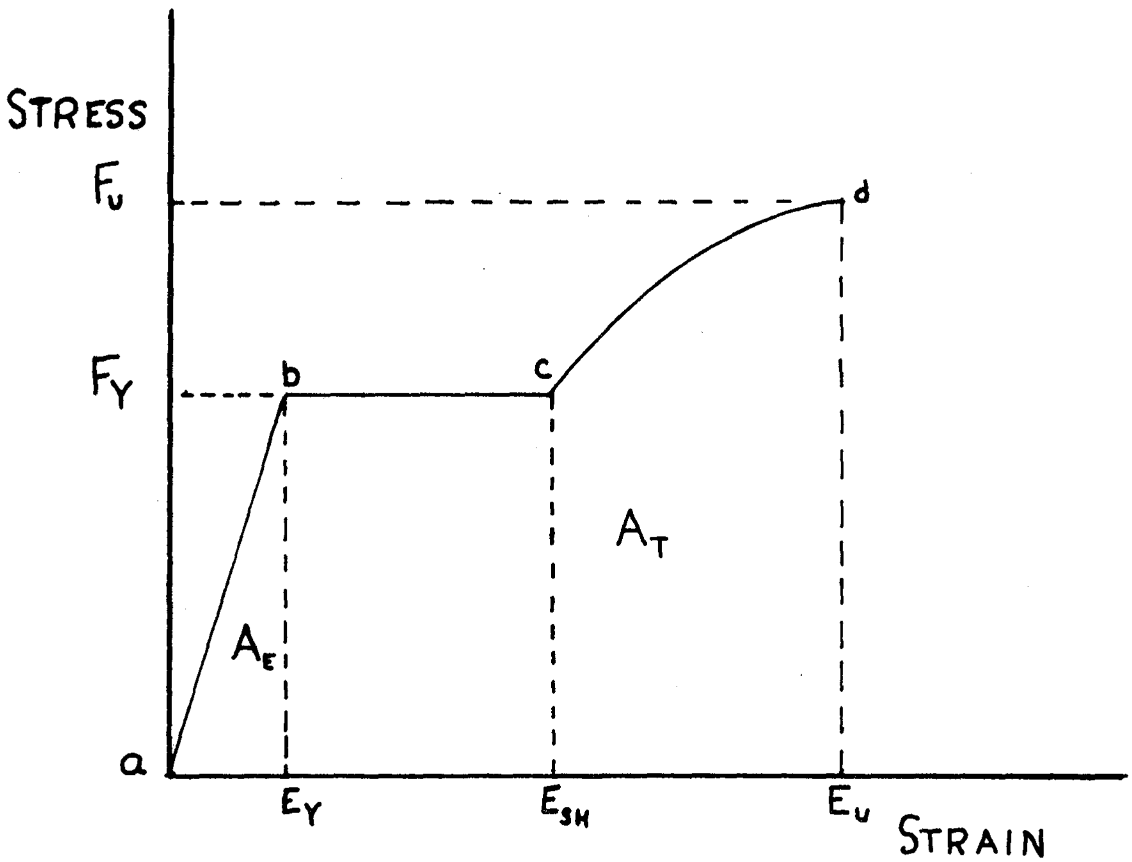


FIGURE 3.10 STRESS STRAIN DIAGRAM FOR STEEL

E_u, is the inelastic absorption capacity.

As stated above, it is the absorption of energy in the inelastic mode which causes the structure to be damaged. It is obvious that as the amount of inelastic energy absorbed increases, so do the permanent displacements and the structural damage. Therefore, it can be concluded that the amount of structural damage is related to the amount of inelastic energy absorbed by the structure.

To determine the nature of the relationship between the inelastic energy absorbed and the structural damage, consider an individual column of a steel building which is subjected to an earthquake. During the course of the shaking, the bending strain in the column, and the amount of energy absorbed, is constantly changing. For the purpose of demonstration consider the distribution of strain along the column at the time when the top of the column has its maximum displacement with respect to its base. At this time, consider the average maximum bending strain in the upper half of the column. If this strain is equal to or less than E_y, the column will have only absorbed elastic energy and therefore remain undamaged. If, on the other hand, the average maximum strain in the upper half of the column is equal to E_u, the column will have absorbed as much energy as it is capable of doing, and the element will certainly fail. In this case the column has to be replaced. Therefore, the ratio of the damage to the column to the initial cost of the column is equal to one. To summarize, let

$$D = \frac{\text{Damage to a Structural Element}}{\text{Initial Value of the Structural Element}}, \text{ and}$$

A_x = Amount of inelastic energy absorbed to develop a strain E_x.

Then if

$$E_x \leq E_y$$

$$A_x = 0$$

$$A_x/A_T = 0$$

$$D = 0$$

And if

$$E_x = E_u$$

$$A_x/A_T = 1.0$$

$$D = 1.0$$

This example suggests that the ratio D might possibly be related to A_x/A_T. The only way to truly justify this relationship is to compare the ratio D, computed in this way, to values of D computed from the damage to real buildings in actual earthquakes. This comparison is performed in chapter 4. In lieu of chapter 4, consider the following conceptual justification. If an element absorbs an amount of inelastic energy, A_x, then a certain fraction of the total inelastic energy absorption capacity of that element has been dissipated. To restore the structure to its pre-earthquake condition, a certain amount of money must be spent to repair or replace the structural elements for which A_x exceeded A_E (elastic energy capacity). Clearly, as A_x becomes a larger fraction of A_T, the amount of money spent to repair the structure becomes a greater fraction of its initial value. Therefore, considering all of the above, it is reasonable to assume:

$$D = \left| \frac{A_x}{A_i} \right|^n \quad . \tag{3.1}$$

To determine the exponent, n, in Eq. (3.1), it is necessary to investigate the structural damage to buildings in past earthquakes. If it were possible to compute D for every building damaged by the San Fernando earthquake, there would be evidence that the general trend is as shown by curve A of fig. 3.11. Curve A would be expected for two reasons: a) the damage was quite small at moderate levels of response,* and b) small changes in the level of response resulted in only small changes in damage. If one were able to compute D for all buildings damaged in Caracas, the general trend would be that of curve B in fig. 3.11. Curve B would be expected for Caracas because: a) at relatively small levels of response the damage was quite high, and b) small changes in the response did result in large changes in the damage. If the problem were considered in the absence of any physical evidence, curve C of figure 3.11 might be expected. In the neighborhood of zero and 100% damage, it would be expected that small changes in the response would not change the damage significantly. Faced with this evidence, the most logical

* It is subsequently shown how the structure's response is related to the ratio A_x/A_T.

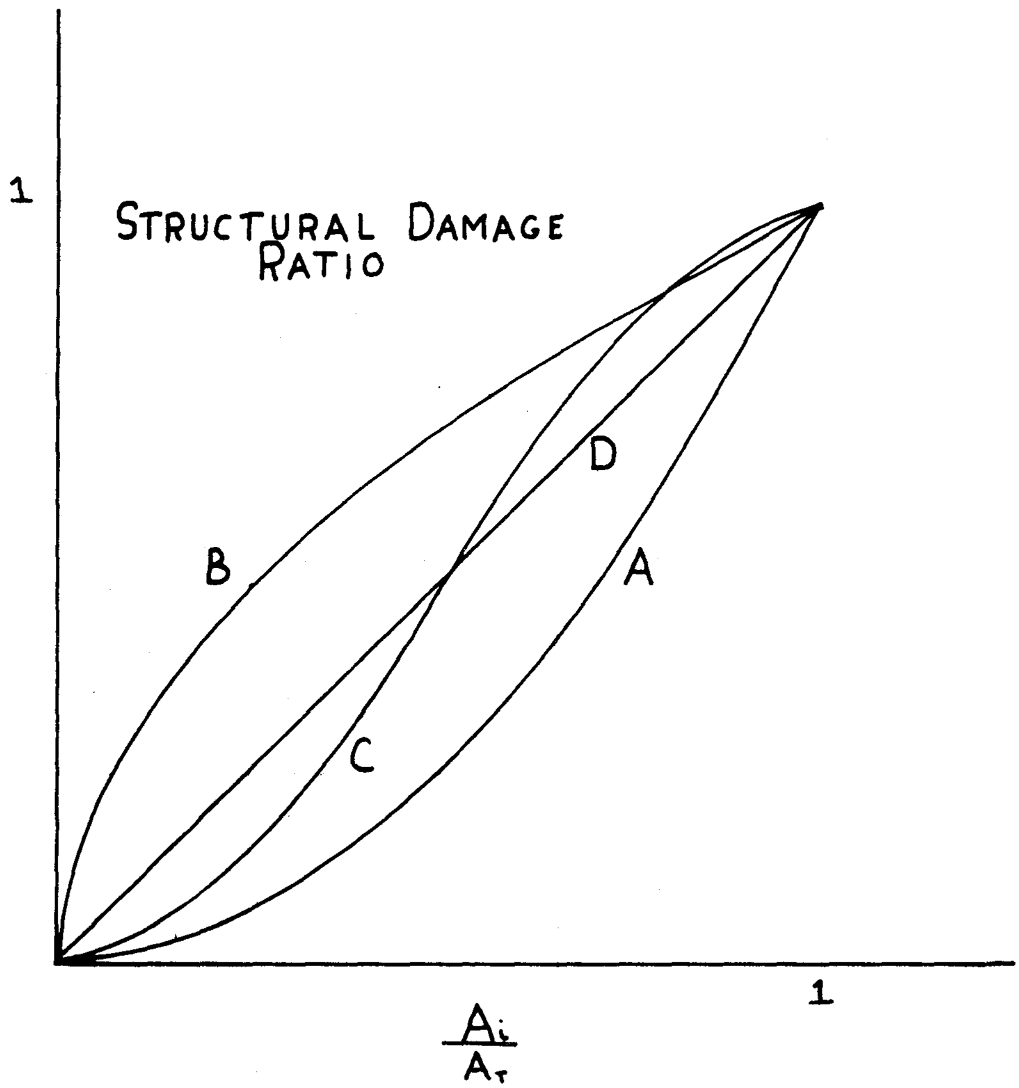


FIGURE 3.11 POSSIBLE SHAPES OF DAMAGE—ENERGY RELATIONSHIP

variation of D with A_X/A_T is shown in curve D, and the best estimate for the exponent n in Eq. (3.1) is n=1.

This concludes the arguments for using the ratio A_X/A_T as a measure of structural damage. The following sections contain the development of damage models for four structural systems.

3.4.1.1 Structural damage models for four different structural Systems

To compute the structural damage to frame buildings, the following idealizations are necessary:

1. Only damage to columns is important (section 3.3.2).
2. All girders are rigid (shear beam behavior).
3. Computation of strains is based on elastic formulas.

The procedure for computing structural damage to frame buildings is as follows:

Step A. For every column of every story, compute A_T.

Step B. Based on the peak interstory displacement and the geometry of the column, compute the maximum bending strain at the quarter point of every column.

Step C. Based on the bending strain computed in B and a stress-strain relationship, compute A_X for every column.

Step D. Compute A_X/A_T for every column in every story. Compute the average A_X/A_T for every story.

Step E. The structural damage is the average of the average A_X/A_T for all stories.

The expressions used to perform these computations are given below. A more complete derivation of these expressions can be found in Appendix A.

STEEL MOMENT RESISTING FRAME

The idealized stress-strain relationship used in the development of the model is shown in fig. 3.12A (15). The relationship consists of three parts:

a) Region AB - (linear elastic region) stress is proportional to strain)

b) Region BC - (yield plateau) constant stress for increasing strain)

c) Region CD - (strain hardening) parabolic variation of stress with strain).

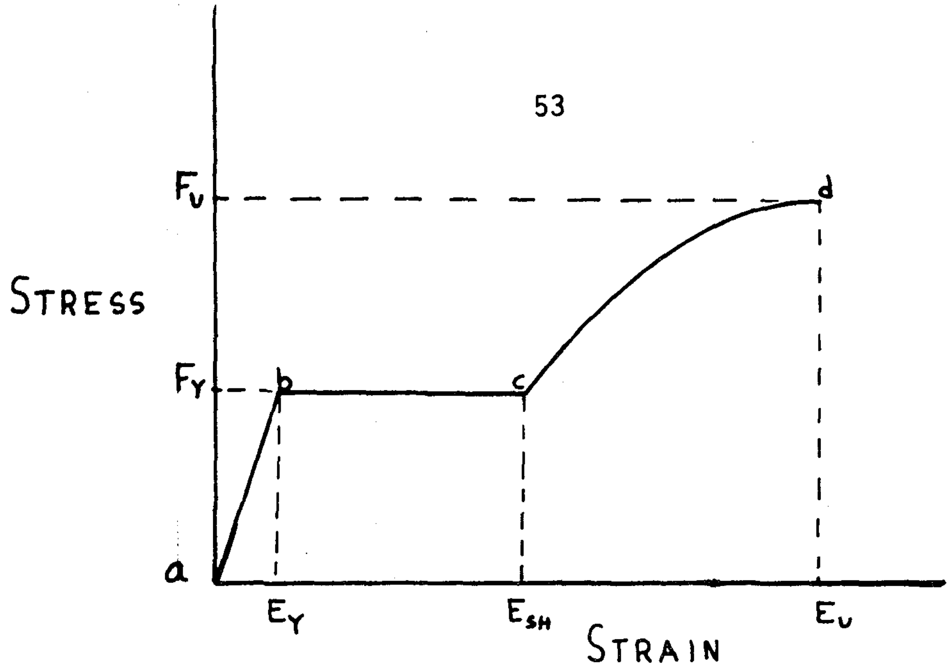
In addition to the values of E_y, E_{SH}, E_u, F_y and F_u which are necessary to define the shape of the stress-strain relationship, it is convenient to define two other parameters which appear in the equations for the region CD:

$$R = E_u - E_{SH} \qquad (3.2)$$

$$S = \frac{\frac{F_u}{F_y}\,(30R + 1)^2 - 60r-1}{15R^2} \qquad (3.3)$$

(Reference 15)

The procedure for computing structural damage to steel frames is summarized on the following page.



(A) Ductile Steel

(B) Confined Concrete

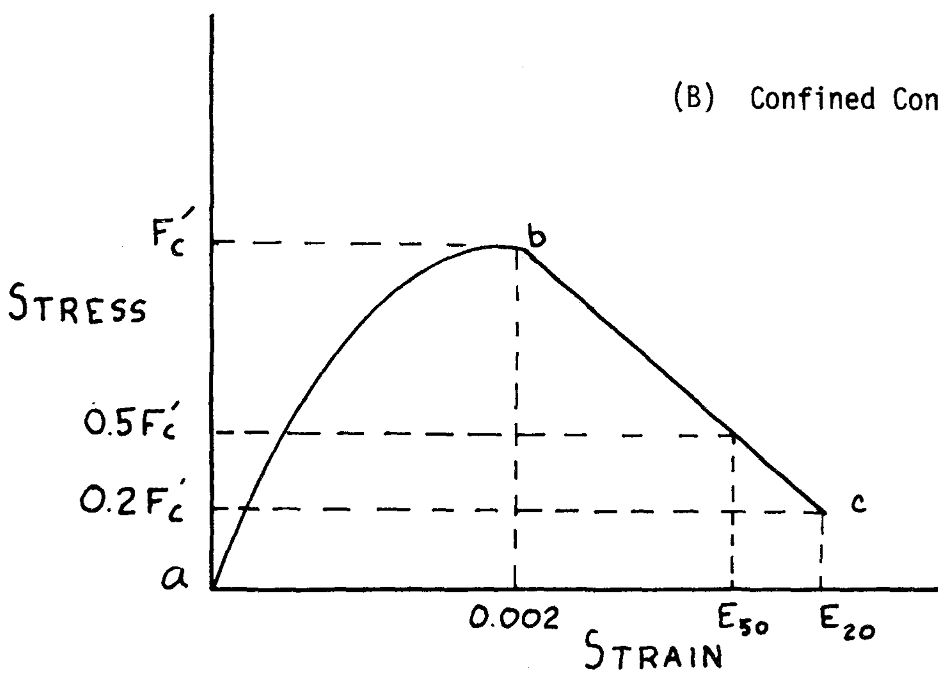


FIGURE 3.12 IDEALIZED STRESS STRAIN MODELS

STEP A: The area under the stress-strain curve from E_y to E_u

$$A_T = F_y \left[(E_{SH} - E_y) + \frac{SR}{60} - \frac{1}{3600} \left\{ S(60\,E_{SH} - 2)(Ln(2/S) \right. \right.$$

$$\left. - \frac{Ln(60\,E_u - 60\,E_{SH} + 2)}{S} \right\} + \frac{1}{60} \left\{ S(E_{SH} - 2)(Ln(60\,E_u - 60E_{SH} + 2) \right.$$

$$\left. \left. - Ln\,2 \right\} + (60-S)(\tfrac{1}{2}\,E_u^2 + E_u E_{SH} - \tfrac{3}{2}\,E_{SH}^2) \left(\frac{1}{2(30R + 1)^2} \right) \right] \qquad (3.4)$$

STEP B: Compute the maximum bending strain at the quarter point of each column. To perform this computation it is assumed that the column is distorted as shown in fig. 3.13A, and the distribution of bending moment (and strain) is as shown in fig. 3.13B. For the lateral load P

$$Y = \frac{PH^3}{12EI}$$

$$M = \frac{PH}{2}$$

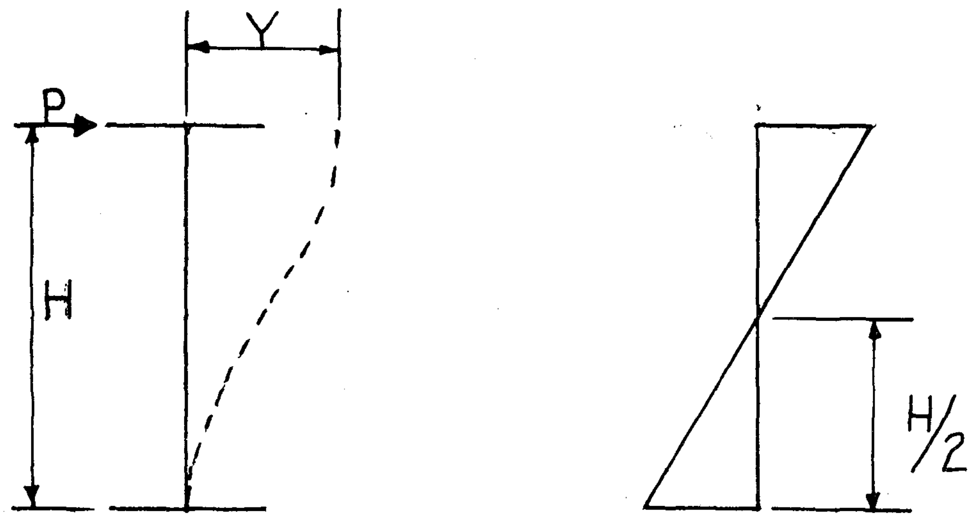
$$Y = \frac{MH^2}{GEI}$$

$$\frac{M}{EI} = \frac{6Y}{H^2}$$

For this column $\frac{M}{EI} = \frac{E_x}{T/2}$, approximately. ($E_x$ = strain, T = depth)

$$E_x = \frac{6YT}{2H^2} \quad \text{at top of column}$$

$$E_x = \frac{6YT}{4H^2} \quad \text{at quarter point of column.} \qquad (3.5)$$



(A) Distortion of Column

(B) Distribution of Strain

FIGURE 3.13 IDEALIZED COLUMN BEHAVIOR

STEP C. Compute the area under the stress-strain curve from E_y up to any value of E_x.

(1) If E_x is less than or equal to E_y

$A_x = 0$. Only elastic energy absorbed in this range.

(2) If E_x is greater than E_y but less than E_{SH}

$$A_x = F_y(E_{SH} - E_x) \tag{3.6}$$

This term gives the energy absorbed during yielding.

(3) If E_x is greater than E_{SH} but less than E_u

$$A_x = F_y \left[(E_{SH}-E_y) + \frac{SR}{60} - \frac{1}{3600}\left(S(60E_{SH}-2)(\mathrm{Ln}(2/S) - \ln \frac{(60E_x-60E_{SH}+2)}{S}\right) + \frac{1}{60}\left\{S(E_{SH}-2)(\ln(60E_x-60E_{SH}+2)-\mathrm{Ln}(2)\right\} + (60-S)(\tfrac{1}{2}E_x^2 + E_xE_{SH} - \tfrac{3}{2}E_{SH}^2)\left(\frac{1}{(2(30R+1)^2}\right)\right] \tag{3.7}$$

This term includes energy absorbed during yielding and the energy absorbed during strain hardening.

STEP D. Compute damage ratio for each column and average damage ratio for each story.

$$D = \frac{A_x}{A_T}$$

$$\overline{D} = \frac{1}{k} \sum_{\substack{\text{All columns} \\ \text{in story}}} A_x/A_T$$

k = number of columns per story

STEP E. Compute the average structural damage ratio for the whole building.

$$\text{Average structural damage ratio} = \frac{1}{n} \sum_{\text{all stories}} \overline{D} \;;$$

$$n = \text{number of stories}$$

CONCRETE MOMENT RESISTING FRAME

The behavior of a concrete moment resisting frame is similar to that of a steel frame. To compute the damage to concrete frames, the same procedure demonstrated for the steel frame can be used with a different set of equations for the stress-strain relationship. The stress-strain relationship which is used to develop a damage model for concrete frames is shown in fig. 3.12B. In this figure, the region AB is a parabolic variation of stress with strain. The parabolic variation governs the region from the origin to compressive strength of the concrete. The region BC in fig. 3.12B gives a linear variation of stress with strain. The slope of this straight line is:

$$Z = \frac{0.5}{E_{50} - 0.002} \qquad \text{(Reference 15)} \qquad (3.8)$$

where E_{50} is the strain at 0.5 F_c' on the falling portion of the curve. E_{50} is determined by the amount of confinement within the concrete core, and is a function of F_c', P_S (the ratio of the area of transverse reinforcement to the area of the confined core), and S_h (the spacing of transverse reinforcing).

$$E_{50} = 3/4\ P_s \left(\frac{b''}{S_h}\right)^{1/2} + \frac{3 + 0.002\ F_c'}{F_c' - 1000} \qquad \text{(Reference 15)}$$

In the development of a damage model, it was intended that the model could be applied to buildings without the knowledge of exact details of the structural frame. For this reason the ACI Code minimum values for P_s, b'' and S_h are used. The values chosen are:

$P_s = 0.12 \dfrac{F_c'}{F_y}$ ACI Section A.6.4.2

$b'' = W-3$ ACI Section 7.14.1

$S_h = 4''$ ACI Section 11.1.2*

Using these values, the expression for E_{50} is

$$E_{50} = 0.45 \left(\frac{F_c'}{F_y}\right)\left(\frac{W-3}{4}\right)^{1/2} + \frac{3 + 0.002\ F_c'}{F_c' - 1000} \qquad (3.9)$$

Concrete is not capable of absorbing energy in the elastic mode. However, in working stress design, it was assumed that the concrete behaved in a linear elastic manner up to 0.45 F_c'. It is reasonable to assume that the energy absorbed in developing a stress of 0.45 F_c' causes no damage to the structure. For the assumed stress-strain curve, the strain which corresponds to 0.45 F_c' is equal to 0.00052.

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* ACI section 11.1.2 gives minimum $S_h = d/2$. 4" is used here because earthquake shears typically require spacings less than the maximum allowable. A spacing of less than 4" would probably not be used in practice.

The area under the stress-strain relationship from zero to 0.00052 is neglected in the computation of A_T, the total energy absorption capacity.

E_{20} is the strain developed on the straight line portion of the stress-strain curve (region BC in fig. 3.12B) for a stress of $0.2\ F_c^{'}$. For the purpose of developing a damage model it is assumed that E_{20} represents the limit of practical usefulness of the concrete. An expression for E_{20} is:

$$E_{20} = \frac{0.8 + 0.002\ Z}{Z} \ . \qquad (3.10)$$

It is now possible to proceed with the equations of the structural damage model for concrete moment resisting frames.

STEP A. Compute the area under the stress-strain curve from 0.00052 to E_{20}

$$A_T = F_c^{'}\ [0.00179 + (E_{20} - 0.002)(1 + 0.002\ Z) - \frac{Z}{2}\ (E_{20}^2 - (0.002)^2] \qquad (3.11)$$

STEP B. Compute the maximum bending strain at the quarter point of every column. See Step B of steel-frame procedure for the derivation

$$E_x = \frac{6YT}{4H^2} \qquad (3.5)$$

STEP C. Compute the area under the stress-strain diagram for any given value of E_x

(1) If E_x is less than 0.00052

$$A_x = 0.0$$

It is assumed that elastic energy is absorbed in this range.

(2) If E_x is greater than 0.00052, but less than 0.002

$$A_x = F_c^{'} [500\ E_x^2 - 8330\ E_x^3 - 0.000134] \tag{3.12}$$

(3) If E_x is greater than 0.002, but less than E_{20}

$$A_x = F_c^{'} [0.00179 + (E_x - 0.002)(1 + 0.002\ Z) - \frac{Z}{2}(E_x^2 - (0.002)^2)] \tag{3.13}$$

STEP D. Compute the damage ratio for each column and the damage ratio for each story.

$$\overline{D} = \frac{1}{k} \sum_{\substack{\text{All columns}\\ \text{in story}}} A_y/A_T; \quad k = \text{number of columns per story}$$

STEP E. Compute the average structural damage ratio for the whole building.

$$\text{Average structural damage ratio} = \frac{1}{n} \sum_{\text{All stories}} \overline{D};$$

$$n = \text{number of stories.}$$

STEEL BRACED FRAMES

The behavior of steel braced frames is more closely related to the behavior of a shear wall rather than the behavior of a rigid frame. For damage considerations, the important difference between braced frames and shear walls is that the braced frame is composed of discrete elements rather than a continuous mass. Structural damage to a braced frame can occur in two ways. There may be damage to columns, which is caused by bending strains in excess of the yield strain. In addition, the interstory displacements of the frame may cause individual braces within the story to buckle. Due to these two sources of structural damage, the total damage to a given story of a braced frame must be computed in two parts.

For the purpose of developing a damage model, it is assumed the structural damage to the columns can be estimated by the amount of inelastic energy absorbed at the quarter-point of the column. This assumption is only valid when the rigidity of the beams and floor slabs which restrain the column's ends is much greater than the rigidity of the column. For these conditions, the deflected shape and distribution of bending strain of the column will be similar to that shown in fig. 3.13A and 3.13B. The procedure used to compute structural damage to the columns of a steel braced frame is identical to steps A, B, and C of the steel frame model. The structural damage ratio for the columns is defined as $Dc = A_X/A_T$.

If the interstory displacement of a story is not large enough to cause the bracing to buckle, then the average structural damage for that floor is equal to the average of the A_X/A_T for all of the columns in that story. If the interstory displacement does cause the bracing to buckle, then it is assumed that it must be replaced.

In this case, the ratio of the damage to the bracing to the value of the bracing is one. To combine bracing damage ratio with the column damage ratio it is necessary to determine the value of the bracing relative to the value of the entire structure. It is assumed that the ratio of the value of the bracing to the value of the structure is equal to the ratio of the volume of bracing to the volume of the structure. For any given floor the ratio L is defined as:

$$L = \frac{\text{Value of Bracing}}{\text{Value of Structure}} = \frac{\text{Volume of Bracing}}{\text{Vol. of Columns + Vol. of Beams + Vol. of Bracing}}$$

To summarize, the computation of structural damage to a braced frame is as follows:

1. Compute damage to the columns using the procedure given for steel frames

   $$D_c = A_x/A_T$$

   $$\overline{D}_c = \frac{1}{k} \sum_{\text{All columns}} A_x/A_T; \qquad k = \text{number of columns per story}$$

2. If the interstory displacement does not cause the bracing to buckle, then,

   $$\overline{D} = \overline{D}_c$$

   $$\text{Average structural damage ratio} = \frac{1}{n} \sum_{\text{All stories}} \overline{D}\ ;$$

   $$n = \text{number of stories}$$

3. If the interstory displacement does cause the bracing to

buckle, then compute L as shown above.

$$\overline{D} = \overline{D}_c + L$$

$$\text{Average structural damage ratio} = \frac{1}{n} \sum_{\text{All stories}} \overline{D} \; ;$$

$$n = \text{number of stories}$$

3.4.1.2 Structural damage to shear walls

The behavior of shearwalls is different from the behavior of frames; therefore, the procedure developed above for the computation of structural damage to frame buildings does not apply. The general method of computing the amount of energy absorbed and comparing this to the energy capacity must be slightly modified to account for: a) Shear walls are far-coupled systems, while frames can be adequately modeled as close-coupled systems, and b) a shear wall absorbs energy through shear and bending, while for frames, the shear mode is relatively unimportant. The implication of the fact that shear walls are far-coupled systems is that the computation of the average strain in a given story must account for the behavior of the entire shear wall rather than just the portion of shear wall in that story. The derivation of an expression which gives the strain at any level of the wall is given in Appendix A. This derivation assumes that the shear wall behaves like a cantilever beam, and, at the time of maximum distortion, there is a triangular distribution of lateral loads acting on the beam. As shown in Appendix A, this leads to the following expression for bending strain at any level.

$$E_x = \frac{\frac{\Delta T}{2}\left[\frac{x^2}{2} - \frac{x^3}{GH}\right]}{\left[\frac{x^4}{24} - \frac{x^5}{120H} - \frac{xH^3}{8} + \frac{11H^4}{120}\right]} \tag{3.14}$$

In this expression x is the distance from the top of the wall to any level, Δ is the maximum total displacement at x, T is the depth of the wall, and H isthe total height of the wall.

The computation of the structural damage to shear walls by the general method would require: a) determination of the total amount of energy which can be absorbed in the shear and bending modes, and b) determination of the amount of energy absorbed in both modes for a given set of distortions. To apply the general method to a shear wall, the following idealizations are used. As shown above, only the bending strain is computed; therefore, only energy absorption in the bending mode is considered. To account for the fact that energy absorbed in shear may cause considerable damage to the wall, the energy capacity of the wall in bending is limited to the amount of energy which can be absorbed in the bending mode prior to shear failure of the wall. To determine this reduced bending capacity, the maximum bending strain at the point of shear failure was determined (12). The reduced bending energy capacity is the area under the stress-strain curve for concrete from zero up to the bending strain at shear failure. For a given wall and set of maximum displacements, the bending energy absorbed at the mid-height of every story is computed and compared to the reduced bending energy capacity. The damage to a shear wall at a given level is:

$$D = \frac{A_x'}{A_R}$$

$$\overline{D} = \frac{1}{k} \sum_{\substack{\text{All shear} \\ \text{walls}}} \frac{A_x}{A_R} \qquad k = \text{number of shear walls}$$

$$\text{Average structural damage} = \frac{1}{n} \sum_{\text{All stories}} \overline{D}$$

$$n = \text{number of stories.}$$

It is not uncommon for a building to be comprised of shear walls and frames. As previously stated and demonstrated in chapter 4, the damage suffered by frame elements and shear wall elements is quite different. This difference is a result of the fact that shear walls are more rigid than frames; therefore, they attract a larger portion of the lateral load. However, to compute the structural damage to a given floor of a building comprised of shear walls and frames, the damage to each element is computed separately. The average structural damage to a floor of such a building is the average of the damage suffered by the frame and shear wall elements. Computing the damage in this way assumes that the shear wall and frame elements are of equal value within the structural system. While this assumption may not be valid for real buildings, the application of this averaging process appears to work fairly well (see chapter 4).

3.4.2 Nonstructural Damage

3.4.2.1 General considerations

As previously stated, nonstructural elements are capable of absorbing energy during an earthquake. Therefore, it should be possible to apply the energy method developed for the structural system to the computation of damage to nonstructural elements. It was also stated that the capacity of these elements is generally ignored. For this reason there is very little information on how much energy nonstructural elements can absorb. Therefore, it is impossible to extend the technique for computing damage to nonstructural elements. However, the general idea of the way elements are damaged during earthquakes can be extended to include nonstructural items.

In the development of damage modes for nonstructural elements, the following procedure was used:

1. As stated previously, the important contribution to earthquake damage comes from elements which are orientated vertically. Within the realm of nonstructural items the vertical elements are drywall partitions, brick masonry walls, concrete block walls, and exterior glazing.

2. Also previously given was a conceptual argument that interstory displacements are the prime cause of damage to vertical elements.

3. In determining a relationship between the damage to an element and interstory displacement, a knowledge of the behavior of the element defines the shape of the relationship. For example, if the element possesses some ductility, then it is reasonable to assume that the relationship between damage and interstory

displacement should be a linear function of interstory displacement. If, on the other hand, the element is very brittle, a step function might be more appropriate.

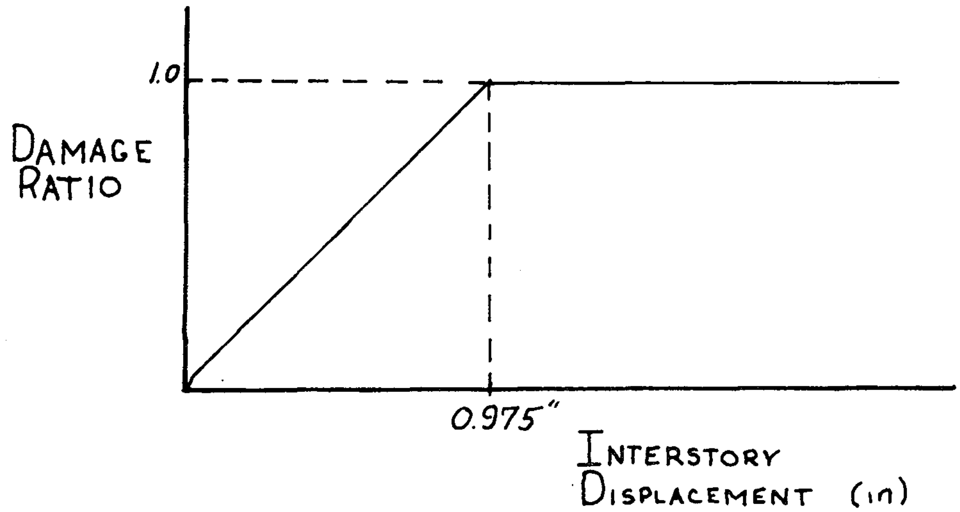
4. In the derivation of a damage model for nonstructural elements, certain limiting values can be used. For example, if an element is isolated from the structural frame by a certain tolerance, then the element will be undamaged until the interstory displacement exceeds this tolerance. If values of ultimate strength and stiffness of an element can be obtained, one can determine a value of interstory displacement at which the element will be 100 percent damaged.

5. The final step in the development of nonstructural damage models is to apply the models to buildings which have been damaged in earthquakes. This comparison determines if the shape of the relationship and limiting values are reasonable. It should be noted that different buildings have various amounts of nonstructural components. For example, one building may have a large number of drywall partitions, while another may have relatively few. For the buildings which were used to test the validity of the nonstructural damage models, it was not possible to determine which buildings had more or less nonstructural elements. Therefore, the models which were developed give a value for drywall partition damage, for example, which does not account for the number of partitions in a given building.

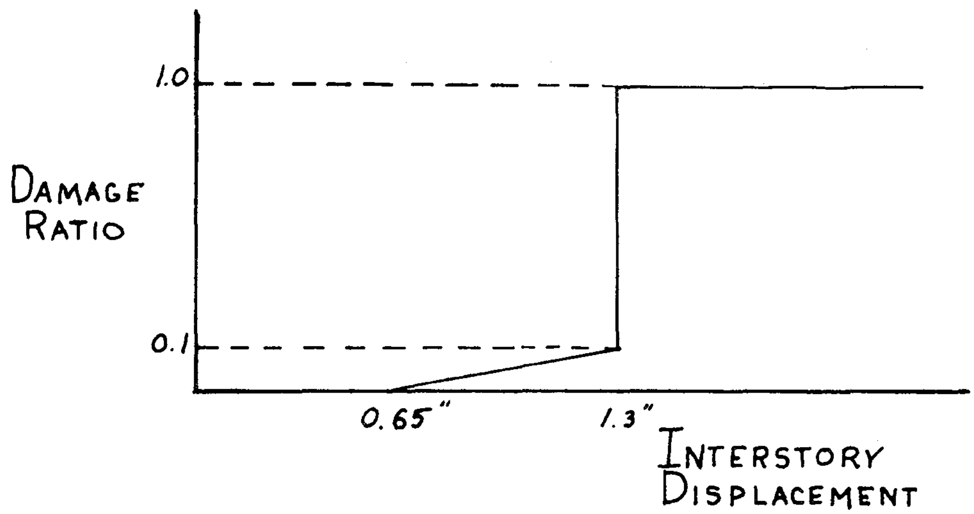
The following paragraphs show how mathematical models for damage to nonstructural elements were developed. Where it is applicable, the concept of energy absorption is used to develop the model.

Drywall partition and ceramic tile are characterized by the fact that they have essentially zero strength when subject to seismic movements; therefore they are incapable of absorbing energy. Seismic protection for partitions and tile is usually provided by allowing a dimensional tolerance in the installation of these elements. This tolerance is usually one to one-and-one-and-a-half inches (13). For a first estimate, it would seem logical to assume that the relationship between partition damage and interstory displacements might be a step function. An attempt to apply a step function to partition damage in real buildings showed that a linear variation, rather than a step function, was more appropriate. A linear variation of partition damage with interstory displacement is the result of two factors. Firstly, partitions are generally plastered and painted, rather than replaced, even when severely damaged. Secondly, the tolerances used in installation vary quite a bit from the values given above; therefore, the value of interstory displacement at which the partitions become engaged to the structural frame will vary from building to building. The partition damage vs. interstory displacement shown in fig. 3.14 is primarily derived from partition damage to real buildings during earthquakes.

The most consistent agreement between the partition damage model and partition damage to real buildings was found when a linear variation of partition damage ratio with interstory displacement was used. The best agreement was obtained when the curve in fig. 3.14A passes through the origin and the point whose coordinates are: damage ratio = 1.0, interstory displacement = 0.975.



(A) Drywall Partitions



(B) Glazing

FIGURE 3.14 NONSTRUCTURAL MODELS

Glazing is another nonstructural element which possesses no resistance to lateral loads. Unlike partitions, glass damage cannot be repaired if the tolerance of installation is exceeded. However, even if this tolerance is not exceeded, it may be necessary to realign and recaulk the window frame. In view of this evidence, it is reasonable to assume that the variation of glass damage with interstory displacement would be a linear function of interstory displacement up to the point where the glass is fully engaged. At this point there should be a discontinuity up to damage ratio equal to one.

An investigation of glazing damage to real buildings in actual earthquake resulted in the glass damage model shown in fig. 3.14. No glass damage was reported when interstory displacements were less than 0.65". Through the study of glass damage in past earthquakes it was found that the end of the linear region should be at the point whose coordinates are: glass damage ratio = 0.1, interstory displacement = 1.3". At an interstory displacement of 1.3" there is a sharp discontinuity. This step represents the point at which the glass becomes fully engaged to the movement of the structural frame. Due to the brittle nature of glass, any glass panel which is distorted beyond tolerance used to install the glass will certainly be totally damaged and require replacement.

Masonry brick and concrete block walls must be treated in a different manner than drywall partitions and glazing. Masonry and concrete walls are not exactly nonstructural elements, because they are capable of resisting lateral loads; therefore they are capable of absorbing energy. It is reasonable to assume that the energy method which was developed for structural damage can be extended to cover masonry brick and concrete block walls.

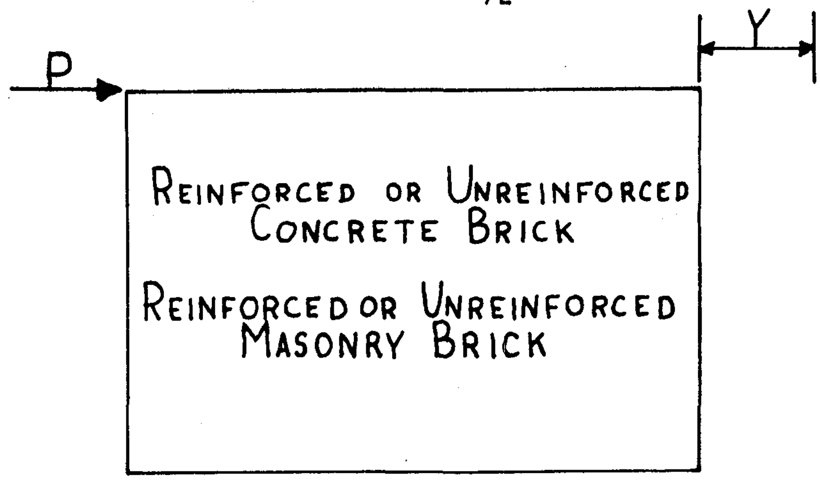
In fig. 3.15A is given the loading condition which walls are subject to during an earthquake. P is the lateral load and Y is the lateral deflection at the top of the wall. Fig. 3.15B is an idealization of the lateral load—lateral deflection curve for a wall. Y_{CR} is the displacement at which major cracking occurs. It is significant to note that displacements less than Y_{CR} will cause minor cracking; therefore, energy absorbed prior to Y_{CR} cannot be neglected. In fig. 3.15B Y_u is the deflection at which resistance of the wall is reduced to the friction along adjacent cracked surfaces.

The energy absorption capacity of the wall is given by the area under the load deflection curve up to deflection Y_u.

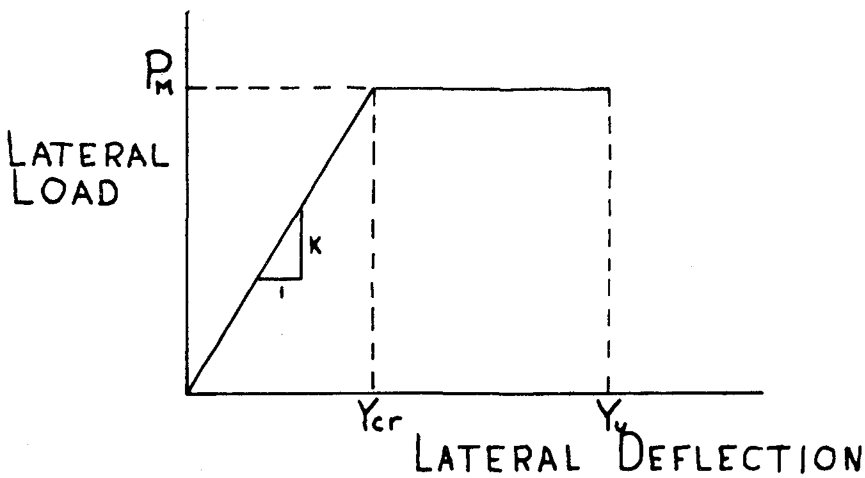
$$A_T = \frac{1}{2} P_m Y_{CR} + P_m(Yu - Y_{CR})$$

$$A_T = k \left[Y_u Y_{CR} - \frac{1}{2} Y_{CR}^2\right] \tag{3.15}$$

The amount of energy absorbed by the wall while going through a deflection Y_i depends upon which portion of the curve Y_i falls. If Y_i is less than or equal to Y_{CR}, the energy absorbed is given by



(A) Loading Condition



(B) Lateral Load - Lateral Deflection

FIGURE 3.15 IDEALIZED BRICK WALL BEHAVIOR

$$A_i = \frac{1}{2} P_i \; Y_i = \frac{1}{2} k \; Y_i^2 \tag{3.15}$$

The ratio of the damage to the wall to the value of the wall is

$$D_W = \frac{A_i}{A_T} = \frac{Y_i^2}{2 \; [Y_u \; Y_{CR} - \frac{1}{2} Y_{CR}^2]} \tag{3.16}$$

If Y_i is greater than Y_{CR}, the energy absorbed is

$$A_i = \frac{1}{2} k \; Y_{CR}^2 + k \; Y_{CR} \; [Y_i - Y_{CR}]$$

$$A_i = k \; [Y_{CR} \; Y_i - \frac{1}{2} Y_{CR}^2] \tag{3.17}$$

For this case the damage ratio for the wall is given by

$$D = \frac{A_i}{A_T} = \frac{[Y_{CR} \; Y_i - \frac{1}{2} Y_{CR}^2]}{[Y_u Y_{CR} - \frac{1}{2} Y_{CR}^2]} \quad . \tag{3.18}$$

This formulation for the damage to walls applies whether the wall is reinforced masonry brick, unreinforced masonry brick, reinforced concrete brick, or unreinforced concrete brick. To develop four damage models it is only necessary to determine the appropriate values of Y_{CR} and Y_u.

An extensive series of static shear loading tests on masonry and concrete brick walls was conducted at Stanford University in the early 1950's. Various types of masonry brick, concrete brick, joint material, and reinforcing steel patterns were included in the tests. Load deflection curves for these tests are reported in references 16 and 17. For the development of a brick wall damage model it is necessary to investigate many load deflection curves, and determine average values of Y_{CR} and Y_u. For example, in the case of unreinforced masonry, the authors of references 16 and 17 tested a number of walls constructed of brick whose compressive strength varied from very low to very high. The values of Y_{CR} and Y_u recorded during the testing changed as the compressive strength of the brick changed. The values of Y_{CR} and Y_u used for the unreinforced masonry brick wall damage model are the average of all the values recorded during the testing. A similar averaging process was used in determining Y_{CR} and Y_u for reinforced brick masonry, unreinforced concrete brick, and reinforced concrete brick. The values used for the damage models are shown below.

Damage Model	Y_{CR} (in)	Y_u (in)
Unreinforced Masonry	0.02	0.05
Reinforced Masonry	0.05	0.15
Unreinforced Concrete	0.02	0.07
Reinforced Concrete	0.06	0.20

When brick walls are used as infill material, there is typically a gap left between the frame and the wall. This gap must be exceeded before the wall will experience any movement. Standard practice in the seismic design of buildings would result in a tolerance of installation of between one-quarter and one-half inches.* For the purpose of a brick wall damage model, 0.375" was chosen as an average tolerance

* Values obtained from the structural engineering firm mentioned in the Acknowledgement.

of installation. The final model recommended the load displacement curve for brick wall is shown in fig. 3.16. The damage models are derived from this curve using the procedure developed above and the appropriate values for Y_{CR} and Y_u.

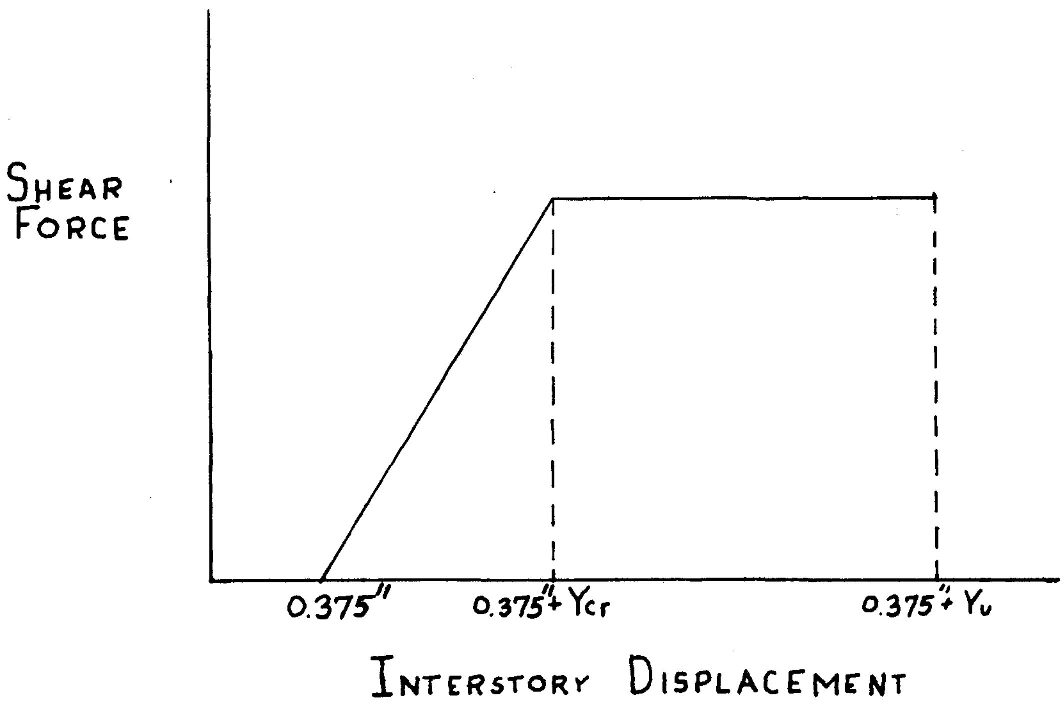


FIGURE 3.16 LOAD-DEFLECTION CURVE FOR BRICK WALLS

CHAPTER 4. APPLICATION OF DAMAGE MODELS TO BUILDINGS

4.1 Background

Mathematical models for the computation of earthquake damage were developed in Chapter 3. Although the damage models are quite simple in concept, the application of these models to multi-story buildings would be a laborious task; therefore, the algorithms for computing damage to different items in different buildings were organized into a computer program. By automating the damage models it became possible to test the validity of the models, and to compute earthquake damage for the buildings which were included in the optimization study.

4.2 Comparison of Computed Damage to Observed Damage

As stated in Chapter 3, the best way to justify the mathematical models for damage is to apply them to buildings which were damaged during earthquakes. To apply the damage models to a building, one must know the general characteristics of the building and of the earthquake which caused the damage. The geometry of the structural system and the kind of structural material would be useful input. The interstory displacement at every floor and the percentage of damage in every category mentioned in Chapter 3 would also be necessary to compare the computed damage to the observed damage.

To compare computed and observed damage, the damage models were applied to several buildings which were damaged during the 1971 San

Fernando earthquake. The buildings which were used in this area of the study are the same as those described in section 3.3.2. Included in Chapter 3 is an explanation of how the damage and motion information for these buildings was obtained. It would have been advantageous to include more buildings in the comparison; however, at the present time, there are only a few buildings for which sufficiently detailed information could be obtained.

The results of the application of the damage models to five buildings which were damaged during the 1971 San Fernando earthquake are given in table 4.1. Except for the other damages category, the numbers in this table represent the ratio of the damage to a particular building component to the initial value of that particular building component. For the sake of comparison, the damage which was reported to have occurred in these buildings is shown adjacent to the damage which was computed by the mathematical models. The pilot buildings from Los Angeles suffered damage to elements which were not included under any of the mathematical models developed in Chapter 3. Among these elements are the heating, ventilating, air conditioning, and elevator systems; suspended ceiling tiles, lighting and plumbing fixtures. To account for the minor damages suffered by these elements, it was decided to include in the damage computations a category which would represent all of these items. In table 4.1 these items are accounted for in the column labeled "Other Damages." The numbers under the "Reported" subheading of the "Other Damages" category were arrived at by dividing the total cost to re-

TABLE 4.1

COMPARISON OF COMPUTED AND REPORTED DAMAGE

	STRUCTURAL DAMAGE		PARTITION DAMAGE		GLASS DAMAGE		OTHER DAMAGES	
	Reported	Computed	Reported	Computed	Reported	Computed	Reported	Computed
HOLIDAY INN MARENGO	0.008	0.020	0.66	0.620	0.0	0.006	0.0	0.00054
HOLIDAY INN ORION	0.004	0.062	0.95	0.968	0.064	0.20	0.00132	0.00112
BANK OF CALI-FORNIA	0.017	L.T.0.001	0.183	0.208	0.122	0.0	0.00185	0.00018
INDIAN HILLS MEDICAL	0.35	0.288	0.0694	0.743	0.0416	0.023	0.120	0.00069
k-B VALLEY CENTER	0.00	0.000	0.201	0.633	0.0	0.007	0.00193	0.0005

pair the items mentioned above by the construction cost for the building in question.

With this data for the five buildings shown in table 4.1, a plot of "Other Damages" versus average peak interstory displacement was constructed. On this plot, the data points for the five buildings were widely scattered. One obvious reason for this scatter is that other damages included different items for each building. An attempt was made to fit a straight line through the origin and these data points. The resulting algorithm for computing damage to items in the "Other Damages" category is:

$$\text{Other Damages} = \frac{\text{Average Interstory Displacement}}{1125.0}$$

where the average interstory displacement is in inches. In table 4.1, it is important to note that the other damages category is given as a fraction of the total cost of the building, while the structural damage, partition damage, and glass damage categories are given as a fraction of the initial value of the particular element.

A close examination of table 4.1 shows that there may be considerable difference between the reported and computed damages. There are many factors which might be cited in an attempt to explain these discrepancies. As mentioned in Chapter 3, there was considerable uncertainty introduced in arriving at the reported damages. It has been assumed that the reported damages are reasonably accurate; however, one must realize that in any given case there may be considerable error in these numbers. The procedure used to compute the response of the buildings shown in table 4.1 was an elastic dynamic analysis. The assumption of elastic behavior is certainly justified for these buildings; however, there may be cases in which the computed response is significantly different from the actual response during the earthquake. This would lead to a computed damage different from the reported damage. There may be other factors which could be cited to explain the discrepancies between the reported and computed damages in table 4.1; however, the largest factor which contributes to the disagreement is the fact that the mathematical models for damage are not sophisticated enough to account for subtle differences in the construction and the earthquake response of seemingly identical buildings. However, it was never intended that the damage models account for these subtle differences. Since this paper represents the first attempt to develop a mathematical model for earthquake damage, and since so little information is currently available to develop and justify the mathematical models, it would be unrealistic to expect exact agreement between computed and reported values. However, it is apparent from table 4.1 that the mathematical models for the prediction of earthquake damage do demonstrate the general trends of damage which was observed in these particular buildings during the San Fernando earthquake. If it had been possible to include more buildings in this comparison of reported and computed damage, it might be justified to modify the damage models and perform another cycle of analysis. However, given the limited amount of data and the uncertainty in the data, it is unrealistic to expect a meaningful result from such an exercise.

It is important to note that the buildings used to test the validity of the damage models did not have any brick walls. Therefore, the four brick wall models developed in Chapter 3 could not be tested. As shown in Chapter 3, the models for brick wall damage were developed using the same theory as the models for structural damage. Since the data from past earthquakes did not provide an opportunity to test the validity of the brick wall models, it has been assumed that these models are accurate to the same degree as the structural damage models.

At this point it might be helpful to summarize the conclusions reached by applying the mathematical models for damage to real buildings, and to restate the limitations of the damage models. Table 4.1 shows that the damage computed by the methods developed in Chapter 3 may be considerably different from the damage actually observed. It is significant to note that the problem of predicting earthquake damage was attempted several ways in the course of this study. None of the methods attempted were as accurate and consistent as the method developed in Chapter 3. Therefore, it is reasonable to state that the method developed in Chapter 3 is currently the best available estimate for the computation of earthquake damage. It should be emphasized that, except for the Other Damages category, the damage computed by the models is the ratio of the cost to repair or replace a given building component to the initial value of that component. For example, if the structural damage ratio were 0.20, this would mean that the cost to repair the structure would be 0.20 times the initial value of the structure. If the damage ratio for a particular element is equal to one, then the amount of money necessary to repair or replace that element is equal to or greater than the initial value of that element. For example, if the brick wall damage ratio is equal to one, it does not mean that the wall is no longer standing. On the other hand, a wall which remains standing but is badly damaged may have the same damage ratio as a wall which has collapsed. Finally it is significant to point out an inconsistency in the formulation of the structural damage models. To compute the strain in a given column, an elastic formula is used. The damage to that column is then computed through the amount of inelastic energy absorbed. This inconsistency could be eliminated by using an elasto-plastic formulation to compute the strains in the columns. However, considering the limited degree of accuracy which can realistically be expected in the computed damage, this more precise formulation is unnecessary and unjustified.

4.3 Damage Computations for Typical Buildings

The purpose for which the earthquake damage models were developed is to apply the models to typical buildings having various levels of seismic protection. The typical buildings were designed according to the Uniform Building Code by a Cambridge structural engineering firm. The general strategy in the design of these buildings was to hold the floor plan constant and to vary the structural system, material of construction, number of stories, and seismic design zone. The damage these buildings might suffer if subjected to a particular earthquake is estimated by the models which were developed in Chapter 3. The buildings which were used in this exercise are:

6-story concrete moment-resisting frame	6-CMRF
11-story concrete moment-resisting frame	11-CMRF
11-story steel moment-resisting frame	11-SMRF
11-story concrete shear wall	11-CSW
17-story concrete shear wall	17-CSW

Each one of these buildings is 200' x 60' in plan. The column spacing is 20' in both directions. The floor system is a 7" flat slab for concrete buildings or a 5" lightweight concrete slab on a 1 1/2" metal deck for steel buildings. Each building was designed by applying the static lateral loads which are required by the U.B.C. regulations for seismic zones 0, 1, 2, 3. In addition, each building was designed for a set of lateral loads twice those of zone 3. This has been referred to as a zone 4 design, although zone 4 does not exist in the U.B.C. In some cases it was found that one design would satisfy the requirements for two or three different seismic zones.

These typical buildings were analyzed with a nonlinear dynamic analysis computer program with an earthquake time history (peak acceleration of 0.27g) used as input. The development of the dynamic analysis program which was used is reported in reference 18. Each building was analyzed in only one direction and the interstory displacements at every floor were used as input to the earthquake damage models. During real earthquakes a building would have displacements along both axes; and a certain portion of the damage to a particular element could be attributed to motion in each direction. However, in the development of the mathematical models for damage, only motions in the plane of the element were considered. The comparison of computed and observed damage given in section 4.2 shows that the total damage to a given element can be adequately estimated by considering the motion in only one direction.

Table 4.2 gives the estimated damages that these typical buildings might experience when subjected to a ground motion of 0.27g. The numbers given under the headings structural damage, partition damage, glass damage, unreinforced brick masonry wall damage, reinforced brick masonry wall damage, unreinforced concrete brick wall damage, and reinforced concrete brick wall damage are the estimated repair cost for those elements expressed as a percentage of the initial value of those elements. Other damages and total damage are expressed as a percentage of the total cost of the building. To compute the total damage, it is necessary to make some assumptions regarding the ratio of the value of a particular element to the value of the entire building. The ratios which were chosen are:

TABLE 4.2

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

$$\frac{\text{Value of Structure}}{\text{Value of Building}} = \frac{1}{4}$$

$$\frac{\text{Value of Drywall Partitions}}{\text{Value of Building}} = \frac{1}{20}$$

$$\frac{\text{Value of Glass}}{\text{Value of Building}} = \frac{1}{20}$$

$$\frac{\text{Value of Unreinforced Brick Masonry Walls}}{\text{Value of Building}} = \frac{1}{20}$$

$$\frac{\text{Value of Reinforced Brick Masonry Walls}}{\text{Value of Building}} = \frac{1}{20}$$

$$\frac{\text{Value of Unreinforced Concrete Brick Walls}}{\text{Value of Building}} = \frac{1}{20}$$

$$\frac{\text{Value of Reinforced Concrete Brick Walls}}{\text{Value of Building}} = \frac{1}{20}$$

It is not realistic to assume that a typical building would have both reinforced and unreinforced walls. In table 4.2, the damage to both kinds of walls is shown for the sake of comparison. To obtain the total damage, it was assumed that the brick walls were unreinforced for zones 0 and 1, and reinforced for zones 2, 3 and 4. When computing the total damage for any particular building, only two of the four brick wall damages shown in table 4.2 were included.

The sum of the component costs accounts for 45 percent of the total value of the building. The remaining 55 percent is contributed by components in the "other" damages category. The component damages are multiplied by the approximate ratio and added together, with other damages included, to give the total damage.

The behavior of frame buildings is different from the behavior of shear wall buildings; therefore the damage estimate for those two kinds of buildings will be discussed separately. For all of the component damages to all of the frame buildings (11-CMRF, 6-CMRF, and 11-SMRF) there seems to be a predominant pattern of damage as the seismic design zone was increased from zone 0 to zone 4. For almost all cases, the component damages and the total damage is greater for a zone 2 or 3 design than it is for buildings designed for zones 0, 1, or 4. This can be explained in the following way. A frame building which is designed for zone 0 or 1 is quite flexible; therefore, the fundamental period of such a building would fall to the right of the predominant peak on a typical earthquake response spectrum. If the same building were designed for zone 2 or 3 it would undoubtedly be somewhat stiffer than a zone 0 or 1 design; therefore, the fundamental period of vibration would be less. By making the period of the building shorter, the location of the building on the response spectrum would be shifted to the left. A modest shift to the left can result in substantial increase in spectral acceleration and possibly a slight increase in spectral displacement. This would lead to increased interstory displacements and increased damage. By designing the building for the zone 4 lateral loads, the building is considerably stiffer than zone 0. For the zone 4 design, the fundamental period may be shifted so far to the left that the spectral displacements are less than those found for the zone 2 or 3 design. Smaller spectral displacements would result in smaller interstory displacements and smaller damages.

The effect on the dynamic characteristics and earthquake response of a building which results from changing the seismic design zone is beyond the scope of this report. An extensive treatment of this topic can be found in Reference 19. For the sake of completeness, the fundamental period and the average interstory displacements for the 0.27g earthquake and an elastic limit earthquake for the various buildings can be found in table 4.3.

Table 4.2 shows that the structural damage to frame buildings is quite small and does not change considerably as the seismic design zone is changed from 0 to 4. At this time there is no way to know if this result is correct or incorrect. However, the study of historical earthquakes demonstrated the fact that earthquake damage is often more closely related to the details of the construction than to the strength of the frame. It was also found that well designed frame buildings have withstood severe shaking with only minor structural damage. These two facts are not sufficient to confirm the accuracy of the computed structural damages in table 4.2; however, it can be concluded that the computed values for structural damage are consistent with actual observations.

TABLE 4.3

FUNDAMENTAL PERIOD AND AVERAGE PEAK INTERSTORY DISPLACEMENTS FOR TYPICAL BUILDINGS

	0.27g Earthquake		Elastic Limit Earthquake	
Building Design Zone	Fundamental Period (sec)	Average Peak Interstory Displacements (ft)	Fundamental Period (sec)	Average Peak Interstory Displacements (ft)
11-CMRF				
0,1	2.656	0.0872	2.656	0.0385
2	2.656	0.0897	2.656	0.0384
3	2.040	0.0710	2.046	0.0294
4	1.474	0.0496	1.474	0.0206
6-CMRF				
0,1	2.805	0.1493	2.805	0.0743
2	2.805	0.1517	2.805	0.0743
3	2.070	0.1243	2.070	0.0541
4	1.383	0.0773	1.383	0.0349
17-CSW				
0	3.340	0.0452	3.340	0.0293
1	3.340	0.0459	3.340	0.0273
2	2.680	0.0646	2.680	0.0297
3	2.320	0.0556	2.350	0.0212
4	2.120	0.0414	2.120	0.0404
11-CSW				
0,1,2	1.840	0.0570	1.840	0.0280
3	1.368	0.0361	1.368	0.0196
4	1.106	0.0340	1.106	0.0166
11-SMRF				
0,1	3.533	0.0860	3.533	0.0554
2	3.119	0.0820	3.119	0.0538
3	2.396	0.0905	2.396	0.0474
4	1.709	0.0693	1.709	0.0355

In table 4.2, the values for drywall partition damage and the four kinds of brick wall damage are quite large. This is also consistent with observed fact, since there have been many cases in which the damage to the structural frame of a building during an earthquake was minor, but the damage to the brittle nonstructural walls was substantial. It may seem unusual that the computed values of brick wall damage are not drastically decreased when the wall is reinforced with steel. This can result from two factors. It is true that the reinforcing steel makes the wall stiffer and more able to resist lateral loads. However, it may be true that the steel's influence on the damage is felt only at values of displacement greater than those necessary to cause the repair cost of the wall to exceed the initial value of the wall. If this is the case, then the computed damage for a reinforced and unreinforced brick wall would be the same, since the mathematical models for damage are valid only up to the point where the repair cost equals the initial value. Another factor which can explain the insensitivity of brick wall damage to the presence or absence of reinforcing steel is the following. Even for a brick wall with reinforcing steel, there is only a very small range of displacments between zero damage and 100 percent damage. This range is typically less than 0.15 inches (16,17). The range of displacements for an unreinforced wall is always less than the range of displacements for a reinforced wall. The peak interstory displacement of the various stories for a typical building can vary over a rather broad range of values. When the interstory displacements vary over a broad range, the probability that the damage on a given floor will be zero (peak interstory displacement is less than the tolerance used to separate the frame from the infill wall) or 100 percent (peak interstory displacement is greater than the value that would cause the repair cost of the wall to exceed the initial value of the wall) is greater than the probability that the damage would be between 0 and 100 percent. If this is true, then the computation of brick wall damage is very nearly a zero-one type decision, and is not very sensitive to presence or absence of reinforcing steel.

The most significant category of table 4.2 is total damage. It is interesting to compare the total damage to an 11-story concrete moment-resisting frame with the total damage to an 11-story steel moment-resisting frame. When comparing the damage to the 11-CMRF and the 11-SMRF, it must be kept in mind that the steel frame was analyzed in the long direction. This slight inconsistency will not have any effect on the conclusion which is stated below.

Table 4.2 shows that for each seismic design zone (0,1,2,3,4) the 11-CMRF had less total damage than the 11-SMRF. This is an interesting result, since the people who promote steel construction often make use of the fact that no steel building has ever suffered structural damage during an earthquake. The extremely low values of structural damage for the 11-SMRF are consistent with this fact; however, table 4.2 also shows that the damage to nonstructural walls in steel frame buildings may be 10 to 20 percent greater than the damage to similar components in reinforced concrete frame buildings. Therefore, it is concluded that in the choice of structural material for earthquake-resistant design one must consider many other factors in addition to the expected structural damage.

Damage to the concrete shear wall buildings in table 4.2 does not follow the same trend as the damage to frame buildings. The case of the 11-story concrete shear wall building (11-CSW) may seem inconsistent since, as the seismic design zone is increased from zone 0 to zone 4, the structural damage is increased and the damage to nonstructural components is decreased. To understand this trend, it is helpful to know how the designs changed for the different seismic zones. The general strategy used to resist the larger design lateral loads which are generated when designing for the higher seismic zones was to replace an interior frame with an interior shear wall. This is a reasonable approach, and it is consistent with standard design practice. However, from the standpoint of structural damage, the effect of this strategy is to replace a ductile element (frame) by a rather brittle element (shear wall). Therefore, even if replacing a frame with a shear wall reduces the peak displacements of a building during an earthquake, the structural damage may be increased, since shear walls are more susceptible to damage than frames. This appears to be the case with the 11-CSW. The results in table 4.2 show that the effect of adding more shear walls to the 11-CSW reduces the displacements, therefore causing a reduction in nonstructural damage. However, the reduction in displacements is not large enough to overcome the increased susceptibility of the building to structural damage. It is interesting to note that the total damage category is steadily decreased as the seismic design zone is increased for the 11-CSW.

For the case of the 17-story concrete shear wall building (17-CSW), there does not appear to be a constant trend of damage as the seismic design zone is increased. There are at least two factors which are combining to give these random results. The first factor is the increased susceptibility of the higher design zones to structural damage which was explained for the 11-CSW. The second factor which effects these results is the change in fundamental period of the building as the structure is made stiffer. This has been explained in the discussion of the damage patterns observed in the frame buildings. It should also be noted that in the analysis of the 17-CSW, zone 0 and zone 1, the shear walls failed in the lower stories. The implications of a major failure or structural damage are not clear. If a shear wall fails at a given level, the only fact of which one can be certain is that the cost to repair the shear wall at that level is at least equal to the initial value of the shear wall at that level. It is this approach which is used in determining the structural damage to elements which fail. It is probably true that the failure of a shear wall will effect the damage to other building components; however, at this time, it is not certain what is the nature or magnitude of this effect. Therefore, the interaction of damage between elements is not included in this analysis.

In considering the damage to shear wall buildings, the most important information is given in the total damage category in table 4.2 . From this table, it is obvious that increasing the seismic design zone will decrease the total damage for the 11-CSW. For the 11-CSW the reduction in damage between zone 0 and zone 4 is 6.2 percent. It is noted that there is not a consistent change in total damage to the 17-CSW as the design zone is increased. The smaller increase in total damage and the erratic pattern damage are a result of the factors mentioned in the preceding paragraph.

It is interesting to compare the total damage to frame buildings with the total damage to shear wall buildings. As shown in table 4.2, there is not a great deal of difference between the total damage of a frame building and the total damage of a shear wall building. In comparing the damage to the 11-CMRF and the 11-CSW, it is found that the total damage to the 11-CSW zone 3 is slightly less than the total damage to the 11-CMRF zone 3. This is a surprising result, since the structural damage for the 11-CSW is about ten times greater than the structural damage for the 11-CMRF. The total damage to the 11-CSW is slightly less than the total damage to the 11-CMRF because, in both cases, the total damage is strongly influenced by the nonstructural components. The 11-CSW is usually a bit less flexible than the 11-CMRF; therefore the peak interstory displacement and the nonstructural damages to the 11-CSW are slightly less. Since tne major portion of the damage is contributed by the nonstructural elements, it is reasonable that the total damage to the 11-CSW be less than the total damage to the 11-CSW. From this argument it is concluded that, in the choice of an optimum structural system for earthquake-resistant design, one must consider the effect that the structural system will have on the damage to nonstructural components.

Perhaps the most significant conclusion which can be drawn from the investigation of the damage to typical buildings when subjected to the 0.27g earthquake is that the total damage does not change very much when the seismic design zone is changed from zone 0 to zone 4. As previously mentioned, there is a reduction of 6.2 percent when the design zone for the 11-CSW is changed from 0 to 4; however, this building seems to represent an exceptional case. The reason for the small reduction in total damage is the dependency of the total damage on the nonstructural components damage. By increasing the seismic design zone for these typical buildings, the reduction in interstory displacements is not great enough to cause a considerable reduction in the nonstructural components' damage. This topic is discussed further in the following paragraphs.

Table 4.4 contains the results of a second cycle of damage analysis for the same typical building used in table 4.2. For the second analysis the input story displacements are those values which result from subjecting the building to an earthquake which would cause first yielding to occur somewhere in the structure. Therefore, the damages given in table 4.4 represent the maximum damages each building could sustain during the largest earthquake in which the structural response

TABLE 4.4

DAMAGES TO TYPICAL BUILDINGS FOR THE ELASTIC LIMIT EARTHQUAKE

Build-ing --- Zone	Struc-tural Damage	Parti-tion 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 4.5

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

remained within the elastic range. The peak acceleration of the elastic limit earthquake for each building is given in table 4.5.

Since each building in table 4.4 has been subjected to an earthquake of different peak acceleration, it is not possible to draw any conclusions about earthquake damage from the consideration of only these results. However, when the results of table 4.2 are combined with the results given in table 4.4, some meaningful conclusions can be reached.

A comparison of the damage to the 11-CMRF and the 6-CMRF in tables 4.2 and 4.4 shows that there is not a large decrease in total damage for the elastic limit earthquake. This can be explained by the fact that the major source of earthquake damage for both buildings when subjected to both earthquakes is the damage to the nonstructural elements. For the 0.27g earthquake (table 4.2), the peak interstory displacements on most floors are much greater than the interstory displacements which cause the repair cost of the nonstructural items to be equal to the initial value of the nonstructural items. Regardless of how large the peak interstory displacement may become, the maximum nonstructural damage ratio any floor can have is one.

For the elastic limit earthquake (table 4.4), the peak interstory displacements are usually smaller than those generated by the 0.27g earthquake. However, for the 6-CMRF and the 11-CMRF the peak interstory displacements on most of the floors for the elastic limit earthquake are greater than the interstory displacements which are necessary to cause the nonstructural damage ratio to be equal to one. Due to the fact that both earthquakes cause the nonstructural repair cost to equal or exceed the initial value of the nonstructural components on approximately the same number of floors of both the 6-CMRF and the 11-CMRF, the total damage suffered by these buildings is about the same for the elastic limit earthquake and the 0.27g earthquake.

For the steel frame buildings (11-SMRF), the total damage suffered during the elastic limit earthquake (table 4.4) is considerably less than the total damage during the 0.27g earthquake (table 4.2). As was the case with the concrete frame buildings, the damages to nonstructural components can explain this result. To ensure that there would be no yielding of the steel frame, the peak acceleration used in the dynamic analysis of the 11-SMRF was much smaller than 0.27g. Due to the smaller input acceleration, the peak interstory displacements during the elastic limit earthquake were considerably less than those generated during the 0.27g earthquake (see tables 4.3 and 4.5). Smaller interstory displacements caused smaller nonstructural damages for the elastic limit analysis of the 11-SMRF. It has been shown that the major portion of the total damage is contributed by the nonstructural elements. Therefore, by considering the changes in the nonstructural damages for the 11-SMRF shown in table 4.2 and table 4.4, it is logical that there would be a drastic reduction in the total damage for the elastic limit earthquake.

An explanation of the large reduction in damage for the elastic limit earthquake for the 17-CSW and 11-CSW would follow the same argument used for the 11-SMRF. It is evident from table 4.4 that the nonstructural damage during the elastic limit earthquake is quite minor. This is true for all cases except the 17-CSW zone 3 and 17-CSW zone 4 designs. For these two buildings there is substantial structural and nonstructural damage during the elastic limit earthquake. (It is important to note that the elastic limit earthquake is not necessarily the maximum earthquake which causes no structural damage). The reason for this is that the 17-CSW zone 3 design and the 17-CSW zone 4 design require earthquakes with peak accelerations of 0.104g and 0.190g, respectively, to reach the limit of elastic structural behavior. These elastic limit earthquakes are large enough to cause both structural and nonstructural damages to these buildings.

As stated in Chapter 2, the Uniform Building Code requirements are intended to produce buildings which will remain elastic during the many small earthquakes which might occur over the service life of the structure. In this context, the earthquakes which generated the damages shown in table 4.4 are the design earthquakes for the various buildings represented in table 4.4. Therefore, the total damages for the various buildings in this table are the values which might be expected if the buildings were subjected to an earthquake which was equal to the design earthquake. For the purpose of an example, assume that a 17-story shear wall building was designed for the UBC zone 3 requirements. If this building were subjected to an earthquake which was just equal to the UBC interpretation of the design earthquake for zone 3, then the expected total damage to this building would be 7.8 percent. A similar interpretation can be extended to all the buildings shown in table 4.4. In this sense, the total damages shown in table 4.4 are an estimate of the amount of damage which is implied when designing various buildings for the various seismic design zones.

From an examination of table 4.4, one might conclude that the design earthquake for the 11-SMRF zone 3 design causes less damage than the design earthquake for the 11-CMRF zone 3 design. This observation is correct; however, it may be misleading. The peak acceleration of the design earthquake for the 11-CMRF zone 3 design is 0.159g, and the peak acceleration of the design earthquake for the 11-SMRF zone 3 design is 0.05g. Therefore, it is reasonable to conclude that the 11-CMRF zone 3 design is providing more seismic protection than the 11-SMRF, since it must be designed for a larger earthquake. Comparisons of this type can be made for all of the buildings shown in table 4.4. The conclusion from such an exercise would be that by designing a variety of buildings for a particular seismic design zone, one does not provide the same level of seismic protection for each building. This conclusion is helpful in explaining the erratic damage patterns which were noticed in the study of damage to buildings during past earthquakes. For example, in the study of the 1971 San Fernando earthquake, there were cases in which several buildings that were designed according to the current U.B.C. zone 3 regulations and located in the same area of Los Angeles suffered very dissimilar patterns of damage due to the earthquake. It is now clear that the dissimilar damages suffered by these seemingly similar buildings is due to the fact that each building had been designed for a different level of seismic protection.

The final phase of the analysis of the typical buildings was to estimate the damages to all of the buildings when subjected to an earthquake of 0.05g peak acceleration. The component damages for this case are shown in table 4.6. The component damages will not be discussed here since this would serve only to restate the conclusions which were drawn from the elastic limit earthquake and the 0.27g earthquake. The total damages for the 0.05g earthquake, along with the total damages for the other two earthquakes, are shown in table 4.7. For the 0.27g earthquake, it was concluded that the total damage was not very sensitive to the seismic design zone. Clearly, this is not the case for the frame buildings when subjected to the 0.05g earthquake. Table 4.7 shows that, for the 0.05g earthquake, the total damage can be considerably less for a zone 4 design than it is for a zone 0 design. It is reasonable to assume that in the lifetime of a typical building, several earthquakes with a peak acceleration of 0.05g might occur. However, it is unlikely that a bulding would experience an earthquake with peak acceleration of 0.27g more than once in its service life. If one wishes to choose an optimum seismic design zone, it would be more appropriate to consider the damages during the smaller earthquake. For the smaller earthquake (0.05g), it is again noted

TABLE 4.6

DAMAGE TO TYPICAL BUILDINGS - 0.005g 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 4.7

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

that the level of seismic design does have a significant effect on the total damage.

This concludes the discussion of the results of applying the mathematical models for earthquake damage to typical buildings. The conclusions which can be drawn from this exercise, along with some pertinent observations from other portions of this study, are summarized in Chapter 5.

CHAPTER 5. CONCLUSIONS

The work reported in this paper can be logically divided into three sections. These sections are: the study of the performance of tall buildings during past earthquakes, the development of mathematical models for earthquake damage, and the application of the damage models to buildings. Although these topics are rather closely related, it is more convenient to discuss the conclusions reached in each portion of the work separately.

5.1 Performance of Tall Buildings in Past Earthquakes

The study of the performance of buildings during past earthquakes is the foundation for all of the work presented in this report. Although this study did not lead to a method for predicting earthquake damage, the insights to the problem gained through this exercise were invaluable in the formulation of the mathematical models for earthquake damage.

In this portion of the study, the difficulty of the earthquake damage problem was first realized. It was found that the problem of predicting earthquake damage had never before been attempted in a comprehensive manner. Historical data on the damage to buildings during past earthquakes is rather scarce and incomplete. The data which was obtained was full of inconsistencies which could not be explained with the existing knowledge of the mechanisms of earthquake damage. It was therefore concluded that an acceptable method for the prediction of earthquake damage could not be developed by utilizing only data from historical earthquakes.

In the course of the study of historical earthquakes, it was possible to identify several factors which influence the nature of earthquake damage to buildings. It was found that the damage to buildings is often more directly influenced by the details of the design and construction than it is by the strength of the structural frame. For many of the buildings studied, the damage to nonstructural elements was far greater than the damage to the structural systems. It was also found that building components which were orientated vertically were heavily damaged during earthquakes. From a conceptual argument it was decided that peak interstory displacement is the best available parameter to use for the prediction of damage to vertical elements.

Finally, the study of historical earthquakes led to the conclusion that an exact prediction of earthquake damage to buildings is not feasible at this time. A serious consideration of the uncertainties in the response of buildings to ground motion and the inconsistencies in the historical data lead to the conclusion that any model which attempts to predict earthquake damage to buildings can give only approximate results.

5.2 Development of Earthquake Damage Models

In the course of this study, the development of a mathematical model for the prediction of earthquake damage was attempted by various methods. Not all of the methods which were attempted are discussed in this report. The method which provided the most logical, consistent, and accurate results is based upon an energy formulation. Concisely stated, the method estimates earthquake damage to a particular component by comparing the amount of energy which the component is required to absorb during the period of peak building response, to the total energy absorption capacity of the component. In the process of applying this general method to particular building components, it became necessary to make some rather broad assumptions about the behavior of these components. These assumptions, combined with the approximations which are necessary to apply the damage models to buildings, probably introduce some error into the results which are presented in Chapter 4. However, considering the infantile nature of the state-of-the art of earthquake damage prediction, the assumptions and approximations which were made must be accepted, at least for the present time.

The energy formulation of the earthquake damage problem can logically be applied to estimate the damage to different structural systems. The systems to which the method was applied are: concrete frames, steel frames, steel-braced frames and concrete shear walls. It was found that the method could be extended to some nonstructural elements; however, for other nonstructural elements, the energy methods were not applicable. When this was the case, algorithms for the prediction of damage to these elements were developed by an approximate correlation between the damages to these elements and the building's maximum dynamic response.

It is significant to point out one serious limitation of the mathematical models for damage which are developed in this report. This limitation is that the damage models are only valid up to the point where the repair cost of an element equals the initial value of the element. It is realized that the thorough repair of a damaged element may result in a repair cost which is several times the initial value of the element. However, in some cases, it may be economically more attractive for the building owner to perform repairs which are only of a cosmetic nature. On the other hand, earthquake damage may encourage a building owner to initiate a repair program which will make the building stronger and more attractive than it was prior to the damage. In either case, the actual repair cost is very dependent upon how much money the building owner is willing to spend. For the development of a mathematical model for the prediction of earthquake damage, it was necessary to determine a realistic estimate of the amount of money a typical building owner would spend when faced with a given degree of physical damage. It has been assumed that a typical building owner will be willing to repair an element only if the repair cost is less than the initial value. If the repair cost exceeds the initial value, it is assumed that the owner will replace the damaged element with one of equal value. For these assumptions, the actual amount of money spent to repair or replace a given building element will never exceed the initial value of that element.

5.3 Application of Damage Models to Typical Buildings

The validity of the mathematical models for earthquake damage was tested by applying them to several buildings which were damaged during the 1971 San Fernando earthquake. It was found that the general trends of damage were predicted by the models; however, the models may give specific values of damage which are somewhat different from the actual damages suffered during the earthquake. For example, for a particular building, the structural damage which was reported after the earthquake was 0.8 percent. An application of the structural damage models to this building resulted in a computed damage of 2 percent. Although there is considerable disagreement between the reported and computed values, it is significant to note that both the computed and the reported values are quite small. This is specifically what is referred to by stating that the damage models can predict the general trends of earthquake damage. Considering the amount of uncertainty inherent in the reported values of earthquake damage and the general agreement between the reported and the computed values, it was concluded that the mathematical models for earthquake damage provide a reasonable and consistent approach for the estimating of earthquake damage to real buildings.

It is not reasonable to assume that the mathematical models for damage are equally accurate for all levels of earthquake intensity. For very small or very large earthquakes, there is more uncertainty associated with the estimation of damage. For the case of very small earthquakes, the assumptions regarding the response of the building and the behavior of various components may not be valid. Also, when the earthquake intensity is quite small, the physical damage to buildings is quite dependent on the function of the building. On the other hand, very large earthquakes may cause the building to approach the point of total collapse. As previously stated, a major structural failure may effect the damage to the nonstructural components. However, the implications of a major failure on damage are not clear, and the possibility of interaction between damaged components is not accounted for in the mathematical models. It is therefore concluded that the mathematical models for damage are most accurate when applied for earthquakes of moderate intensities.*

The final portion of this study was the application of the mathematical models for earthquake damage to a number of hypothetical buildings which might be constructed in the metropolitan Boston area. These buildings had been designed in accordance with the Uniform Building Code regulations for various levels of seismic protection, and they were subjected to three artificial earthquakes of different peak accelerations. It was found that the damages to these buildings as estimated by the mathematical models are consistent with the damages reported for similar buildings during similar earthquakes. For most cases, it was found that the damage to nonstructural elements was far greater than the damage to the structural system.

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* By moderate intensities the author is referring to Modified Mercalli Intensities 6, 7, and 8.

It was found that, for an earthquake with a peak ground acceleration of 0.27g, the damage to typical buildings was not substantially reduced by increasing the seismic design zone. For an earthquake with a peak ground acceleration of 0.05g, the damage to typical buildings was usually considerably less for a zone 4 design than it was for a zone 0 design. Since the smaller earthquake may occur several times during the service life of the building, it is concluded that there is some benefit to be gained by designing for larger lateral forces.

It was found that the structural damage to shear walls is greater than the structural damage to moment-resisting frames. However, if the addition of shear walls to a building causes a significant reduction in the building's response, this will cause a reduction in the damage to the nonstructural elements. The net result may be that a shear-wall building will have smaller total damage than similar frame buildings during the same earthquake.

The final observation which is made in Chapter 4 is that designing various buildings for the same seismic design zone does not necessarily provide the same level of seismic protection for each building. This unexpected result may provide a partial explanation of the erratic damage patterns which were noticed in the damge to buildings during past earthquakes.

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APPENDIX A

A.1 DERIVATION OF EXPRESSIONS FOR AREA UNDER STRESS - STRAIN CURVES

A.1.1 DUCTILE STEEL

The expressions for the stress-strain relationship for ductile steel are: (SEE Fig. 3.12 A or REF. 15)

Region AB - $f_s = E_s \epsilon_s$

Region BC - $f_s = f_y$ A-1

Region CD -

$$f_s = f_y \left\{ \frac{m(\epsilon_s - \epsilon_{sH}) + 2}{60(\epsilon_s - \epsilon_{sH}) + 2} + \frac{(\epsilon_s - \epsilon_{sH})(60 - m)}{2(30r + 1)^2} \right\} \qquad \text{A-2}$$

where

$$m = \frac{\frac{f_{su}}{f_y}(30r + 1)^2 - 60r - 1}{15r^2}$$

$$r = \epsilon_{su} - \epsilon_{sH}$$

The total area under the curve from B to D is:

$$A_T = \int_{\epsilon_y}^{\epsilon_{sH}} f_y \, d\epsilon_s + \int_{\epsilon_{sH}}^{\epsilon_u} f_y \left\{ \frac{m(\epsilon_s - \epsilon_{sH}) + 2}{60(\epsilon_s - \epsilon_{sH}) + 2} + \frac{(\epsilon_s - \epsilon_{sH})(60 - m)}{2(30r + 1)^2} \right\} d\epsilon_s$$

$$A_T = \left\{ f_y \left[(\epsilon_{sH} - \epsilon_y) + \frac{m}{60}(\epsilon_{su} - \epsilon_{sH}) + \left[\frac{-m(60\epsilon_{sH} - 2)}{3600} \right] \left\{ \text{Ln}\left(\frac{2}{m}\right) - \text{Ln}\left[\frac{60\epsilon_{su} - 60\epsilon_{sH} + 2}{m} \right] \right\} - \frac{m(\epsilon_{sH} - 2)}{60} \left[\text{Ln}(60\epsilon_{su} - 60\epsilon_{sH} + 2) - \text{Ln}(2) \right] + \frac{60 - m}{2(30r+1)^2} \left[\frac{\epsilon_{su}^2}{2} + \epsilon_{su}\epsilon_{sH} - \frac{3\epsilon_{sH}^2}{2} \right] \right] \right\} \qquad \text{A-3}$$

Area under the curve from point B to a strain ϵ_x

1) If $\epsilon_x < \epsilon_{SH}$

$$A_i = \int_{\epsilon_Y}^{\epsilon_x} f_y \, d\epsilon = f_y \left[\epsilon_Y - \epsilon_Y \right] \qquad \text{A-4}$$

2) If $\epsilon_{SH} \leq \epsilon_x \leq \epsilon_{SU}$

$$A_i = \int_{\epsilon_Y}^{\epsilon_{SH}} f_y \, d\epsilon + \int_{\epsilon_{SH}}^{\epsilon_x} f_y \left\{ \frac{m(-\epsilon_{SH} + \epsilon_x) + 2}{60(-\epsilon_{SH} + \epsilon_x) + 2} \qquad \frac{(\epsilon_x - \epsilon_{SH})(60 - m)}{2(30r+1)^2} \right\} d\epsilon$$

$$A_i = f_y \left[\epsilon_{SH} - \epsilon_Y + \frac{m}{60}\left(\epsilon_x - \epsilon_{SH}\right) + \left[\frac{-m(60\epsilon_{SH} + 2)}{3600} \right] \left\{ \mathrm{Ln}\left(\frac{2}{m}\right) - \right.\right.$$

$$\left. - \mathrm{Ln}\left[\frac{60\epsilon_x - 60\epsilon_{SH} + 2}{m} \right] - \frac{m(\epsilon_{SH} - 2)}{60} \left[\mathrm{Ln}\left(60\epsilon_x - 60\epsilon_{SH} + 2\right) - \mathrm{Ln}(2) \right] \right\}$$

$$\left. + \frac{60 - m}{2(30r+1)^2} \left[\frac{\epsilon_x^2}{2} + \epsilon_x \epsilon_{SH} - \frac{3\epsilon_{SH}^2}{2} \right] \right] \qquad \text{A-5}$$

A.1.2 Confined Concrete

The expressions for the stress - strain relationship for confined concrete are: (see fig. 3.12 B. or Ref 15)

Region AB

$$f_c = f'_c \left\{ \frac{2\epsilon_c}{0.002} - \left(\frac{\epsilon_c}{0.002}\right)^2 \right\} \qquad \text{A-6}$$

Region BC

$$f_c = f'_c \left\{ 1 - Z(\epsilon_c - 0.002) \right\} \qquad \text{A-7}$$

Where:

$$Z = 0.5 / (\epsilon_{50} - 0.002)$$

ϵ_{50} is the strain in Region BC at $0.5f'_c$

$$\epsilon_{50} = \frac{3}{4} \rho_s \sqrt{\frac{b''}{S_h}} + \frac{3 + 0.002f'_c}{f'_c - 1000}$$

b'' = Width of Confined Core = (Total Width - 3") = W - 3"
S_h = Spacing of hoops or spirals = 4"
ρ_s = Ratio of hoop volume to core volume = $\dfrac{0.12 f'_c}{f_y}$

$$\epsilon_{50} = 0.45 \frac{f'_c}{f_y} \sqrt{\frac{W-3}{4}} + \frac{3 + 0.002f'_c}{f'_c - 1000}$$

For damage computations need strain in region AB which corresponds to $0.45f'_c$

$$0.45 \cancel{f'_c} = \cancel{f'_c} \left[\frac{2}{2\times10^{-3}} \epsilon_s - \frac{1}{4\times10^{-6}} \epsilon_c^2 \right]$$

$$0.45 = 10^3 \epsilon_c - 25\times10^4 \epsilon_c^2$$

$$\epsilon_c^2 - 40\times10^{-4}\epsilon_c + 4\times10^{-6} = -1.8\times10^{-6} + 4\times10^{-6}$$

$$\epsilon_c = 0.52\times10^{-3} \leftarrow \text{strain at } 0.45f'_c$$

The strain at failure is that value in region BC at which the concrete stress is $0.2 f'_c$

ϵ_{20} = Strain at $0.2 f'_c$ in region BC

$$0.2 \cancel{f'_c} = \cancel{f'_c}\left[1 - Z(\epsilon_{20} - 0.002)\right]$$

$$\epsilon_{20} = \frac{1}{Z}\left[0.8 + 0.002 Z\right]$$

Find the total area under the curve from 0.52×10^{-3} to ϵ_{20}

$$A_T = f'_c \int_{0.00052}^{0.002} (10^3 \epsilon_c - 2.5 \times 10^4 \epsilon_c^2)\, d\epsilon_c + \int_{0.002}^{\epsilon_{20}} f'_c(1 - Z\epsilon_c + 0.002 Z)\, d\epsilon_c$$

$$A_T = f'_c\left[0.00179 + (\epsilon_{20} - 0.002)(1 + 0.002Z) - \frac{Z}{2}\left(\epsilon_{20}^2 - 0.002^2\right)\right] \qquad \text{A-8}$$

Find the area under the curve from 0.52×10^{-3} to any strain ϵ_x.

If $\epsilon_x < 0.002$

$$A_i = f'_c\left[500 \epsilon_x^2 - 8330 \epsilon_x^3 - 0.000134\right] \qquad \text{A-9}$$

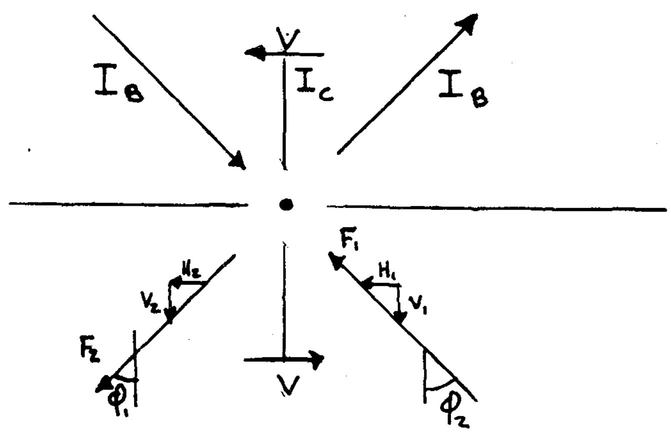
If $0.002 < \epsilon_x < \epsilon_{20}$

$$A_i = f'_c\left[0.000179 + (\epsilon_x - 0.002)(1 + 0.002Z) - \frac{Z}{2}\left(\epsilon_x^2 - 0.002^2\right)\right] \qquad \text{A-10}$$

A.2 DEFLECTION NECESSARY TO BUCKLE THE BRACING in A BRACED FRAME

Assumptions
1) All columns in A story deflect the same amount
2) Brace is pinned At ends
3) Brace buckles elastically
4) After brace buckles building behaves like shear beam

Forces on A Joint OF THE FRAME



$V_1 = F_1 \cos\phi_1$
$H_1 = F_1 \sin\phi_1$

$V_2 = F_2 \cos\phi_2$
$H_2 = F_2 \cos\phi_2$

Equations of Equilibrium for bottom Half of Joint

$$H_1 + H_2 = V \qquad (1)$$

$$V_1 = -V_2 \qquad (2)$$

$$F_1 \cos\phi_1 = F_2 \cos\phi_2 \longrightarrow F_2 = F_1 \frac{\cos\phi_1}{\cos\phi_2}$$

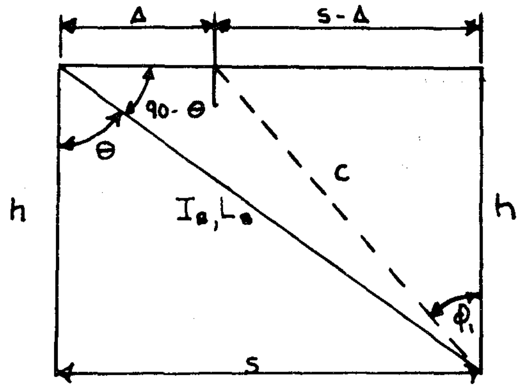
$$F_1 \sin\phi_1 + F_2 \sin\phi_2 = V_2$$

$$F_1 \left[\sin\phi_1 + \frac{\cos\phi_1}{\cos\phi_2} \sin\phi_2 \right] = V$$

Solving for F_1

$$F_1 = \left[\frac{V}{\sin\phi_1 + \cos\phi_1 \tan\phi_2} \right] \longleftarrow \text{A-11}$$

From the deflected geometry solve for $\sin\phi_1$ & $\cos\phi_1$



$$\sin\phi_1 = \frac{S-\Delta}{C}$$

$$\cos\phi_1 = \frac{h}{C}$$

Solve for C from Law of Cosines

$$C^2 = L_B^2 + \Delta^2 - 2L_B\Delta\cos(90-\theta)$$

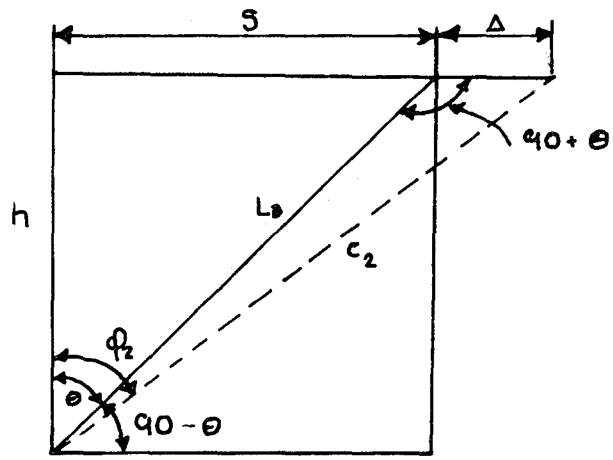
$$\cos(90-\theta) = \sin\theta = \frac{S}{L_B}$$

$$C^2 = L_B^2 + \Delta^2 - 2L_B\Delta\frac{S}{L_B} = L_B^2 + \Delta^2 - 2\Delta S$$

neglect Δ^2 since it is very small compared to L_B^2

$$C^2 = L_B^2 - 2\Delta S \qquad C = \sqrt{L_B^2 - 2\Delta S} \qquad \text{A-12}$$

From the deflected geometry of the adjacent bay solve for $\tan\phi_2$



$$\tan\phi_2 = \frac{S+\Delta}{h}$$

The force in a brace due to the shear V

$$F = \left[\frac{V}{\sin\phi_1 + \cos\phi_1 \tan\phi_2}\right] \qquad \text{A-11}$$

$$\sin\phi_1 + \cos\phi_1 \tan\phi_2 = \frac{S-\Delta}{C} + \frac{\cancel{H}}{C}\cdot\frac{S+\Delta}{\cancel{H}}$$

$$= \frac{S-\Delta}{C} + \frac{S+\Delta}{C}$$

$$= \frac{2S}{C}$$

$$F = \frac{CV}{2S}$$

At the moment the brace buckles, the frame will behave as a shear beam, and the shear in the column as a function of the displacement Δ is:

$$V = \frac{12\,E I_c \Delta}{h^3} \qquad \text{A-13}$$

There force The force in the brace is

$$F_1 = \frac{6\,E I_c \Delta C}{h^3 S} \qquad \text{A-14}$$

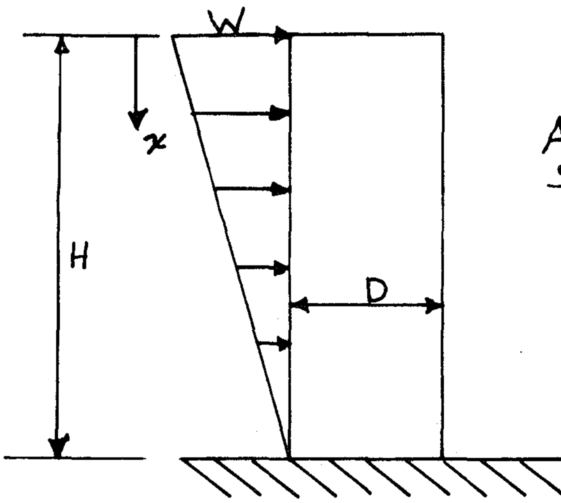
The critical load for the brace is:

$$F_{CR} = \frac{\pi^2 E I_B}{L_B^2} \qquad \text{A-15}$$

When the deflection Δ causes the force in the brace, F_1, to exceed F_{CR} the brace will buckle

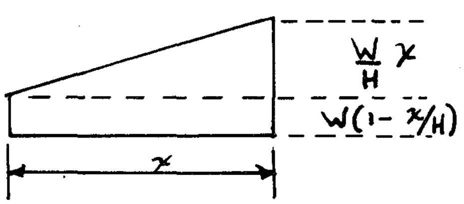
$$\Delta_{cr} = \frac{\pi^2 E I_B h^3 S}{6 E I_c C L_B^2} \qquad \text{A-16}$$

A.3 Shear Wall - Bending Strain at Any Height



Assumed Lateral Loading on Shear Wall.

Derive an expression for Bending Moment as a function of x



$$M_x = \frac{1}{2} x \frac{Wx}{H} \frac{2x}{3} + x W(1 - x/H) \cdot x/2$$

$$M_x = \frac{Wx^3}{3H} + \frac{Wx^2}{2} - \frac{Wx^3}{2H}$$

$$M_x = \frac{Wx^2}{2} - \frac{Wx^3}{6H}$$

Find the deflection as a function of x

$$M_x = EI\Delta'' = \frac{Wx^2}{2} - \frac{Wx^3}{6H}$$

Intergrate twice to get:

$$EI\Delta = \frac{Wx^4}{24} - \frac{Wx^5}{120H} + C_1 x + C_2$$

From the boundry Conditions:

$$C_1 = -\frac{1}{8}WH^3 \qquad C_2 = \frac{11WH^4}{120}$$

Therefore

$$EI\Delta = \frac{Wx^4}{24} - \frac{Wx^5}{120H} - \frac{WH^3}{8}x + \frac{11WH^4}{120}$$

Providing $x \neq H$ in this expression, solve for W

$$W = \frac{EI\Delta}{\left(\frac{x^4}{24} - \frac{x^5}{120H} - \frac{H^3x}{8} + \frac{11H^4}{120}\right)} \qquad (1)$$

From Elastic Beam Theory

$$\frac{M_x}{EI} = \frac{\epsilon_x}{Y} \quad (2) \quad (Y = \text{distance to Neutral Axis})$$

For the Shear Wall And Assumed loading

$$\frac{M_x}{EI} = \frac{W}{EI}\left(\frac{x^2}{2} - \frac{x^3}{6H}\right) \qquad (3)$$

Combining (1), (2) and (3)

$$\frac{M}{EI} = \frac{\epsilon_x}{Y} = \frac{1}{EI}\,\frac{EI\Delta\left(x^2/2 - x^3/6H\right)}{\left(\frac{x^4}{24} - \frac{x^5}{120H} - \frac{H^3x}{8} + \frac{11H^4}{120}\right)}$$

Approximately $Y = D/2$, Therefore:

$$\epsilon_x = \frac{\frac{\Delta D}{2}\left(\frac{x^2}{2} - \frac{x^3}{6H}\right)}{\left(\frac{x^4}{24} - \frac{x^5}{120H} - \frac{xH^3}{8} + \frac{11H^4}{120}\right)}$$