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6 A SEISMIC RISK SIMULATION MODEL
FOR ARMY FACILITIES: PHASE ONE,
DEVELOPMENT OF DETERMINISTIC MODEL.

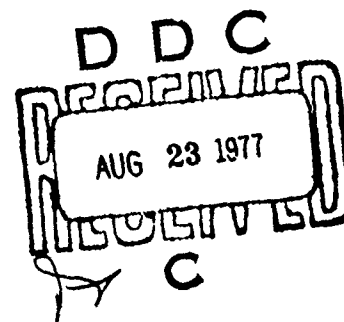
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by
10 Ronald G. Merritt



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strengthening or replacing the facility to mitigate the effects of seismic activity. The cost to repair damage and the costs to strengthen and replace four-, seven-, and ten-story facilities are presented for three basic building configurations and a continuum of loads ranging from the the 1968 Structural Engineers Association of California Zone 3 requirements[★] to a 1.0 g response spectrum. An example for use of the model is given, along with two decision methodologies. ★

★ *Recommended Lateral Force Requirements and Commentary* (Seismology Committee, Structural Engineers Association of California [SEAOC], 1968).

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FOREWORD

This investigation was performed by the Structural Mechanics Branch (MSS), Materials and Science Division (MS), U.S. Army Construction Engineering Research Laboratory (CERL) under ILIR project A91D 04 048, "Seismic Risk: Balancing Future Loss vs. Additional Cost." The applicable QCR is 1.04.004. Dr. R. G. Merritt was the Principal Investigator.

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Dr. W. E. Fisher is Chief of MSS, and Dr. G. R. Williamson is Chief of MS. COL J. E. Hays is Commander and Director of CERL, and Dr. L. R. Shaffer is Technical Director.

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A SEISMIC RISK SIMULATION MODEL FOR ARMY
FACILITIES: PHASE ONE, DEVELOPMENT OF
DETERMINISTIC MODEL

1 INTRODUCTION

Background

Because the lateral-force-resisting systems of existing Army facilities were designed for seismic loads significantly less than current requirements, such facilities may provide inadequate protection from seismic loads defined in current specifications and thus may pose a hazard to life and property. To assess this hazard, existing facilities must be evaluated for seismic risk. The evaluation must provide the information needed to determine what action to take to mitigate the potential hazard.

Three major courses of action may be considered: the existing facility may be (1) left unchanged, with damage resulting from seismic loads being repaired at some future date, (2) strengthened to meet current specifications, or (3) replaced with a facility meeting current design specifications. Thus, the problem becomes one of providing information that will assist in making a rational decision relative to these three courses of action.

Purpose

The purpose of this study is to develop a decision tool for assessing (1) the seismic hazard to Army facilities and (2) the cost of mitigation schemes for reducing the hazard. This report describes the first phase of the study, in which a computer simulation model was developed. The model simulates the design of a facility, predicts the damage to be expected for a given level of seismic activity, and develops cost information for the three decision alternatives defined above. Future phases of development will contain refinements that will improve the model's sensitivity and provide additional decision information.

Approach

Seismic Risk Analysis Overview

Underlying the process of providing the required decision information is the technology of seismic risk analysis. Seismic risk analysis is a set of activities that results in determination of the probability of occurrence of a certain level of seismic activity at a particular place and time. The risk analysis overview presented in this section is tailored to the U.S. Army Corps of Engineers construction program and is in part taken from notes provided to the Corps by H. C. Shah of Stanford University.¹

The assessment of risk to an existing facility from seismic activity can be considered in two parts. The "expected seismic activity" at the facility site is determined first, followed by the effects of this expected seismic activity on the facility. The information from these two steps is used in a decision methodology to provide guidance on either accepting or reducing the risk.

Determining the expected seismic activity at a site requires that a site risk analysis be performed. In simplest terms, the site risk analysis involves modeling historical seismic data for the region to derive expressions for the future expected seismic activity. Shah identifies four basic steps in determining the seismicity of a site:

1. Determination of the location of seismic sources by combining seismological (e.g., attenuation) and geological (e.g., location and description of faults) information for a selected area surrounding the site with information on past seismic events affecting the site. These data provide basic qualitative information on the susceptibility of the site to seismic activity.

2. Description of the seismicity of the sources. Included in this step are the models for forecasting magnitude, intensity,

¹ H. C. Shah, "Seismic Risk Analysis," Notes for course given to Corps of Engineers, August 11-15, 1975, San Francisco, CA.

duration, and peak ground acceleration at a site based on the information from the first step. These models are usually simple probabilistic models that construct histograms for the site based on past data and attenuation relationships. A computer program that will perform the histogram calculations based on a simple Poisson probabilistic model is available.²

3. Careful study of the microcharacteristics of the site, including characteristics of the soil-structure-foundation interaction. Although this interaction is not well understood, it is often of critical importance.

4. Assembling data to provide information on peak ground acceleration at the site, and duration and number of occurrences of a particular level of acceleration. For any computerized seismic simulation model, knowledge of the peak ground acceleration is essential. Knowledge of the number of occurrences is essential for the decision methodology, if other than a deterministic approach to seismic risk assessment is taken.

The second part of the risk assessment problem considers the effects of seismic activity on the facility. This study takes an analytical approach to prediction of damage--perhaps the most straightforward approach for those inexperienced in seismic hazard evaluation. It is based on elastic code design of the facility;³ i.e., nonlinear effects are neglected. Structural behavior under code-defined loading is used to predict the damage to be expected from a given level of seismic activity. The analysis model is a simple story stiffness type. The total simulation model goes beyond damage prediction and provides guidance for upgrading needed to bring the facility within acceptable risk limits.

² H. C. Shah.

³ *Seismic Design for Buildings*, TM 5-809-10 (Department of the Army, April 1973); and *Recommended Lateral Force Requirements and Commentary* (Seismology Committee, Structural Engineers Association of California [SEAOC], 1968).

Simulation Model Requirements

Model simulation is a commonly used technique in decision methodology. Construction of a simulation model requires that the assumptions be clearly presented and that all factors of significance be accounted for. Once the model is constructed, it must be capable of transforming the facility configuration into information for use in deciding whether to repair the facility after an earthquake, to strengthen the facility, or to replace it. Facility configuration input must be minimal; only information pertinent to the decision should be required. The model must provide information within a short time frame and be flexible enough to be reused at minimal cost whenever new requirements are proposed. All facilities must be compared on the same basis, and differences in decision information must result from differences in the facility configurations.

The model described in this report was developed to meet the above requirements.

2 SEISMIC RISK SIMULATION MODEL

The seismic risk simulation model translates the configuration of a given facility into information pertinent to assessing the risk to the facility from seismic loads. The model analyzes four- to ten-story-high reinforced concrete structures having a reasonably symmetric configuration. The model consists of four parts, which can be referred to as input, design, analysis, and cost tabulation. Appendix A describes the model in detail, while this chapter defines its scope and limitations.

Input

Input to the model includes:

1. Number of stories
2. Story height (assumed same for all floors)
3. Story weight
4. Number of bays in each direction
5. Ratios of moments of inertia of shear wall to frame columns over the building height
6. Overturning moment factor
7. Floor area per column
8. Total construction cost per square foot for structural elements
9. Total construction cost per square foot for nonstructural elements
10. Structure loads in terms of code variables
11. Convergence option
12. Calibration constants.

For an existing facility, most of the information in items 1 through 10 is readily available. Except for story height and structural and nonstructural construction costs, all variables are story-dependent, allowing for flexibility in the specification of the structural configuration. A static analysis is performed on the

elastic model, and loads on the structure are defined in terms of the code used to design the facility.

Chapter 3 describes the procedure for determining the calibration constants, which correlate cost information with the variables related to the structure's behavior.

Design

The model simulates the design of the structure; that is, the program sizes the column elements and shear wall elements so that their resistance meets or exceeds the seismic code requirements. The simulated design, which only approximates a detailed design, is necessary to determine parameters to strengthen and replace the facility. Since a priori rationales for proceeding from the existing facility to a strengthened or suitable replacement facility do not exist, simulation of the facility design is accomplished using the following algorithm. An initial frame column depth is assumed. From this, the stiffness matrix for the frame is generated. If shear walls are to be included in the configuration, the stiffness matrix for the shear wall elements is also determined. The frame and shear wall stiffness matrices are added, the matrix inverted, and story displacements determined along with the moments and forces in the frame members.

If the member moments and forces exceed the capacity of the element, the size of the element (and thus its capacity) is increased; if the moments and forces within the frame element are less than the capacity of the element, the size of the element may be decreased, depending on the convergence option. This process is repeated until the member forces are less than the member capacity. Convergence for a frame structure is obtained within a few iterations. If shear walls are included and if convergence is not obtained within a specified number of iterations, the design is assumed not to be adequate.

Factors entering into the design include dead load, live load, seismic load, and overturning moment. To simplify the design procedure, the floor diaphragms and their accompanying beams and girders

are assumed infinitely rigid, and columns are assumed to have fixed ends.

Analysis

The analysis portion of the model determines the member forces from the computed story displacements. The member forces are based on second-order displacement difference expressions.

Cost Tabulations

The model also provides cost computations. The following paragraphs provide an overview of the three cost algorithms in the model. Whenever cost is referred to, it is assumed that the cost is the original construction cost for the structure.

The damage/repair algorithm is based on calculation of a damage factor. From Blume's work,⁴ the reserve energy of a structure is equated to the area under a hypothetical elastoplastic load deflection. This curve provides an expression for ductility in terms of the demand placed on the structure relative to its capacity to resist that demand. From this expression, a damage factor which is a measure of the reserve capacity of a structure may be computed. A portion of the structural and nonstructural damage resulting from seismic loading greater than the facility design level is assumed proportional to the damage factor.

It is generally accepted that nonstructural damage is related to factors other than the overall damage factor. One such factor is relative story displacement; i.e., increased relative story displacement correlates positively with increased nonstructural damage. An additional expression for nonstructural damage is therefore included in the model.

Thus, the total cost to repair the structure is the sum of the general structural and nonstructural damage factor for the overall structure and the nonstructural damage factors for the individual

⁴ *Effects Prediction Guidelines for Structures Subjected to Ground Motion*, JAB-99-115, UC-11 (URS/John A. Blume and Associates, Engineers, July 1975).

stories. Although it is recognized that other factors, such as structure torsion and foundation characteristics, may also contribute to substantial structural and nonstructural damage, these factors are not accounted for in this initial model.

The strengthening algorithm considers the difference in stiffnesses between the designs simulated for the current requirements and the requirements under which the facility was designed. As a criterion for strengthening, the story displacements of the strengthened design must be less than or equal to those of the initial design. When shear walls are used to strengthen the structure, the diagonal terms of the stiffness matrix are representative of relative story stiffnesses and are used to compute the strengthening costs. For nonshear wall cases, the stiffness matrix to strengthen the structure is generated initially by taking 10 percent of the initial frame design stiffness matrix. The total strengthening cost for both the structural and nonstructural elements is a function of the difference between the stiffness of the existing facility and that of the simulated design of the strengthened facility.

The replacement algorithm measures the increase in material needed in the simulated design for higher requirements. The column area and the percentages of shear wall steel difference provide a measure of the increase in material costs due to replacement. The structural costs are considered proportional to these ratios. The nonstructural costs, which are not likely to increase under replacement, are considered proportional to a calibration factor times the original nonstructural costs.

3 MODEL CALIBRATION

Model calibration is the process of determining unknown model constants which insure that the information produced by the model agrees with available data. The process is similar to curve fitting. The constants for the model were determined such that damage/repair, strengthening, and replacement costs can be determined for four-through ten-story reinforced concrete facilities with a variety of configurations for a variety of levels of seismic load.

Table 1 gives the matrix of building configurations under consideration. The building configurations are defined in terms of three parameters--structural system, dimensional ratio, and ratio of shear wall to frame moments of inertia. In this study, the shear wall moment of inertia is held constant over the story levels, with the ratio of wall moment of inertia to frame moment of inertia being computed at the first story. An actual facility is transformed into a story stiffness model by specifying the ratio of wall to frame stiffness for the first story, and the relative frame stiffness at stories above the first. The structural systems range from a frame (K equals 0.67), to a light shear wall (K equals 0.80), to a massive shear wall (K equals 1.33); the ratio of the shear wall moment of inertia to the frame moment of inertia varies from 0 to 5,000. The dimensional ratio pertains to the overturning moment factor in the model. The larger the dimensional ratio, the wider the building in the direction of the load. The floor and roof weights used in the simulation are typical of industrial buildings. For computation of cost, 14 configurations and a continuum of loads ranging from 1968 Structural Engineers Association of California (SEAOC) Zone 3 requirements to a 1.0 g response spectrum ⁵ were considered.

In the calibration of the model, data pertinent to the building configurations in Table 1 and the results of initial runs are

⁵ W. K. Stockdale, *Modal Analysis Methods in Seismic Design for Buildings*, Technical Report N-132/ADA012732 (U.S. Army Construction Engineering Research Laboratory [CERL], June 1975).

Table 1

Model Reinforced Concrete Building Facility Configurations

<u>Configuration Number</u>	<u>Structural System</u>	<u>Dimensional Ratio</u>	Ratio of Shear Wall Moments of Inertia to Frame Moment of Inertia (IW/IF)
1	K=0.67	1.0	0
2	K=0.67	1.6	0
3	K=0.80	1.0	100
4	K=0.80	1.0	200
5	K=0.80	1.0	500
6	K=0.80	1.6	100
7	K=0.80	1.6	200
8	K=0.80	1.6	500
9	K=1.33	1.0	1000
10	K=1.33	1.0	2000
11	K=1.33	1.0	5000
12	K=1.33	1.6	1000
13	K=1.33	1.6	2000
14	K=1.33	1.6	5000

performance of the model with arbitrary calibration constants were used to determine a final set of model calibration constants applicable to all configurations for the above loading range. Appendix B presents the details of the calibration process along with the supporting data.

A difficulty experienced in calibration of the model was that the data available for calibration are inadequate for the degree of detail incorporated into the model. The damage study of the San Fernando earthquake performed at the Massachusetts Institute of Technology (MIT),⁶ which provided much of the supporting data, does not consider the structural system type. Strengthening data are sparse and keyed to critical facilities. Replacement data are generally available only for moderately low seismic levels; extrapolation to the levels considered in this report may be misleading.

⁶ R. V. Whitman et al., *Damage Statistics for High-Rise Buildings in the Vicinity of the San Fernando Earthquake*, R73-24, Report No. 7, Structures Publication 363 (Department of Civil Engineering, Massachusetts Institute of Technology [MIT], April 1973).

4 EXAMPLE MODEL OUTPUT

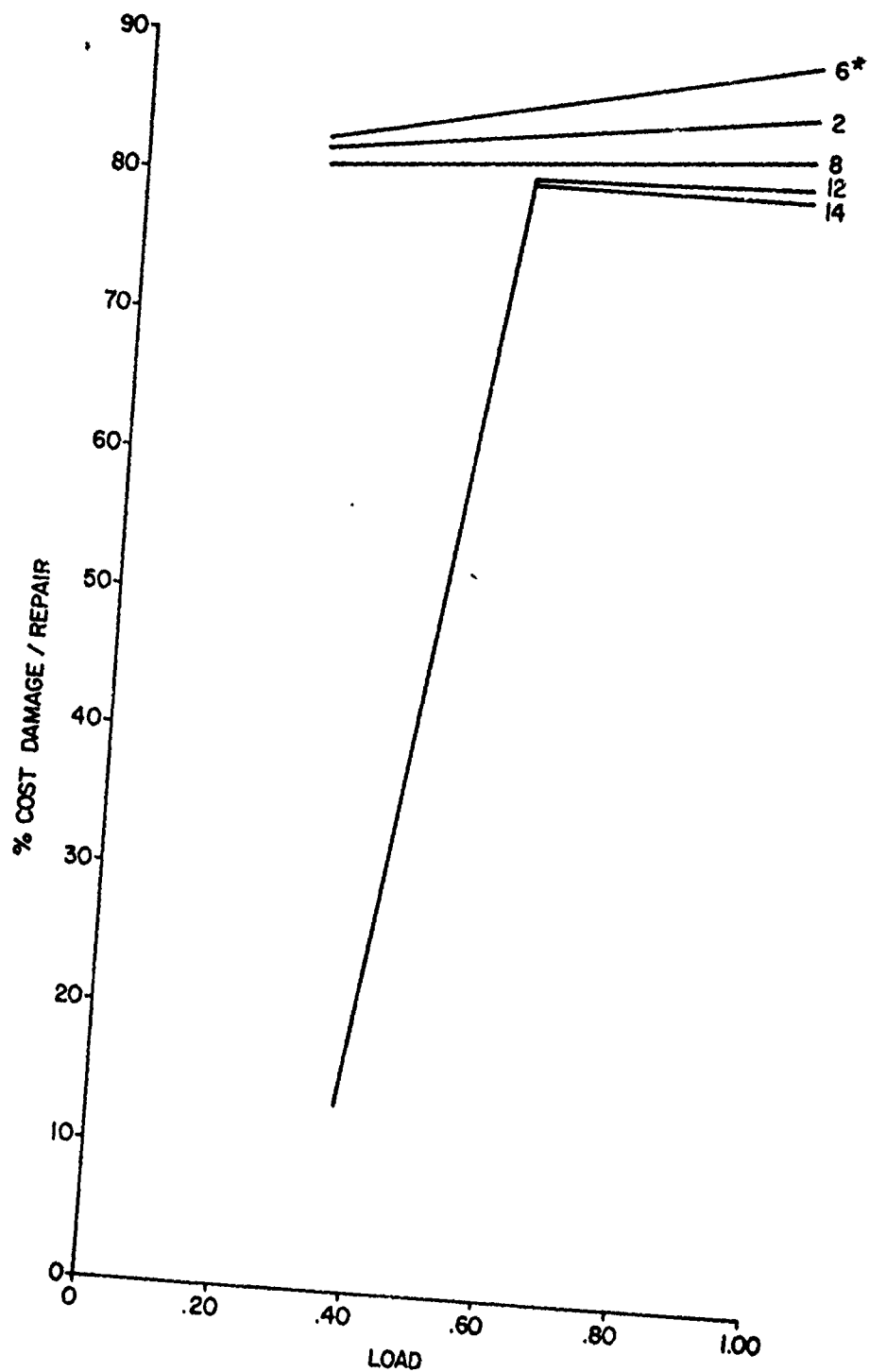
This chapter presents an example of the initial information obtained from the model for a variety of structural configurations.

For purposes of illustration, it is assumed that a number of reinforced concrete building configurations (four to ten stories) are to be assessed for seismic hazard and determination of the cost of the hazard mitigation schemes presented in Chapter 1. It is also assumed that the facilities satisfy 1968 SEAOC Zone 3 requirements, but now must meet higher requirements (up to and including a 1.0 g response spectrum acceleration).

To obtain cost information on a particular building configuration, the configuration parameters required to create a story stiffness model can be provided and the simulation model run. This is, however, time consuming, particularly if several hundred buildings are under consideration. To obtain information on a particular building configuration easily without rerunning the model for each particular configuration, representative building configurations are selected, the cost information determined, and interpolations of the cost curves according to configurations employed. Thus one set of cost curves determined for a set of representative building configurations provides information on many particular building configurations.

Of the 42 configurations (four, seven, and ten stories) in Table 1, 21 were selected for determination of cost information. The dimensional ratio was assumed to be 1.6, and the ratio of shear wall to frame stiffness assumed to be 0, 100, 200, 500, 1000, 2000, or 5000 for four-, seven-, and ten-story configurations. This set of 21 structural configurations provides the building configuration data space. Costs to repair, strengthen, and replace were derived for these configurations for seismic loads ranging from about 0.30 g to 1.0 g response spectra. Limited extrapolation will allow for somewhat higher and lower levels.

Figures 1 through 9 present the configuration-load-cost data for 15 of the 21 configurations with damage/repair, strengthening, and



*Numbers on this and following figures refer to configurations defined in Table 1.

Figure 1. Four-story damage/repair cost percentage.

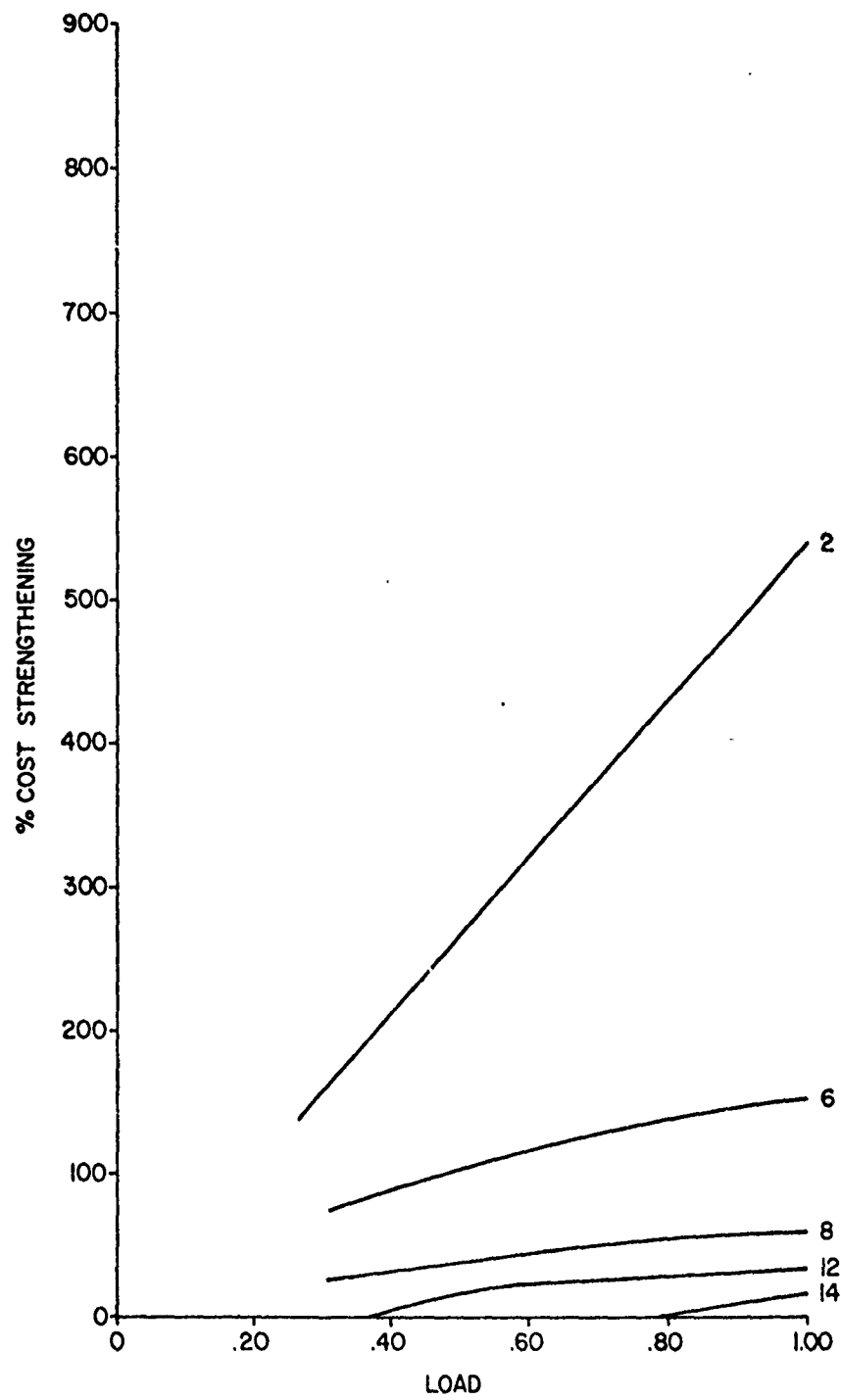


Figure 2. Four-story strengthening cost percentage.

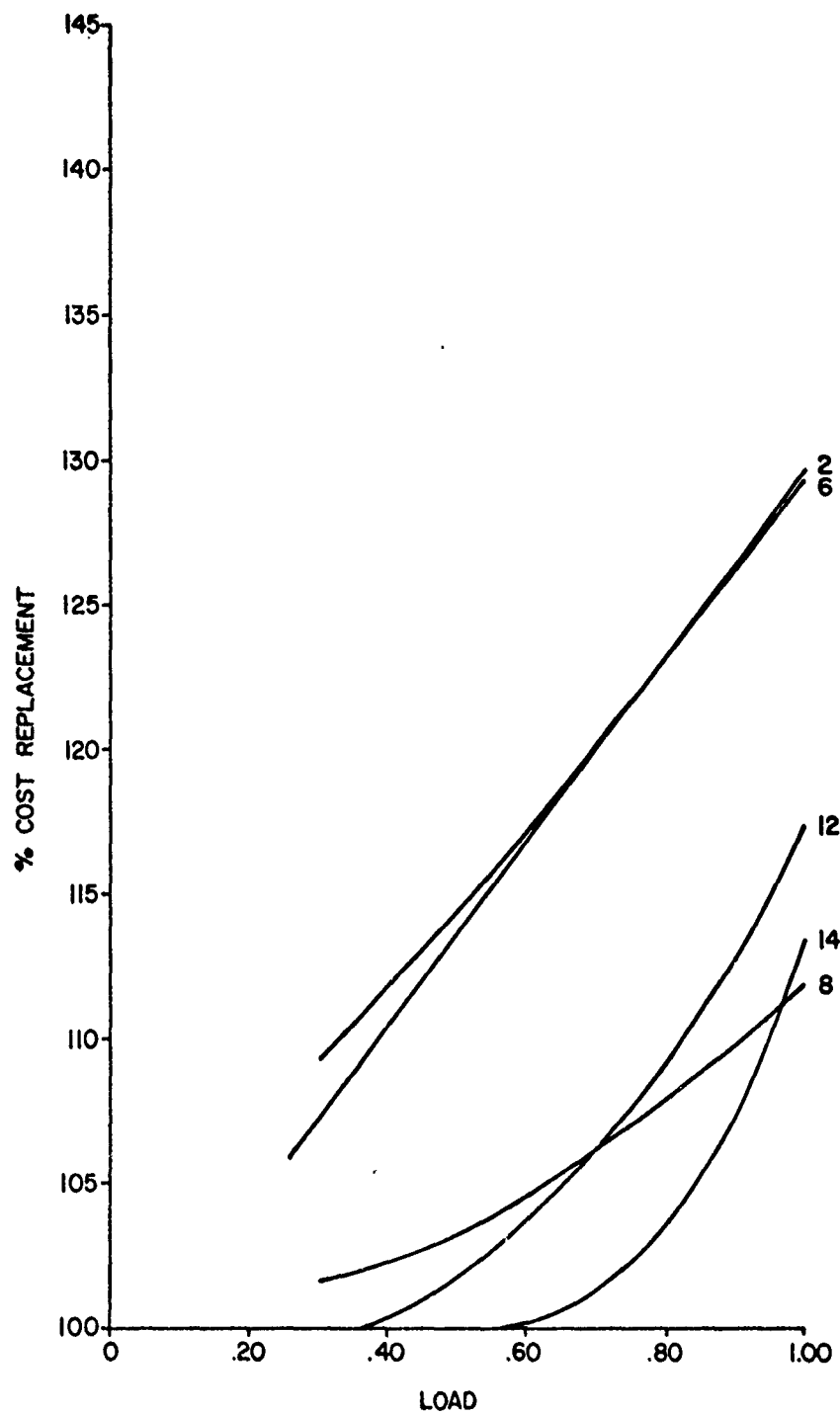


Figure 3. Four-story replacement cost percentage.

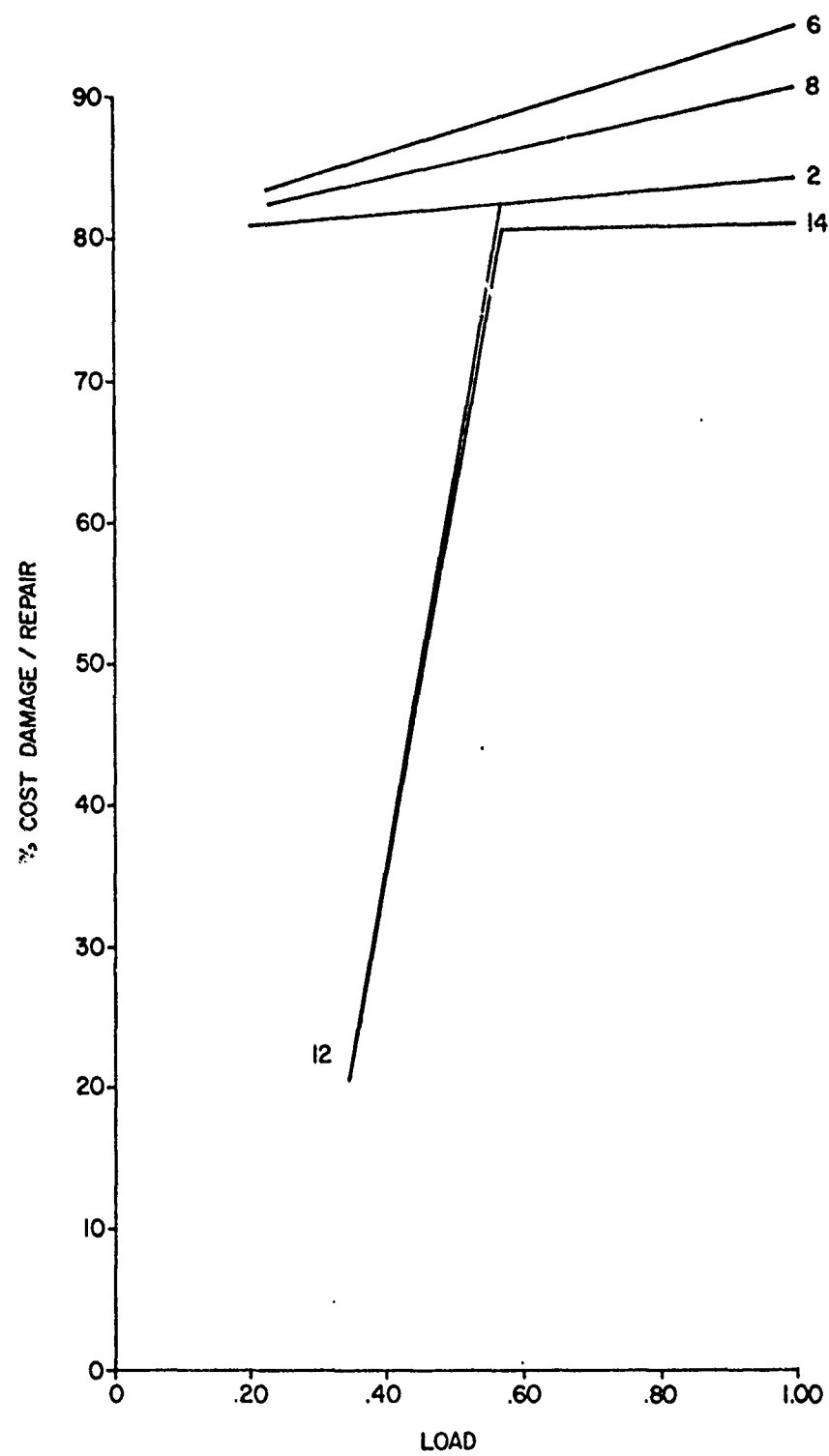


Figure 4. Seven-story damage/repair cost percentage.

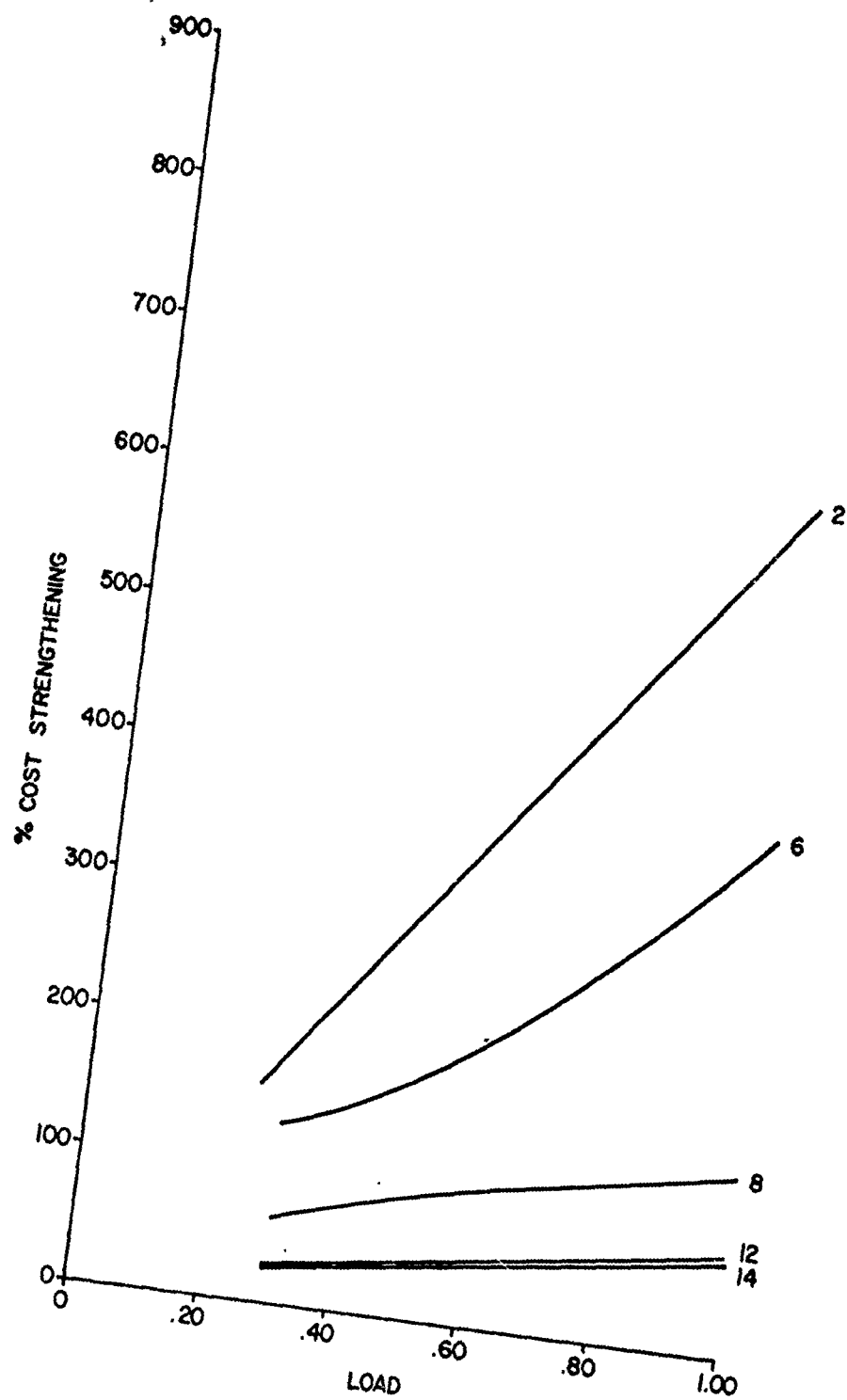


Figure 5. Seven-story strengthening cost percentage.

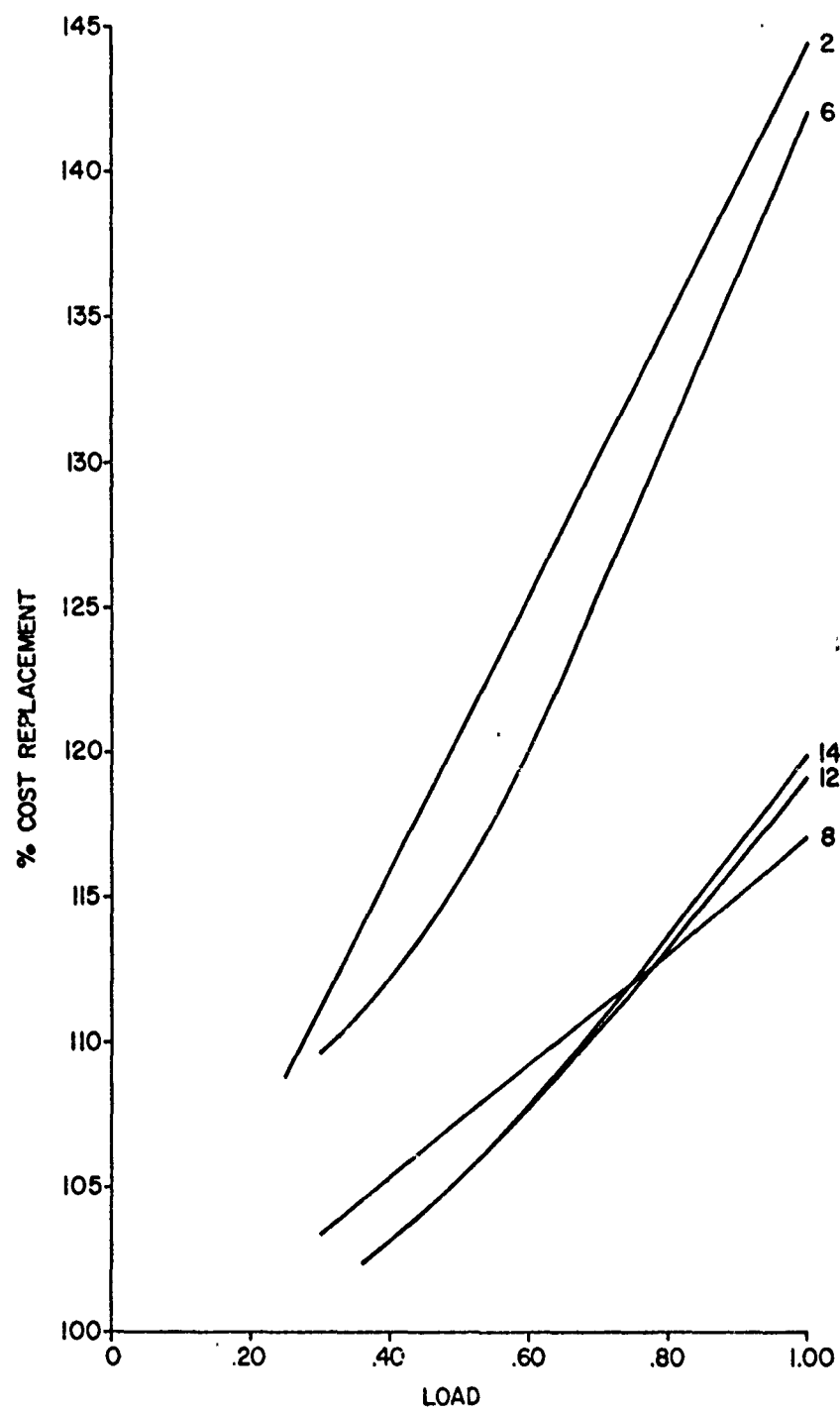


Figure 6. Seven-story replacement cost percentage.

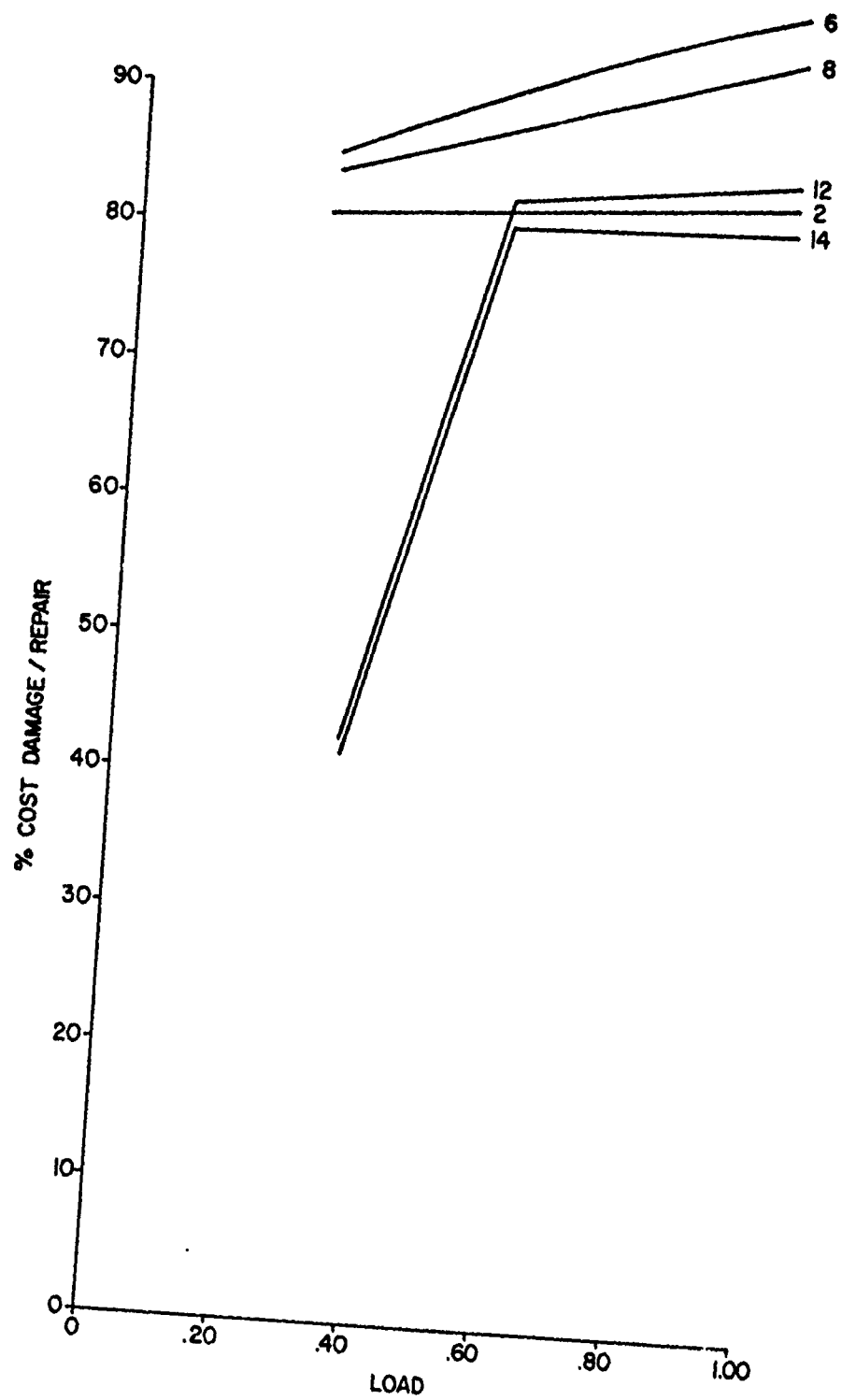


Figure 7. Ten-story damage/repair cost percentage.

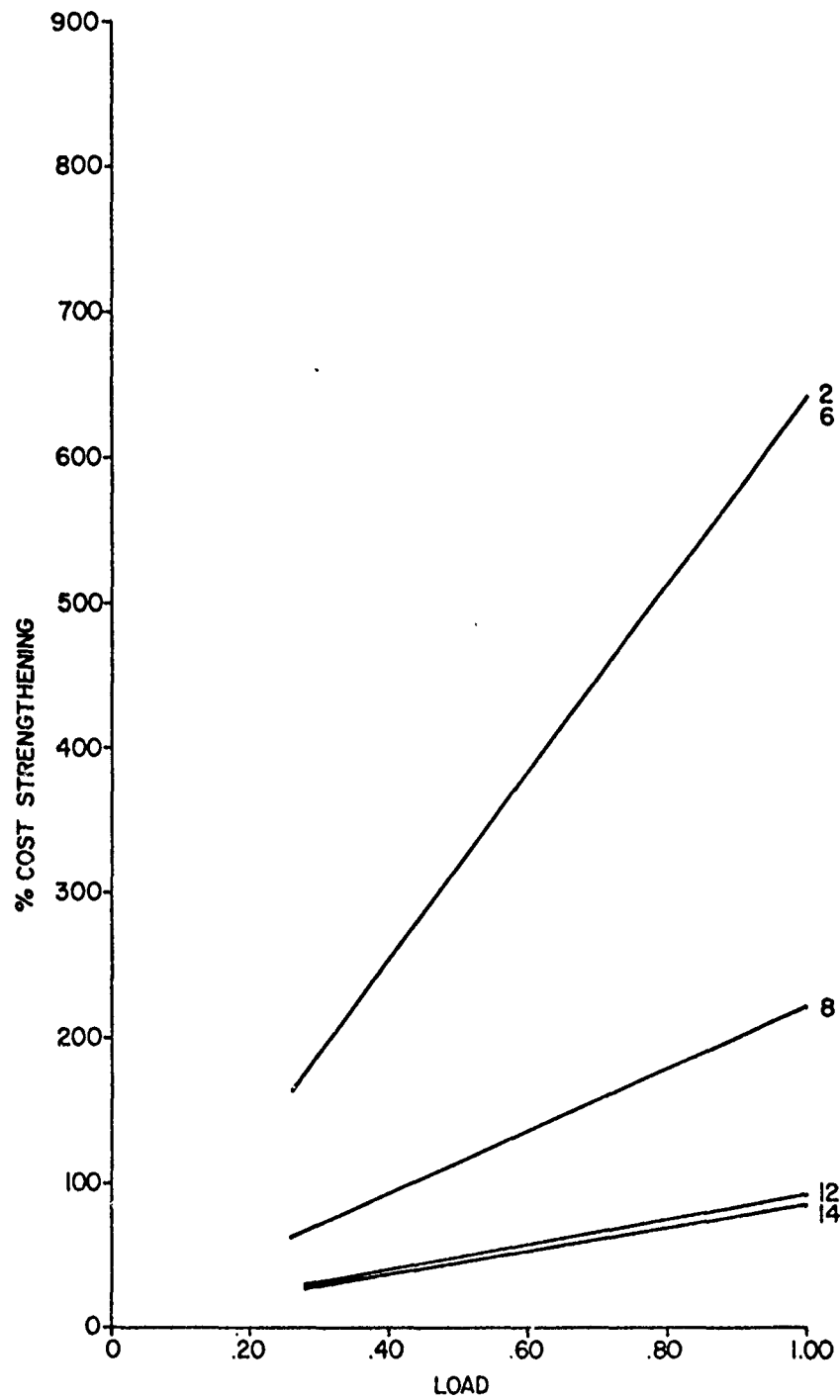


Figure 8. Ten-story strengthening cost percentage.

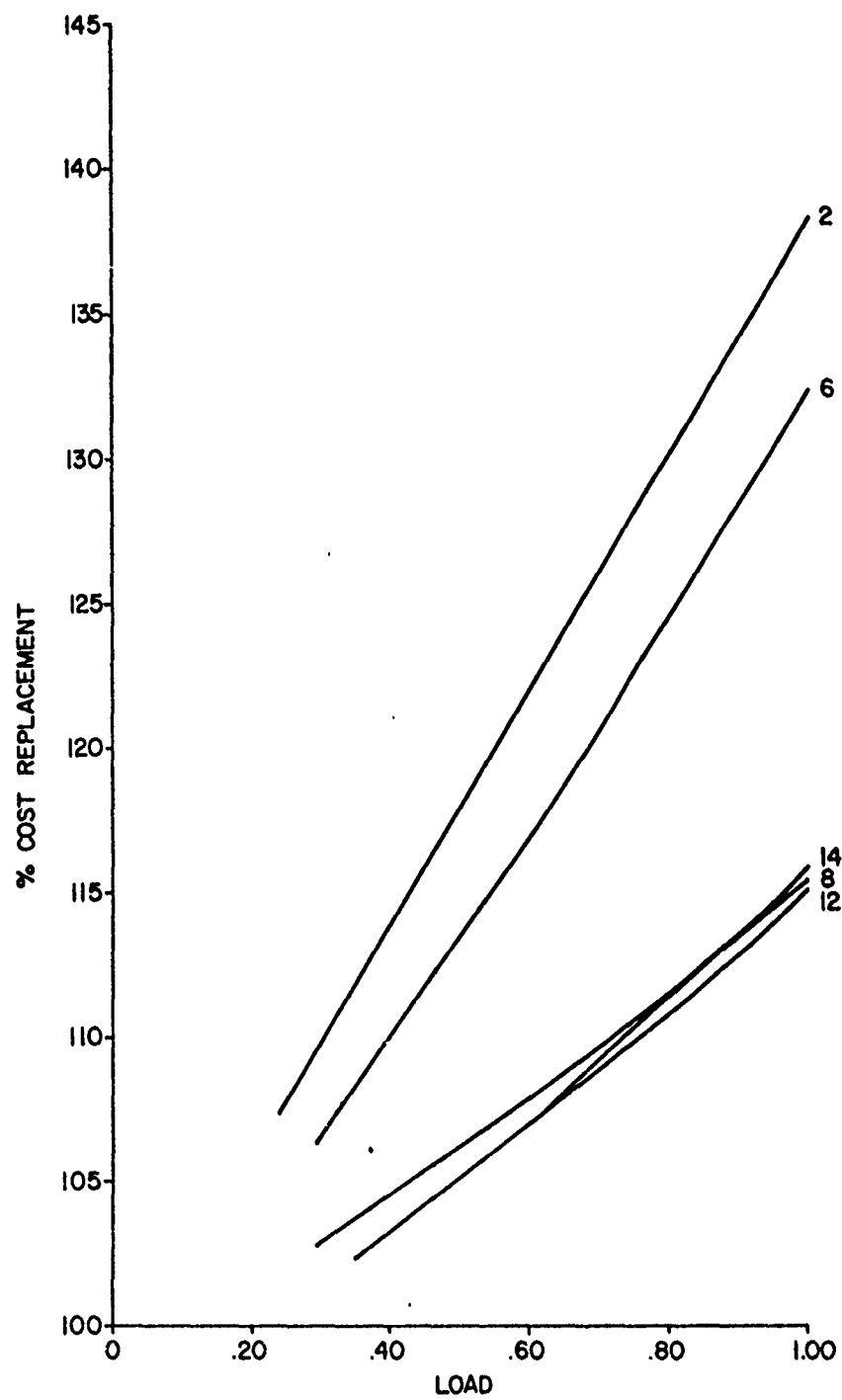


Figure 9. Ten-story replacement cost percentage.

replacement percentage costs of the original facility tabulated separately. Figures 10 through 15 present the percentage costs of the six remaining facilities with damage/repair (D/R), strengthening (S), and replacement (R) costs on one graph. The relative magnitudes of costs of the decision alternatives can be easily ascertained for each facility. The facility numbers used in the figures refer to the configurations listed in Table 1.

The model cost information was analyzed to facilitate understanding of the cost estimates derived from the model and of the costing algorithms detailed in Appendix A, as well as to provide information for improving the cost algorithms in subsequent refinements of the simulation model. This analysis led to the following observations on the damage/repair, strengthening, and replacement costs for the four-, seven-, and ten-story structures:

1. Damage/repair costs

- a. Damage/repair costs increase with demands on the structure for all structure configurations.

- b. The model is not as sensitive to damage/repair as would be desirable; damage/repair costs are near 80 percent for all structure configurations. The reason for this is that the damage factor for structural damage (DF_S) is 1.0 at comparatively low levels of demand because of the moderately low value of ultimate ductility (μ_u equals 5.0). For a 1.0 g response spectrum demand for a facility designed for 1968 SEAOC Zone 3 requirements, the damage should probably be 100 percent, whereas for a 0.25 g response spectrum demand on the same facility, the damage could be less than one-half of this value.

- c. The story displacement damage factor (DF_{NS}) helps increase the sensitivity of the model with respect to damage/repair costs. In refinements of this model, damage may be defined only in terms of relative story displacement.

- d. Damage/repair costs generally decrease with increasing ratios of wall stiffness to frame stiffness. This seems reasonable because of the corresponding decrease in relative story displacement.

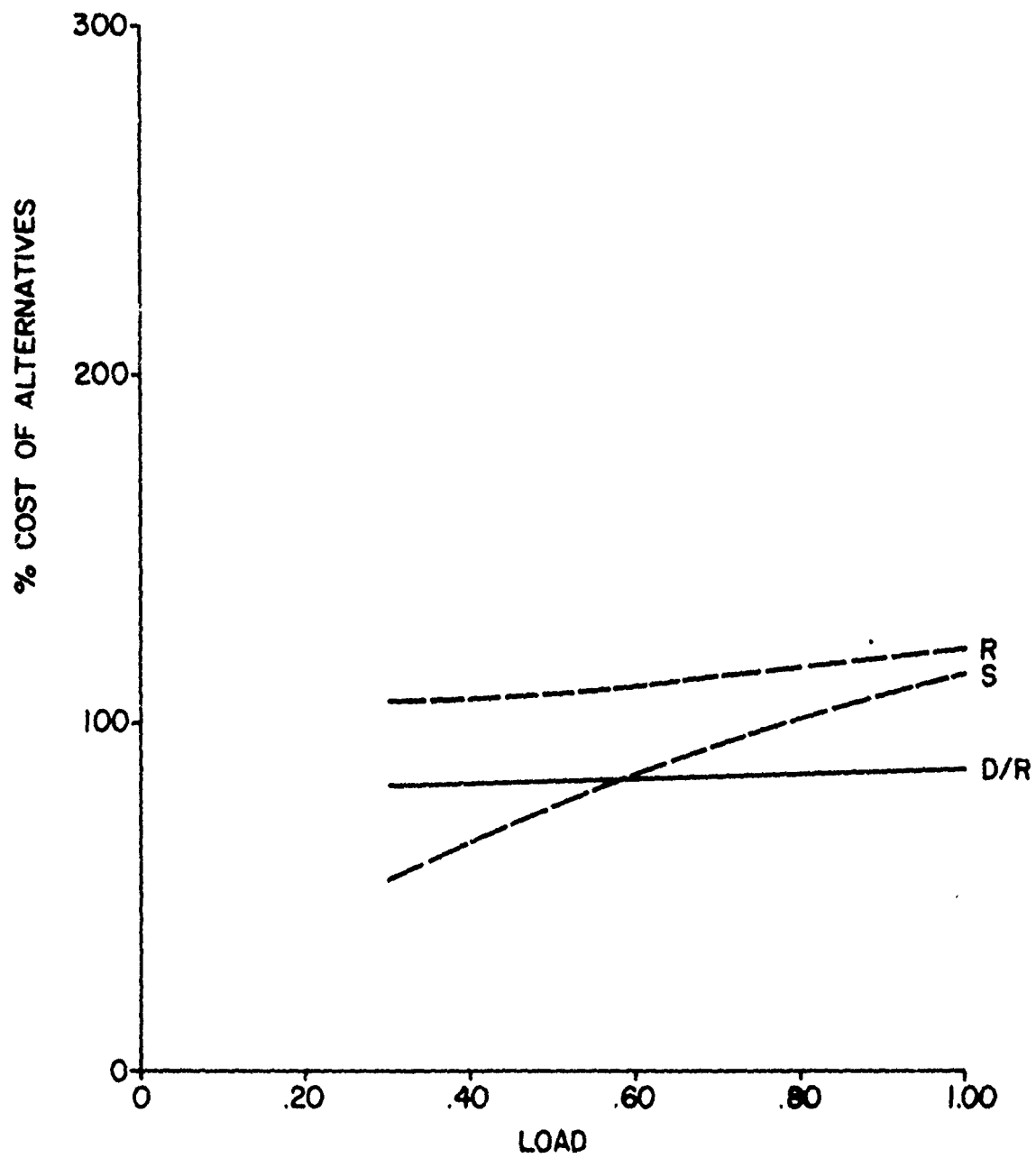


Figure 10. Percentage cost of alternatives for four-story configuration 7.

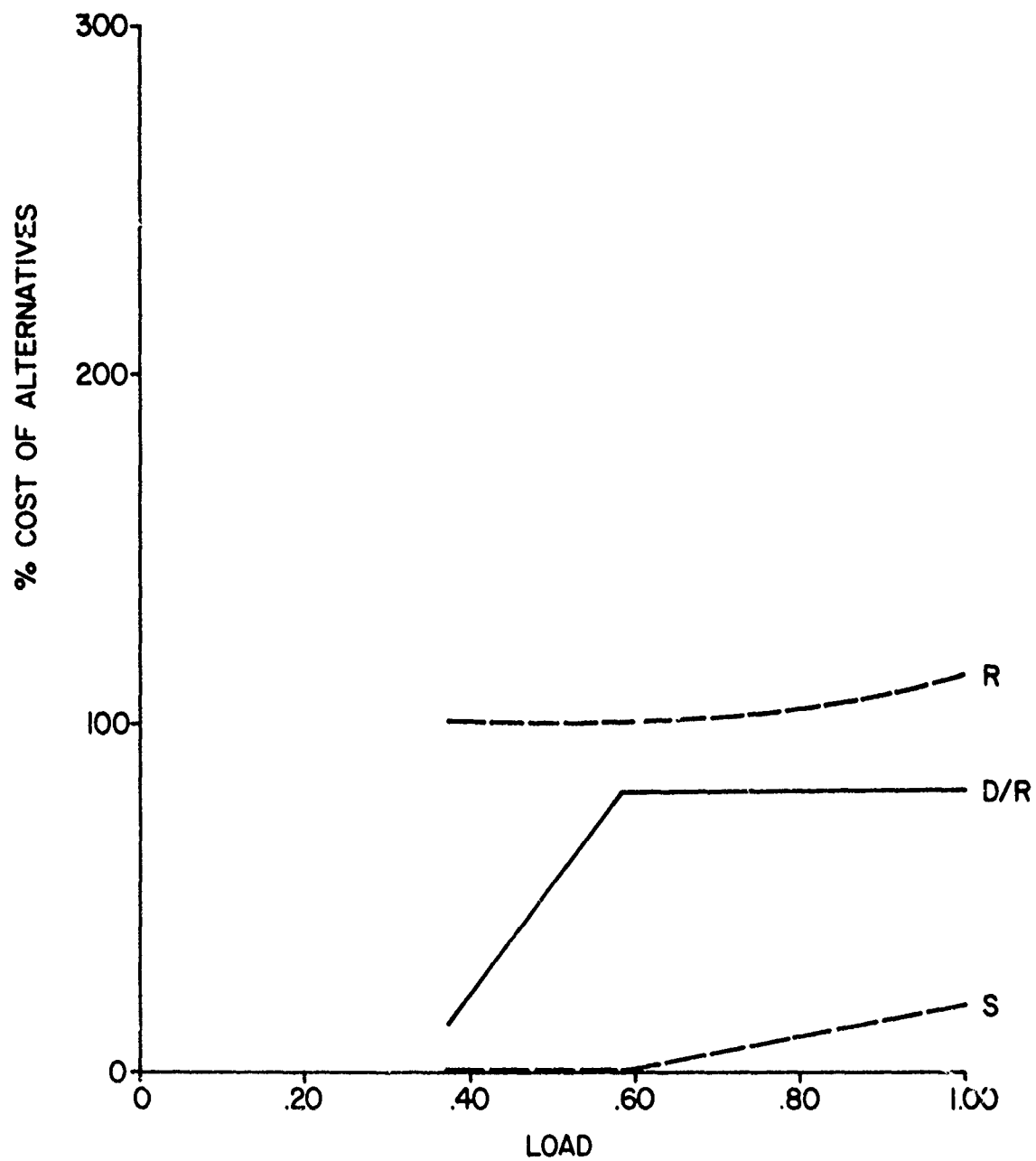


Figure 11. Percentage cost of alternatives for four-story configuration 13.

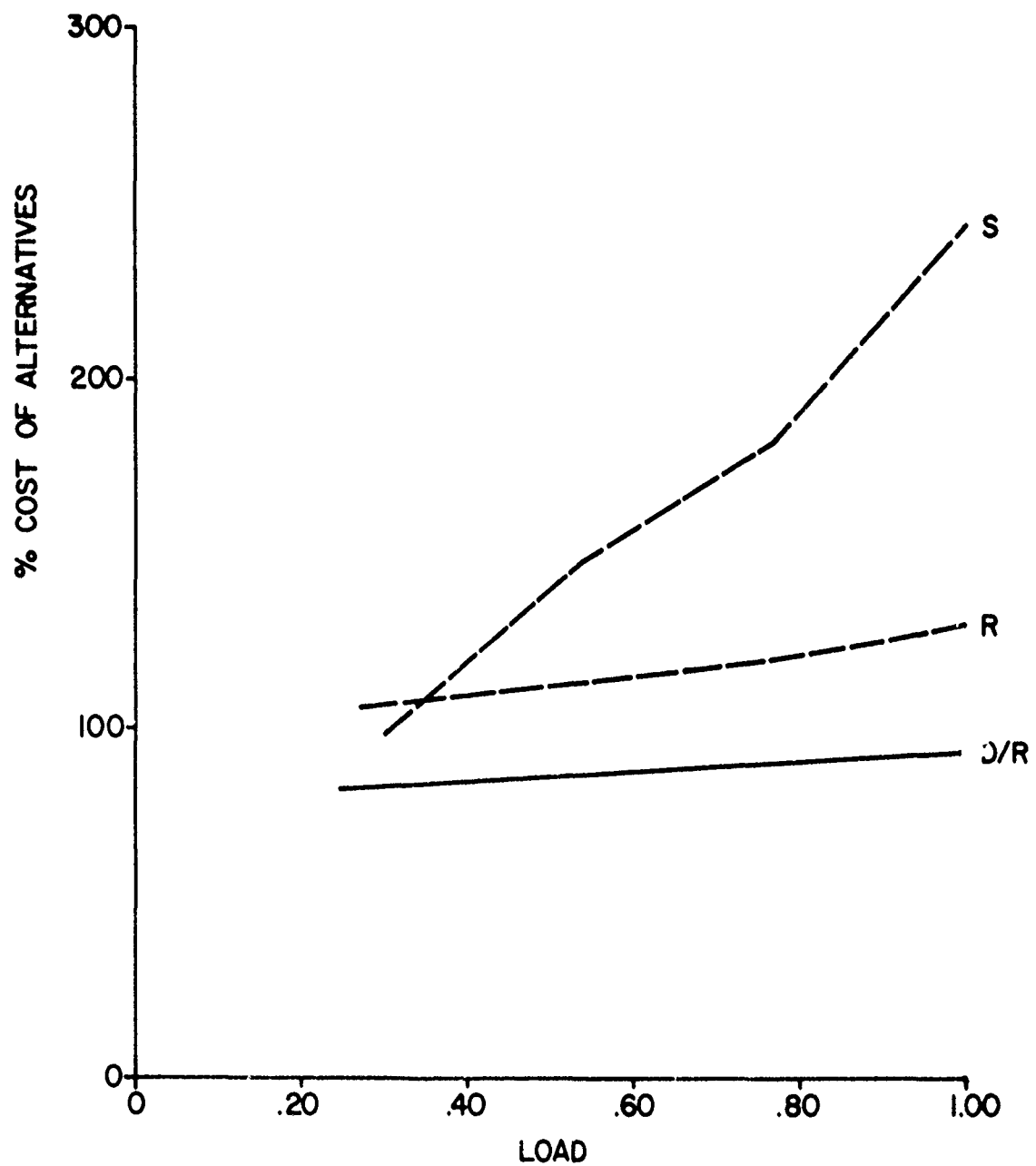


Figure 12. Percentage cost of alternatives for seven-story configuration 7.

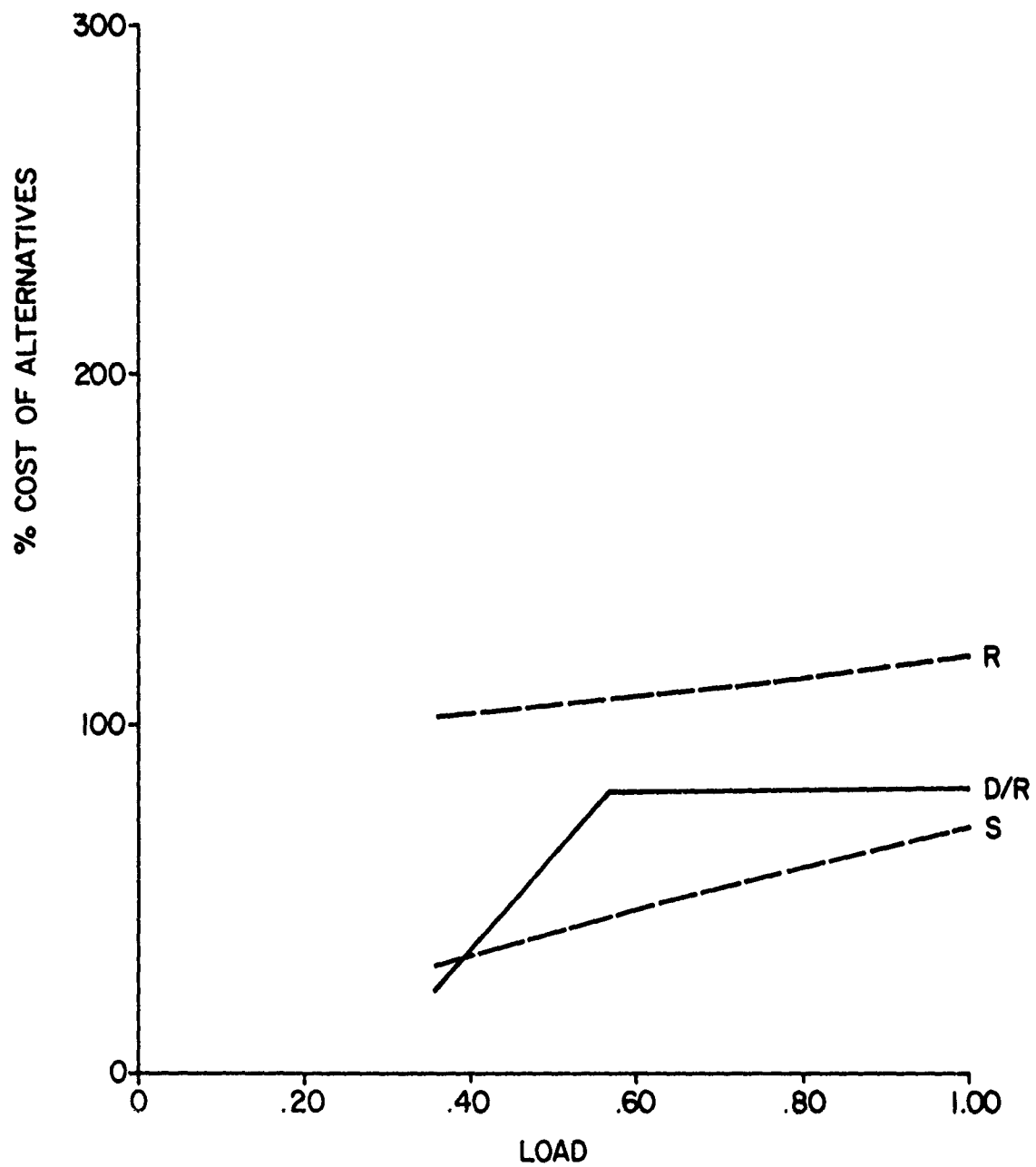


Figure 13. Percentage cost of alternatives for seven-story configuration 13.

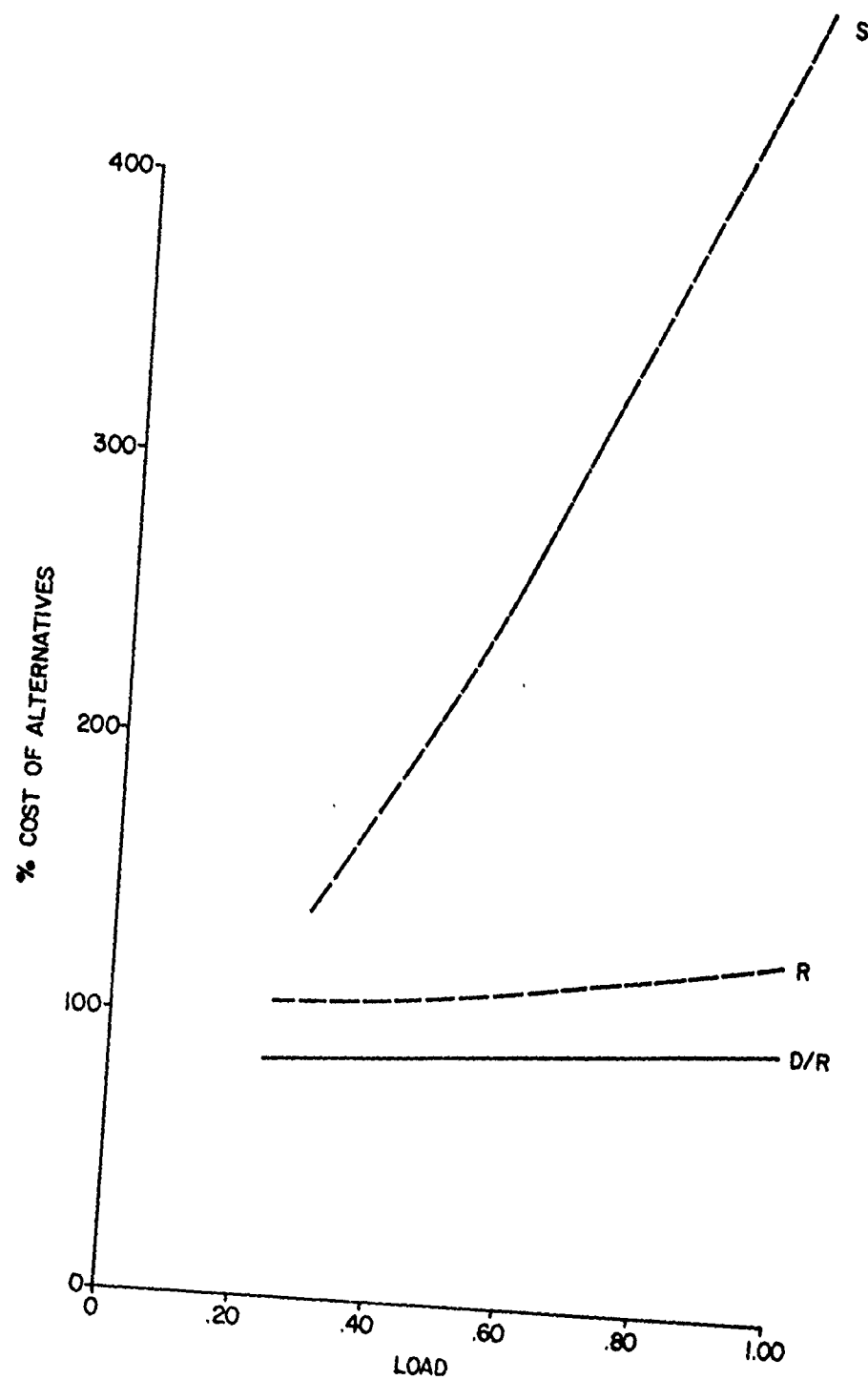


Figure 14. Percentage cost of alternatives for ten-story configuration 7.

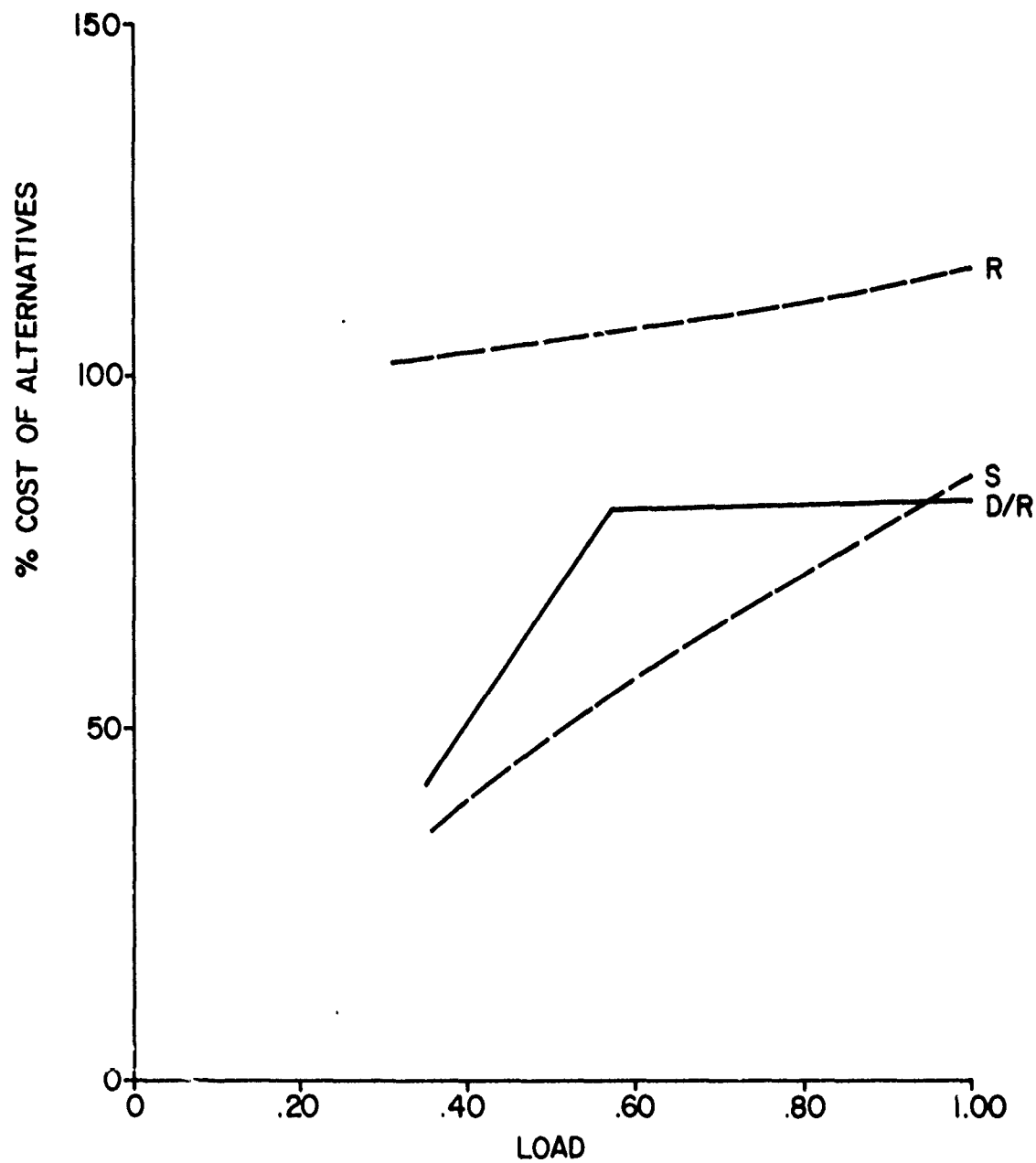


Figure 15. Percentage cost of alternatives for ten-story configuration 13.

e. Two important variables aid in understanding the cost information--mean cost and standard deviation of cost. The first variable provides a measure of the relative magnitude of the cost data, whereas the second variable determines the dispersion of the cost data. Table 2 presents this information for all 21 configurations. In general, for damage/repair costs, the standard deviation (the dispersion) is least for massive shear wall configurations (K equals 1.33), somewhat greater for frame configurations (K equals 0.67), and largest for light shear wall configurations (K equals 0.80). In engineering terms, this indicates that damage/repair cost predictions based on this model are least uncertain for massive shear wall configurations, more uncertain for frame configurations, and quite uncertain for light shear wall configurations. Here again, the relative story displacement damage factor DF_{NS} is the principal quantity defining the dispersion or uncertainty. The mean of the cost information with respect to two configuration sizes--large and small--for all story cases for a structural system with K equal to 0.67 indicates that a small configuration will provide higher damage/repair costs. For the four-story configurations where K equals 0.80 or 1.33, the smaller configuration in general has a lower damage/repair cost. For the ten-story configuration where K equals 0.80 or 1.33, the smaller configuration has the higher damage/repair costs. These results are again attributable to the relative story displacement damage factor.

f. Based on this model, massive shear wall structures have the lowest damage/repair costs while light shear wall structures have the highest damage/repair costs.

2. Strengthening costs

a. Strengthening costs increase almost linearly with the demands on the structure for all structural configurations.

b. Strengthening costs are widely dispersed.

c. The strengthening algorithm based on relative stiffness is very sensitive to structural system type and to load magnitude. For the small amount of strengthening data available, the model is

Table 2
Mean and Standard Deviation of Costs for Configurations
Over Decision Alternatives

Configuration Number	Damage/Repair		Strengthening		Replacements	
	<u>m</u>	<u>s</u>	<u>m</u>	<u>s</u>	<u>m</u>	<u>s</u>
Four Story						
2	84.0	1.9	348.4	163.4	118.5	9.6
6	86.6	3.0	118.6	34.8	119.0	8.7
7	84.9	2.2	87.4	25.2	113.0	6.8
8	82.1	1.0	43.6	14.3	106.1	4.5
12	81.1	.3	19.9	14.2	107.4	7.6
13	80.6	.1	7.1	8.8	104.2	6.1
14	80.2	.1	3.9	7.9	104.1	6.4
Seven Story						
2	82.7	1.3	394.7	187.2	127.8	14.4
6	89.7	4.5	240.8	107.7	124.4	14.2
7	89.0	4.2	167.7	60.9	117.3	9.3
8	86.9	3.2	99.2	29.3	110.2	5.9
12	83.2	.9	52.2	17.1	110.4	7.2
13	81.7	.5	50.8	17.0	110.5	7.6
14	80.7	.2	50.1	16.8	110.6	7.6
Ten Story						
2	82.0	1.0	412.5	197.5	123.9	12.4
6	91.4	5.0	409.2	199.5	119.0	11.2
7	91.0	4.9	300.6	144.7	115.8	9.5
8	88.6	4.1	147.7	61.5	108.9	5.5
12	83.6	1.0	65.0	22.7	108.6	5.5
13	82.0	.6	62.0	21.7	108.8	5.6
14	80.8	.2	60.0	21.0	108.9	5.8

adequate. The model is calibrated on the basis of a massive shear wall structure of eight, ten, and 11 stories; strengthening costs increase substantially for both the light shear wall structure (K equals 0.80) and the frame structure (K equals 0.67). Strengthening costs also increase fairly substantially as the number of stories of the building increases. Both of these trends in strengthening cost data are reasonable. Since strengthening cost data are widely dispersed, standard deviations are often over one-third of the mean value and may approach one-half of the value. This is not the case for either damage/repair costs or replacement costs. Greater variability is demonstrated in frame and light shear wall structures than in massive shear wall structures in general. The analysis of the strengthening algorithm is complicated by the simulated design procedure. At low levels, moment appears to control the column design whereas at high levels, shear appears to control the design.

d. Based on this analysis, massive shear wall structures require the least strengthening while frame structures require substantially more, with light shear wall structures between the two.

3. Replacement costs

a. Replacement costs increase nearly linearly with the level of demand on the structure for all structural configurations.

b. Replacement costs are less dispersed than strengthening costs, but more dispersed than damage/repair costs. In general, replacement cost dispersion decreases as the ratio of shear wall to frame stiffness increases.

c. Replacement costs for light shear wall and frame structures increase over those for massive shear wall structures; that is, replacing a frame structure with a frame structure costs substantially more than replacing a massive shear wall structure with a massive shear wall structure. Replacement costs based on increased material usage are accurate, and the simulated design may provide more accurate information here than in the strengthening and damage/repair algorithms. Replacement costs are slightly higher for seven-story facilities than for ten-story facilities (a reflection on the available calibration data).

5 DECISION METHODOLOGY

The computer simulation model is a decision tool which uses design simulation to translate a structure's behavior into variables that correlate with costs. Associated with cost information for repair, strengthening, and replacement of a given facility is a rationale for choosing one of the three following decision alternatives:

1. Leave the facility unchanged and anticipate repair of the structure in the case of damage due to seismic activity (inherently one assumes that hazard to life--a variable not quantified in the model--is small and acceptable)
2. Strengthen the facility by addition of lateral-force-resisting elements to present code levels
3. Replace the facility with a facility designed for present code levels.

The rationale for making the decision may be simple (minimum cost) or complex (utility theory). It is important to realize that the decision rationale is not built into the simulation model, but is an adjunct. This chapter describes two possible decision rationales.

The first decision rationale is termed the Current Minimum Cost Rationale (CMCR). This decision rationale states "select the alternative that will result in the minimum cost based upon the cost information from the computer model." This rationale would be the most logical if uncertainty is not considered, i.e., if the problem is deterministic.

The second decision rationale, termed the Expected Monetary Cost Rationale (EMCR), is an extension of the first, in that it introduces a level of uncertainty quantification into the decision procedure. The EMCR states "select the alternative that will result in the minimum expected monetary cost based upon the computer simulation model and the site dependent seismic history." The expected monetary cost for an event E , is determined by multiplying the cost associated with the event (C) times the probability of its occurrence ($P(E)$). The

result is interpreted as the amount of money the event may be expected to cost over a long period of time. If one considers all possible events in a mutually exclusive way (i.e., no two events have anything in common with each other), the chance or probability of an event measures the likelihood of the event relative to a period of time. Thus, this more sophisticated rationale extends the cost estimates into the future based on data from the past. There are a number of ways of using data based on seismic activity in a particular area to construct a "seismic histogram" defining the relative frequencies of the largest expected seismic event in a given number of years. Shah⁷ has developed a computer program which uses earthquake data in terms of coordinates of the epicenter and magnitude in an appropriate attenuation equation to arrive at a seismic histogram for a particular site.

The model is easily used in conjunction with this second decision rationale. Once the seismic histogram is obtained, the probability of a certain Modified Mercalli Intensity (MMI) level earthquake is determined. The MMI can then be correlated with peak ground acceleration and, based on this expected peak ground acceleration, the response spectrum approach can be used to determine code input requirements to the simulation model. For a particular structural configuration, the simulation model with the given series of design input requirements will then provide a series of costs to repair, strengthen, or replace.

There are a host of other decision rationales. However, these two are perhaps the simplest and provide a starting place. Application of both these rationales for the cost data provided in Chapter 4 is straightforward once the site risk has been determined. The completed simulation model will provide the necessary input for both these rationales in addition to considering other decision rationales.

⁷ H. C. Shah, "Seismic Risk Analysis," Notes for course given to Corps of Engineers, August 11-15, 1975, San Francisco, CA.

6 CONCLUSIONS AND RECOMMENDATIONS

Conclusions

This report has described a computer simulation model developed in the first phase of a study designed to provide a decision tool for assessing the seismic hazard to Army facilities and the cost of mitigation schemes for reducing the hazard. Observation of the simulation model's behavior and the supporting data has led to the following conclusions:

1. Cost information can be correlated with structural behavior by selecting suitable variables related to the structural behavior. In the model design, base shear, relative story displacement, building stiffness, and the dimensions of building elements are the variables considered. A calibration procedure correlates cost information with these variables.

2. The simulated design concept appears to provide the best means of comparing the behavior of structures at different load requirements on the same basis. This approach was necessary for estimating strengthening and replacement costs.

3. For the degree of detail incorporated into the model, the data available for calibration are inadequate. The MIT damage study on the San Fernando earthquake does not consider the structural system type. The strengthening data are sparse and keyed to critical facilities. The replacement data are for moderately low seismic levels and extrapolation to the levels considered in this report may be misleading.

4. The damage/repair algorithm selected is insensitive at high levels of seismic force because of the comparatively low value of ultimate structural ductility. Damage/repair costs increase with level of seismic activity for a given building configuration. Damage/repair costs are least for massive shear wall configurations and highest for light shear wall configurations.

5. Strengthening costs increase almost linearly with seismic level. Strengthening costs decrease with an increase in ratio of shear wall to frame stiffness. Strengthening costs also increase with the building story configuration.

6. Replacement costs increase nearly linearly with seismic level. Replacement costs in general are highest for frame building systems and lowest for massive shear wall building systems. Replacement costs decrease with increasing ratio of shear wall to frame stiffness.

Recommendations

The following recommendations are made based on the results of the first phase of this study:

1. The simulated design procedure in the model should be augmented with a more flexible interactive design procedure in the case of strengthening and replacement algorithms, if analytical prediction of strengthening and replacement is to be a viable approach.
2. A site risk model should be incorporated into the overall model.
3. The decision methodology should be examined and implemented.
4. Additional data for calibration of the model should be obtained either through disaster studies or statistical simulation.
5. The simulation model should be used as a true simulator for developing repair/damage, strengthening, and replacement cost data for many structural configurations. This is a realistic approach dictated by the lack of data and the resources involved in model calibration for many particular cases.

APPENDIX A:

COMPUTER MODEL

This appendix describes the computer model in detail, including the structural design/analysis algorithms and the cost computation models. The first portion of the appendix presents the derivation of the equations for the elastic structural model. The second portion describes the design algorithm, and the third part contains a detailed description of the cost computation models for facility repair, strengthening, and replacement.

Derivation of the Structural Model Equations Governing Model Behavior⁸

The following assumptions were made in deriving the structural model:

1. All lateral force is resisted by a frame system of a combination frame-shear wall system.
2. The model foundation is rigid.
3. The floor diaphragms are rigid with girders of infinite stiffness.
4. Lateral forces are distributed according to the SEAOC provisions.
5. Shear walls act as short deep beams with stiffness components in bending only.
6. Structure overturning moment and dead/live load are accounted for in the analysis and assumed to increase the moments on frame and shear wall elements.
7. Sizing of elements is based on column effective depths and the percentage of steel contained in the shear wall.

⁸ J. C. Shang, *Design of Combined Frames and Shear Walls*, Advanced Engineering Bulletin 14 (Portland Cement Association, 1973).

8. Elongation and contraction of the shear wall and column elements is neglected.

The derivation is based on a finite difference model of a structure of N stories (with N being greater than or equal to 4) and three bays in the direction of the applied loads. This is a standard model for finite difference analysis of building structural models.

Figure A1 shows the frame system configuration used in determining the lateral forces distribution. Equilibrium of joint i requires that the external forces (F_i^f) be balanced by the internal shears (V_i^f and V_{i+1}^f):

$$F_i^f = V_i^f - V_{i+1}^f \quad i = 2, 3, \dots, N-1 \quad [\text{Eq A1}]$$

However, internal shears can be expressed as follows in terms of story displacements:

$$V_i^f = \sum \frac{12EI_i}{6h_i^3} (w_i - w_{i-1}) \quad i = 2, 3, \dots, N-1$$

[Eq A2]

$$V_{i+1}^f = \sum \frac{12EI_{i+1}}{6h_{i+1}^3} (w_{i+1} - w_i) \quad i = 2, 3, \dots, N-1$$

- where E = modulus of elasticity
 I_i = moment of inertia of column at level i
 h_i = story height
 w_i, w_{i+1} = story displacement at i^{th} and $i^{\text{th}} + 1$ level
12 = stiffness parameter for the fixed-end columns
6 = number of equivalent columns (i.e., there are three bays with interior column stiffness twice that of the exterior columns).

To simplify the expressions and the subsequent derivation, it is assumed that h_i is equal to the story height (h). Thus,

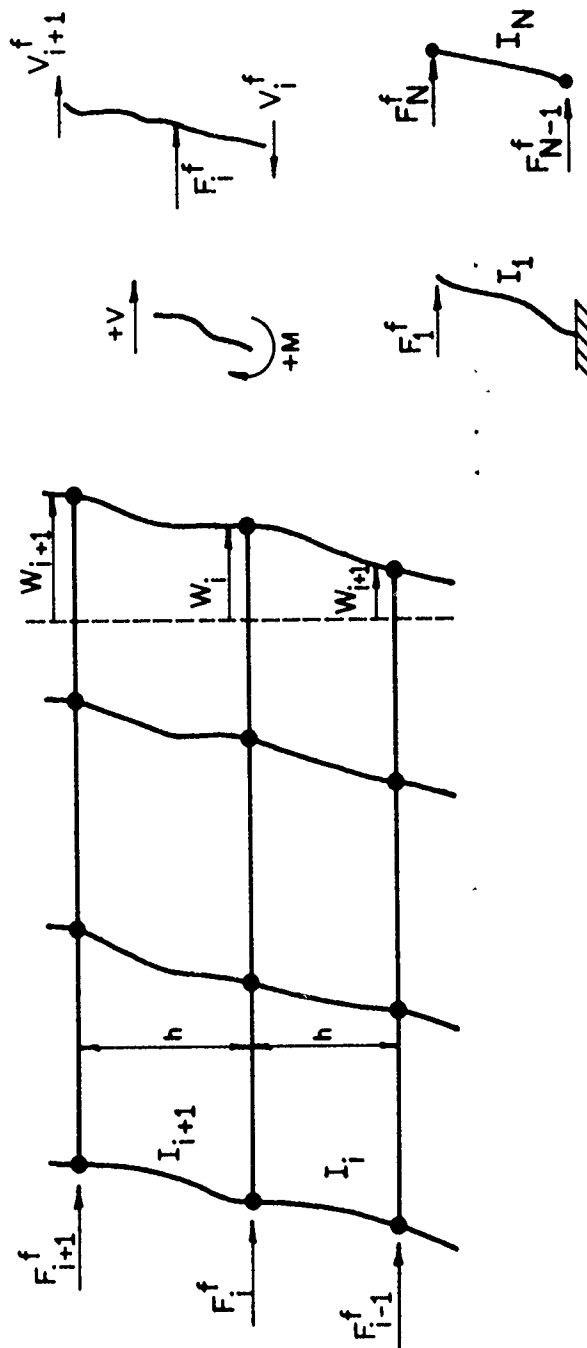


Figure A1. Frame forced-displacement configuration.

$$V_i^f = 6\left(\frac{12EI_i}{h^3}\right)(w_i - w_{i-1}) \quad i = 2, 3, \dots, N-1$$

[Eq A3]

$$V_{i+1}^f = 6\left(\frac{12EI_{i+1}}{h^3}\right)(w_{i+1} - w_i) \quad i = 2, 3, \dots, N-1$$

The equilibrium expressions for the frame then become:

$$F_i^f = 6\left(\frac{12E}{h^3}\right)\{-I_i w_{i-1} + (I_i + I_{i+1})w_i - I_{i+1}w_{i+1}\} \quad i = 2, 3, \dots, N-1 \quad [\text{Eq A4}]$$

$$F_1^f = V_1^f - V_2^f = 6\left(\frac{12E}{h^3}\right)\{(I_1 + I_2)w_1 - I_2 w_2\} \quad [\text{Eq A5}]$$

$$F_N^f = V_N^f = 6\left(\frac{12E}{h^3}\right)I_N(w_N - w_{N-1}) \quad [\text{Eq A6}]$$

For the shear wall system (Figure A2), the equations of equilibrium for the lateral force distribution can be written as follows:

$$F_i^S = V_i^S - V_{i+1}^S \quad i = 3, 4, 5, \dots, N-2 \quad [\text{Eq A7}]$$

where F_i^S = portion of design lateral force acting on the shear wall at floor i

V_i^S, V_{i+1}^S = portion of shear in shear wall at floors i and $i+1$, respectively.

However, the internal shear on the wall can be expressed in terms of the internal bending moment (M^S) of the wall as follows:

$$M_{i-1}^S + hV_i^S - M_i^S = 0$$

[Eq A8]

$$M_i^S + hV_{i+1}^S - M_{i+1}^S = 0$$

or

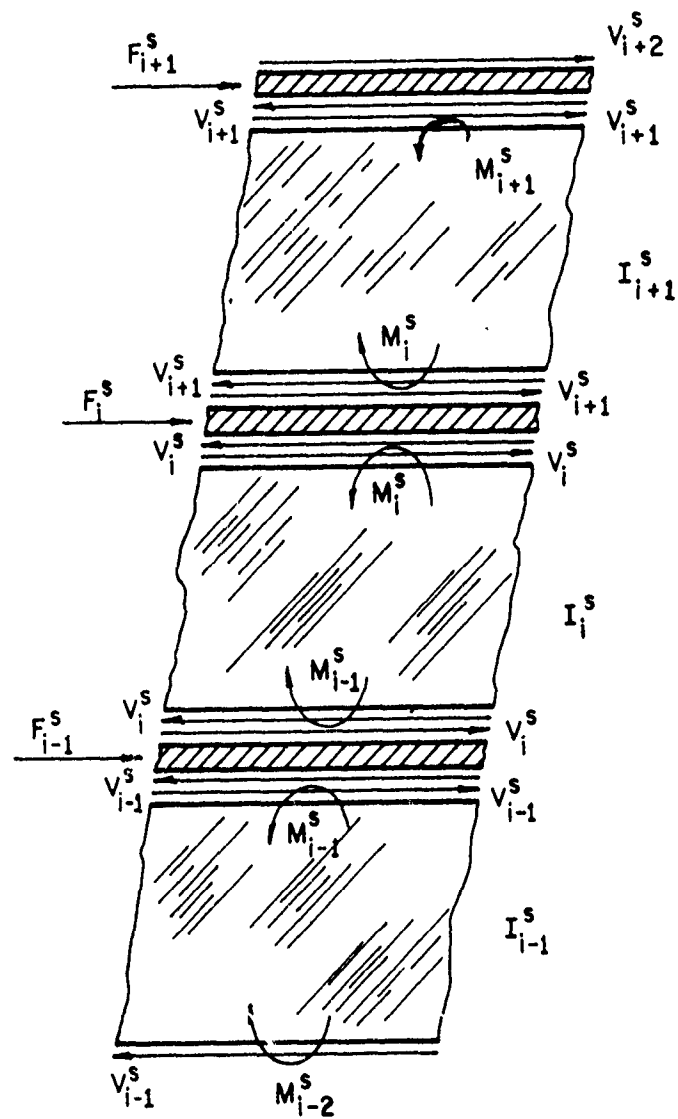


Figure A2. Shear wall force configuration.

$$V_i^S = (M_i^S - M_{i-1}^S)/h \quad [\text{Eq A9}]$$

$$V_{i+1}^S = (M_{i+1}^S - M_i^S)/h$$

and

$$F_i^S = (-M_{i-1}^S + 2M_i^S - M_{i+1}^S)/h \quad [\text{Eq A10}]$$

From the beam-bending formula (Eq A11)

$$\frac{d^2 w}{dh^2} = -\frac{M}{EI} \quad [\text{Eq A11}]$$

and central difference considerations, it was determined that the bending moment at floor i (M_i) can be expressed as:

$$M_i = \frac{-EI_i^S}{h^2} (w_{i+1} - 2w_i + w_{i-1}) \quad [\text{Eq A12}]$$

Thus, the lateral force on the structure may be expressed in terms of the structure story displacements as follows:

$$F_1^S = V_1^S - V_2^S = \frac{EI_1^S}{h^3} \left\{ \frac{I_2^S}{I_1^S} w_3 - 2\left(1 + \frac{I_2^S}{I_1^S}\right) w_2 + \left(6 + \frac{I_2^S}{I_1^S}\right) w_1 \right\} \quad [\text{Eq A13}]$$

$$w_1 = -w_{-1}, \quad w_0 = 0 \quad I_0 = I_1$$

$$F_2^S = V_2^S - V_3^S = \frac{EI_2^S}{h^3} \left\{ \frac{I_3^S}{I_2^S} w_4 - 2\left(1 + \frac{I_3^S}{I_2^S}\right) w_3 + \left(4 + \frac{I_1^S + I_3^S}{I_2^S}\right) w_2 - 2\left(1 + \frac{I_1^S}{I_2^S}\right) w_1 \right\} \quad [\text{Eq A14}]$$

$$F_i^S = \frac{EI_i^S}{h^3} \left\{ \frac{I_{i+1}^S}{I_i^S} w_{i+2} - 2\left(1 + \frac{I_{i+1}^S}{I_i^S}\right) w_{i+1} + \left(4 + \frac{I_{i-1}^S + I_{i+1}^S}{I_i^S}\right) w_i - 2\left(1 + \frac{I_{i-1}^S}{I_i^S}\right) w_{i-1} + \frac{I_{i-1}^S}{I_i^S} w_{i-2} \right\} \quad [\text{Eq A15}]$$

$$i = 3, 4, 5, \dots, N-2$$

$$F_{N-1}^S = V_{N-1}^S - V_N^S = \frac{EI_{N-1}^S}{h^3} \left\{ -2w_N + \left(4 + \frac{I_{N-2}^S}{I_{N-1}^S} \right) w_{N-1} - 2 \left(1 + \frac{I_{N-2}^S}{I_{N-1}^S} \right) w_{N-2} + \frac{I_{N-2}^S}{I_{N-1}^S} w_{N-3} \right\} \quad [\text{Eq A16}]$$

$$F_N^S = V_N^S = \frac{EI_{N-1}^S}{h^3} (w_N - 2w_{N-1} + w_{N-2}) \quad [\text{Eq A17}]$$

The above set of equations for F_i^f and F_i^S ($i = 1, 2, \dots, N$) are the equations that govern the behavior of the structural model due to a lateral distribution of forces over the model. The final equilibrium expressions for the structure are obtained by adding F_i^f and F_i^S for $i = 1, 2, 3, \dots, N$.

Thus

$$F_i = F_i^f + F_i^S = H_i(w) \quad i = 1, 2, 3, \dots, N \quad [\text{Eq A18}]$$

where F_i = total external lateral load

F_i^f, F_i^S = as defined above

$H(w)$ = a function of the stiffness of the elements of the structure and the absolute lateral displacements.

Dead and live load, which are also assumed in the model, are considered further in the Design Considerations section. A series of runs for typical building configurations was made to examine the response and the moments/forces in frame-shear wall elements. The results obtained compared favorably with those of Khan and Sbarounis,⁹ whose work provided the basis for this check on the structural model.

Shear expressions for the shear wall were included in initial consideration of the model. Although this unnecessarily complicated

⁹ F. R. Khan and J. A. Sbarounis, "Interaction of Shear Walls and Frames," *Proceedings ASCE Structural Division*, ST3 (June 1964).

the model, it did provide some insight into the relative magnitude of shear wall bending story displacement and shear wall shear story displacement. For massive shear walls in low buildings, shear wall shear story displacement dominates shear wall bending story displacement. However, for buildings of four or more stories and even moderately heavy shear walls, shear wall bending becomes the dominant contributor to story displacement.

Design Considerations

The computer program simulates a structure's design for a given load requirement to provide consistent information for different sets of load requirements. The simulated design is generated to meet a set of criteria. These criteria, which require that the column and shear wall resistances equal or exceed the force demands, are as follows:

1. Columns shall be sized for ultimate strength considerations for axial and bending moment.
2. Columns shall be checked for tension from overturning moment.
3. Columns shall be checked for shear strength.
4. Shear walls shall be sized for elastic strength considerations in shear.
5. Shear walls shall be checked for tension from overturning moment.

Examination of the set of forces on the columns and shear wall will facilitate understanding of the design simulation process. For the frame (Figure A3)

$$M_i^f = \frac{6EI_1^f}{h^2} (w_i - w_{i-1}) + \sum_{j=i}^N W_j (w_i - w_{i-1}) \pm \sum_{j=i}^N F_j (j-i+1) h (w_i - w_{i-1}) / \delta_i$$

$$M_1^f = \frac{6EI_1^f}{h^2} w_1 + \sum_{j=1}^N W_j w_1 \pm \sum_{j=1}^N F_j (jh) w_1 \delta / \delta_1 \quad i = 2, 3, \dots, N \quad [\text{Eq A19}]$$

where the first term is the bending moment induced in a fixed-ended

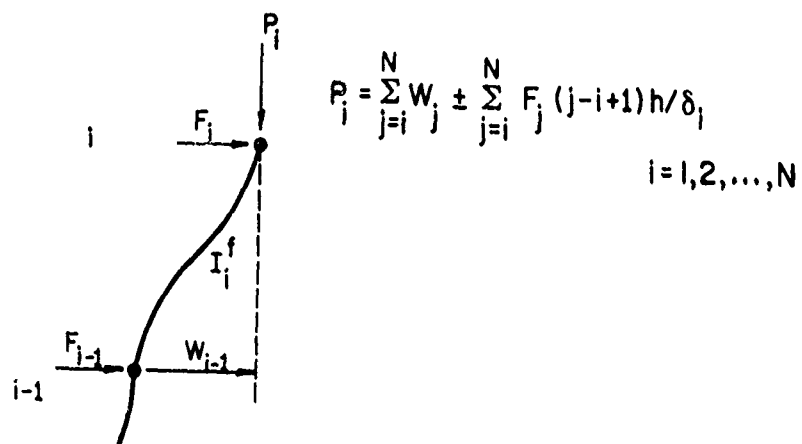


Figure A3. Column axial load relationship for design moments.

column from story displacement, the second term is the bending moment induced by the dead and live load acting eccentrically on the column, and the third term is the compression or tension induced by the overturning moment acting eccentrically on the column with a moment arm of δ_i . The expressions for shear in the frame are as follows:

$$V_i^f = \frac{12EI_i^f}{h^3}(w_i - w_{i-1}) \quad i = 2, 3, \dots, N$$

$$V_1^f = \frac{12EI_1^f}{h^3} w_1$$

[Eq A20]

For axial loads on the frame, the following expression is apparent from Figure A3:

$$P_i^f = \sum_{j=i}^N W_j \pm \sum_{j=i}^N F_j(j-i+1)h/\delta_i \quad i = 1, 2, \dots, N$$

[Eq A21]

For the shear wall (Figure A4)

$$M_i^S = \frac{-EI_i^S}{h^2}(w_{i+1} - 2w_i + w_{i-1}) + \text{SGN ABS} \left(\sum_{j=i}^N 6W_j(w_i - w_{i-1}) \right. \\ \left. \pm \sum_{j=i}^N F_j(j-i+1)h(w_i - w_{i-1})/\delta_i \right)$$

[Eq A22]

$i = 2, 3, \dots, N-1$

$$M_1^S = -\frac{EI_1^S}{h^2}(w_2 - 2w_1) + \text{SGN ABS} \left(\sum_{j=1}^N 6W_j w_1 \right. \\ \left. \pm \sum_{j=1}^N F_j j h w_1 / \delta_1 \right), \quad M_N^S = 0$$

where again the first term is due to bending moment induced in the shear wall resulting from story displacement, the second term is bending moment induced by dead and live loads acting eccentrically on the wall, and the third term is the compression or tension induced by the overturning moment acting on the wall eccentrically with an arm $c \approx \delta_i$.

The dead load and overturning moment increase the internal moment on the frame and shear wall members. The signs of the first, second,

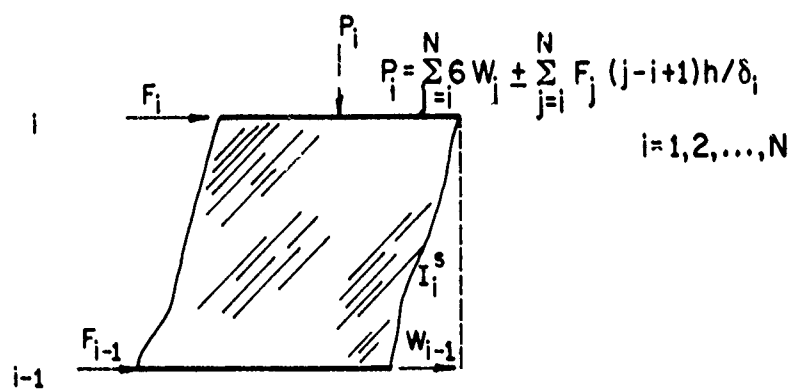


Figure A4. Shear wall axial load relationship for design moments.

and third terms of Eqs A19 and A22 are in agreement.

The expressions for shear in the shear wall are as follows:

$$V_i^S = \frac{EI_i^S}{h^3} \left\{ -w_{i+1} + \left(2 + \frac{I_{i-1}^S}{I_i^S}\right) w_i - \left(1 + 2\frac{I_{i-1}^S}{I_i^S}\right) w_{i-1} + \frac{I_{i-1}^S}{I_i^S} w_{i-2} \right\}$$

$$V_1^S = \frac{EI_1^S}{h^3} \{4w_1 - w_2\} \quad [\text{Eq A23}]$$

$$V_N^S = \frac{EI_{N-1}^S}{h^3} \{w_N - 2w_{N-1} + w_{N-2}\}$$

For axial loads on the shear wall, the following expression is apparent from Figure A4:

$$P_i^S = \sum_{j=i}^N 6W_j \pm \sum_{j=i}^N F_j(j-i+1)h/\delta_i \quad i = 1, 2, \dots, N \quad [\text{Eq A24}]$$

The design algorithm currently considers three types of structural systems with three load distribution configurations: a ductile moment-resisting space frame (K equals 0.67), a dual bracing system with a ductile moment-resisting frame (K equals 0.80), and a box system (K equals 1.33). For the system where K equals 0.67, 100 percent of the dead, live, and lateral loads on the structure are carried by the frame. In the cases where K equals 0.80 or 1.33, the frame is assumed to carry 25 percent of the dead, live, and lateral loads, and the shear wall system is designed to resist 100 percent of the dead, live, and lateral loads. It is important to note that in these two cases, the load on the frame members is determined on the basis of 100 percent dead, live, and lateral load. The internal forces in the member are then scaled to 25 percent, and the resistance for an adequately designed member must be larger than this value. This is a somewhat different procedure from establishing the total member internal forces for 25 percent of the applied loads and proceeding to design the member on this basis. In effect, the frame and shear wall system are designed together instead of individually for certain percentages of load.

The lateral displacements of frame and shear wall systems are compatible, resulting in a somewhat different distribution of moment and shear forces between the frame and shear wall elements than if an individual member design procedure was employed. This simplified approach should provide designs comparable to those obtained by individually designing the frame and shear wall members. Individual design of frame and shear wall members would have proved more cumbersome for this initial model and certainly not convenient for defining the structural configurations in terms of ratios of moments of inertia of structural elements.

The frame design is considered critical in the simulated design. For even moderate ratios of shear wall to frame moments of inertia, the shear wall system easily meets the design requirements.

Columns within the model are sized on the basis of ultimate strength considerations for columns under both axial load and bending moment. From Winter,¹⁰ the ultimate compressive and moment values are determined from the interaction diagram of Figure A5.

For

$$p_t m = (A_s + A'_s) f_y / (0.85 f'_c b t) \quad [\text{Eq A25}]$$

where A_s = area of tension steel
 A'_s = area of compression steel
 b = column width
 t = column depth
 f_y = yield strength of steel
 f'_c = yield strength of concrete,
 and

¹⁰ G. Winter et al., *Design of Concrete Structures* (McGraw-Hill, 1964).

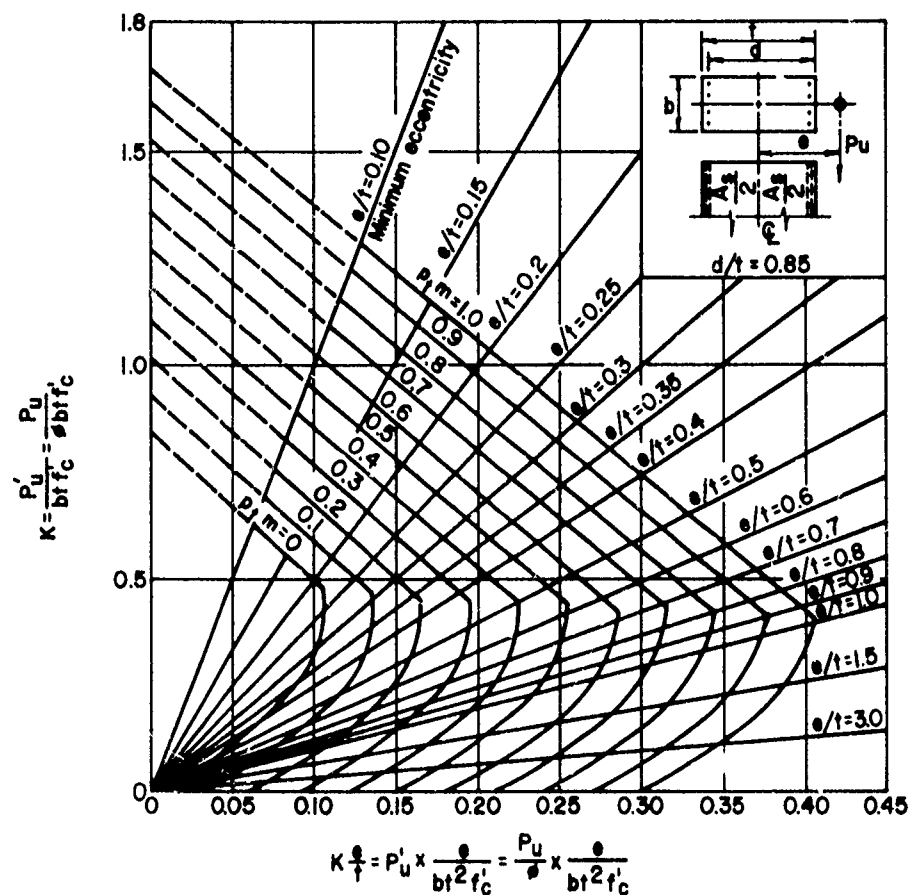


Figure A5. Bending and axial load-rectangular sections. Reprinted with permission of the American Concrete Institute, from C. S. Whitney and Edward Cohen, "Guide for Ultimate Strength Design of Reinforced Concrete," *Journal of the American Concrete Institute*, Vol 28, No. 5 (November 1956), p 455.

$$e = \frac{M_u}{P_u} \quad [\text{Eq A26}]$$

where e = the eccentricity ratio

M_u = ultimate moment capacity of the column

P_u = ultimate axial force capacity of the column,

the column design algorithm consists of the following steps:

Step 1. Compute the column eccentricity from the ratio of internal forces M and P :

$$e = \frac{M}{P} \quad [\text{Eq A27}]$$

Step 2. If e/t is less than 0.1, set e equal to 0.1.

Step 3. Compute the column reduction factor R from

$$R = 1.07 - 0.008 (h/0.3b) \quad [\text{Eq A28}]$$

where h = the column height

b = the column width

Step 4. Compute the point of the intersection of the curves $p_t m$ and $e/t = \text{constant}$ and determine the interaction diagram column design parameters (K_p and K_m) from Figure A5. A ratio of column effective depth to width (ϕ_m) of 0.85 has been selected. The ultimate strength of the column for axial load becomes

$$P_u = K_p \phi_m b t f'_c \quad [\text{Eq A29}]$$

and the ultimate strength of the column for bending becomes

$$M_u = P_u e = K_m \phi_m b t^2 f'_c \quad [\text{Eq A30}]$$

Step 5. For the column the following design checks are made:

- a. If M_u is less than M , the column dimensions are increased.
- b. If M_u is greater than TM , the column dimensions are decreased. T is a convergence factor varying from 1.5 to infinity depending on the extent to which the member resistance is allowed to exceed the requirement.
- c. If P_u is less than P , the column dimensions are increased.
- d. If P_u is greater than TP , the column dimensions are decreased.
- e. For shear, if $\phi_s 2\sqrt{f_c} b^2$ is less than V , the column dimensions are increased.
- f. For tension, if $4p f_y b^2$ is less than

$$\sum_{j=i}^N F_j (j-i+1)h/\delta_i - \sum_{j=i}^N W_j$$

the column dimensions are increased.

If any of the column dimensions are changed in Step 5, the new structural configuration is computed and the process repeated. The process usually takes several iterations to converge. Once satisfactory convergence has been obtained, the design has been simulated.

In the shear wall design algorithm, the wall configuration is assumed to have fixed dimensions and the design parameter is the percent steel in the wall. The wall provides shear resistance and resistance to overturning. The nominal shear stress in the shear wall is given by

$$V_u = \phi_s (2\sqrt{f_c} + 0.5 p_s f_y) A^s \quad [\text{Eq A31}]$$

where ϕ_s = capacity reduction factor (0.85)

V_u = shear resistance

f_c = compression strength of concrete

P_s = percent steel in the shear wall

f_y = yield strength of steel

A^S = area of the shear wall.

The shear stress in the wall cannot be greater than this value. If the value is exceeded, the percentage of steel in the wall is increased. If the resistance is exceeded for a certain convergence option, the percentage steel in the wall is decreased.

The overturning moment is checked by requiring the wall resistance to exceed

$$P_u = 0.5 p_s f_y A^S / 2.0 \quad [\text{Eq A32}]$$

where P_u = overturning moment resistance

P_s, f_y , and A^S = as defined above.

Here again, percentage steel in the wall is increased for a design not exceeding P_u . Based on these expressions, the design is simulated for a given structural configuration and lateral force requirement. The design algorithm may be further complicated by constraining the design frame and shear wall moments of inertia. It is possible, and in the past has been useful, to require a frame member at level i to have a percentage of the moment of inertia of a frame member at level 1. It is also convenient to have constant moment of inertia shear walls over N stories.

The degree to which the simulated design agrees with the actual design is an indication of accuracy of the simulation model. However, the designs may not agree perfectly because (1) the simulation design is devoid of detail and built around a very simple model that behaves only approximately like the real structure, and (2) the goal is relative rather than absolute decision information. Since all the simulated designs are generated in the same way, the relative nature of the decision information is preserved.

The Costing Algorithms

Transforming structural behavior into information pertinent to the decision-making process requires algorithms that extract information from this behavior and transform it into information that can be interpreted in terms of cost. The computer program for the simulation model contains three basic cost algorithms--a facility damage/repair cost algorithm, a facility strengthening cost algorithm, and a facility replacement cost algorithm. The rationale behind the development of each of these algorithms is discussed to provide insight into the generation of information for decision making. Whenever cost is referred to, it is assumed that the cost is the construction cost for the structure.

Damage/Repair

The facility damage/repair cost algorithm is based on the computation of a quantity called the damage factor. This damage factor relates both the code base shear requirement for the facility and the structure's behavior to the amount of damage that structural and nonstructural components might be expected to sustain if the design code requirements are exceeded. The damage factor is thus directly related to the difference in code requirements.

It is well known that structures designed under different levels of code requirements either for different seismic zones or at different times experience damage when subjected to seismic activity exceeding the design requirement, i.e., when the demand on the structure exceeds its capacity. Some of this experience is quantified in Whitman's¹¹ study of the effects of earthquakes of different magnitudes on structures designed in accordance with the Uniform Building Code (UBC).¹² It is also well known that the inelastic behavior of the

¹¹ R. V. Whitman, *Methodology and Pilot Application*, R74-15, Report No. 10, Structures Publication 385 (Department of Civil Engineering, MIT, July 1974).

¹² Uniform Building Code (International Conference of Building Officials, 1973).

materials in a structure and the action of both structural and non-structural elements provide the structure with a reserve energy; these actions are often not fully accounted for in the design because of a lack of knowledge of their precise behavior. Blume¹³ has considered damage to structures using a method termed the Reserve Energy Technique. This method relates the hysteretic elastic energy capacity of a structure under load to the true combination elastic-inelastic energy capacity available in the structure. Using this concept, it is possible to relate the code design requirement demand, the current facility capacity, and the capacity a structure may be required to have under a higher demand to the damage a structure may be expected to see under higher demands. For a structure that behaves as an elastoplastic, one-dimensional model with a ductility ratio μ ,

$$\mu = \frac{\Delta^*}{\Delta_e} \quad [\text{Eq A33}]$$

where Δ^* = a measure of capacity of a structure before collapse

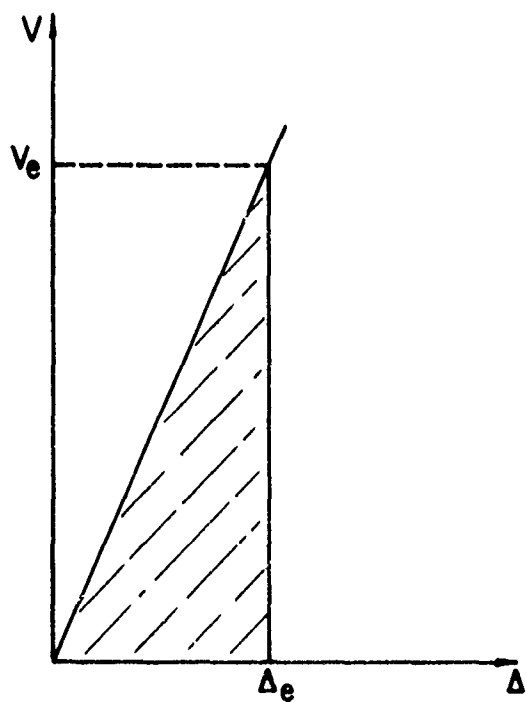
Δ_e = a measure of capacity of a structure in the elastic range.

From Figure A6, equating the energy under the demand "deflection" curves provides an expression for the structure's ductility in terms of the demand D_s on the structure and the capacity C_s of the structure:

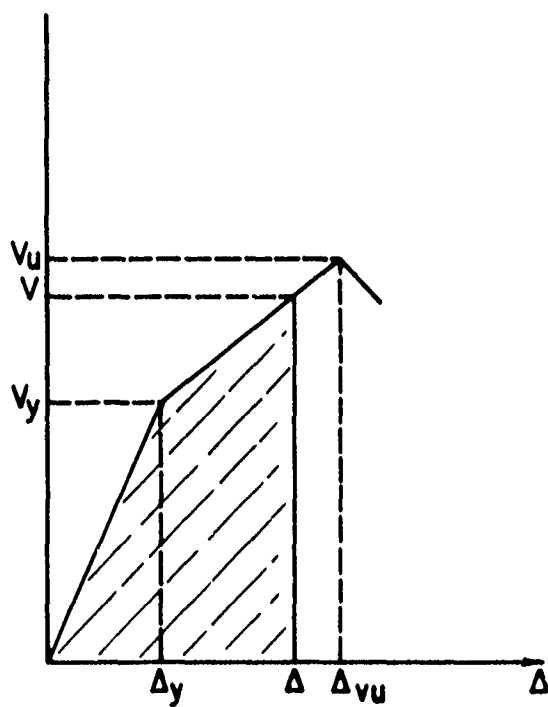
$$\mu = \frac{1}{2} \left[\left(\frac{D_s}{C_s} \right)^2 + 1 \right] \quad [\text{Eq A34}]$$

Once the structure's capacity has been determined (in the case of the simulation model, the capacity is merely the design base shear) and the subsequent demand on the structure is identified (for the simulation model this is an anticipated base shear the structure may be called upon

¹³ *Effects Prediction Guidelines for Structures Subjected to Ground Motion*, JAB-99-115, UC-11 (URS/John A. Blume and Associates, Engineers, July 1975).



Elastic Demand Model



Inelastic Capacity Model

Figure A6. Demand and capacity energy models.

to withstand), Eq A34 defines the extent the reserve energy will be called upon to withstand the demand. The ductility μ provides a relative measure of the extension of the structure behavior into the inelastic region. Since it is merely a relative measure, a formula is needed to translate the measure into an absolute damage variable for the structure. Blume suggests the following relationship:

$$DF_{SNS} = \begin{cases} 0 & \mu \leq 1 \\ \left(\frac{\mu-1}{u_{\mu}-1}\right)^k & 1 \leq \mu \leq u_{\mu} \\ 1 & 1 \leq \mu \end{cases} \quad [\text{Eq A35}]$$

where DF_{SNS} = structure damage factor

μ = required ductility to meet the demand

u_{μ} = ultimate ductility of the structure

k = an "economic factor."

For purposes of uniformity in description, u_{μ} is written as calibration factor c_1 and k is written as calibration factor c_2 . Thus, DF_{SNS} becomes

$$DF_{SNS} = \begin{cases} 0 & \mu \leq 1 \\ \left(\frac{\mu-1}{c_1-1}\right)^{c_2} & 1 \leq \mu \leq c_1 \\ 1 & c_1 \leq \mu \end{cases} \quad [\text{Eq A36}]$$

u_{μ} is a relative measure of the capacity of a structure to the point of collapse. A DF_{SNS} value of 0 implies that the demand on the structure was below the design capacity, and for an elastic design, no inelastic reserve energy was required. For DF_{SNS} values greater than 0 but less than 1, some inelastic reserve energy being used to resist the demand resulted in damage to the structure. For DF_{SNS} values of 1, the structure has collapsed. In terms of the computer simulation model, D_s and C_s are the structure's required and designed base shear, k and μ_u are calibration constants, and DF_{SNS} is an absolute measure of structural damage to both structural and nonstructural components.

The damage factor DF_{SNS} is not the only possible measure of

structural and nonstructural damage. Steinbrugge¹⁴ and others have indicated that there is a high degree of correlation between relative story displacement and nonstructural damage (e.g., cracks in walls, glass breakage, etc.). Whereas DF_{SNS} provides an overview of absolute structural/nonstructural damage for a one-dimensional elastoplastic model of the structure, a second damage factor examines the structure behavior in more detail to arrive at another relative measure of damage for nonstructural elements of the structure. For the simulated design under the design load, the relative story displacement is merely the difference in story displacements between the top and bottom of the story. For the initial design subject to a higher demand requirement than it was originally designed for, the elastic relative story displacement can be computed. Utilizing an expression similar to the one above for ductility gives the following expression for the nonstructural damage factor:

$$DF_{NS} = \begin{cases} 0 & \Delta < 0 \\ \left(\frac{\Delta}{D_r c_3}\right)^{c_4} & 0 \leq \Delta/D_r < c_3 \\ 1 & c_3 \leq \Delta/D_r \end{cases} \quad [Eq A37]$$

where DF_{NS} = partial nonstructural damage factor

Δ = elastic relative story displacement for higher demand

D_r = story drift criteria

c_3, c_4 = calibration constants.

Since both DF_{NS} and DF_{SNS} relate to damage from seismic loads, they are interrelated. Preselected weighting factors are used to weight the effects of the two factors in obtaining the overall damage factor expression. The expression for the total damage factor is thus

$$DF = w_1 DF_{SNS} + w_2 DF_{NS} \quad [Eq A38]$$

¹⁴ K. V. Steinbrugge et al., *Studies in Seismicity and Earthquake Damage Statistics*, COM-71-00053 (National Oceanic and Atmospheric Administration, Department of Commerce, 1969), Appendix A.

where DF = total damage factor

DF_{SNS} = overall design structure damage factor

DF_{NS} = relative story displacement damage factor

w₁, w₂ = weighting factors.

For purposes of this study, w₁ equals 0.75 and w₂ equals 0.25. DF is based upon four calibration constants that are determined from real data, and the weighting factors quantify the relationships between the two damage factors.

The cost of damage/repair is proportional to DF. The structural damage/repair costs for the ith floor (SC_i) are written

$$SC_i = CSS \cdot DF_{SNS} \cdot AR_i \quad [Eq A39]$$

where CSS = structural costs per square foot of facility

AR_i = area of the ith floor of the facility

DF_{SNS} = as defined above.

The nonstructural damage/repair costs for the ith floor (NSC_i) are written

$$NSC_i = CNS \cdot DF \cdot AR_i \quad [Eq A40]$$

where CNS = nonstructural costs per square foot of facility

DF, AR_i = as defined above.

The total structural and nonstructural damage/repair cost for a given facility is merely the summation over the floor levels. The rationale behind these expressions for cost is based on the difficulty in separating the structural and nonstructural damage in DF_{SNS}. Thus, both SC and NSC contain indeterminate nonstructural and structural costs.

Strengthening

The facility strengthening model is based on the quantity of stiffness that must be added to the facility to bring its displacements into agreement with those of the facility designed for the higher requirements. That is, the added stiffness K_{ss} (in matrix form) is

$$K_{SS} = K_S - K_i \quad [\text{Eq A41}]$$

where K_i = the stiffness matrix for the original structure

K_S = the stiffness matrix for a newly designed facility under the higher requirement.

Stiffness is of the form CEI/ℓ^3 where C is a constant, E is the material modulus, I is the section's moment of inertia (the only term which varies), and ℓ is the length of the dimension. The square root of the I value is proportional to area and is taken proportional to the cost of the added material to the facility, thus defining relative costs. In order to "normalize" the cost for strengthening over the facility, the added stiffness is divided by the average stiffness for the original simulated facility design. This is accomplished by computing the average of diagonal terms of K_i , and dividing into each of the diagonal terms of K_{SS} . In a parallel argument to that used in the damage/repair model, a stiffening factor for both structural and nonstructural components is defined in terms of the following ratio of stiffness expressions for the i^{th} story:

$$\text{stiffening factor} = \sqrt{K_{SS}(i,i)/\bar{K}_i} \quad [\text{Eq A42}]$$

where $K_{SS}(i,i)$ = i^{th} diagonal term of the strengthening stiffness matrix
 \bar{K}_i = average stiffness of the original simulated design
 (average of the diagonal terms of K_i).

Structural costs are proportional to the structural strengthening factor defined by

$$SF_S = c_5 \sqrt{K_{SS}(i,i)/\bar{K}_i} \quad [\text{Eq A43}]$$

where SF_S = structural strengthening factor

c_5 = calibration constant

$\sqrt{K_{SS}(i,i)/\bar{K}_i}$ = as defined above.

Nonstructural costs are proportional to the nonstructural strengthening factor defined by

$$SF_{NS} = c_6 \sqrt{K_{SS}(i,i)/\bar{K}_i} \quad [\text{Eq A44}]$$

where SF_{NS} = nonstructural strengthening factor
 c_6 = calibration constant
 $\sqrt{K_{SS}(i,i)/\bar{K}_i}$ = as defined above.

The total strengthening factor (SF) is merely the sum of SF_S and SF_{NS} :

$$SF = SF_S + SF_{NS} \quad [\text{Eq A45}]$$

The cost of strengthening is proportional to the sum of the individual strengthening factors times the facility area times the cost per unit area of the original facility. Thus, the structural strengthening cost for the i^{th} floor is written

$$SC_i = CSS \cdot SF_S \cdot AR_i \quad [\text{Eq A46}]$$

and the nonstructural strengthening cost for the i^{th} floor is written

$$NSC_i = CNS \cdot SF_{NS} \cdot AR_i \quad [\text{Eq A47}]$$

The total structural and nonstructural cost for a given facility is merely the summation over the floor levels.

Replacement

The facility replacement model is based on measuring the increase in material needed in a simulated design for higher requirements. Since the lateral-force-resisting elements such as columns and shear walls generally require greater material quantities to withstand greater lateral forces, the increase in area of column elements and the increase in steel ratio for shear wall elements are taken to be proportional to the cost of facility replacement.

The facility replacement factor for both structural and nonstructural elements is defined by

$$RF = c_7(b_1d_1 - b_0d_0) + c_8(p_1 - p_0) \quad [Eq A48]$$

where RF = replacement facility factor

c_7, c_8 = calibration constants for frame and shear wall quantities respectively

b_0, d_0 = simulated column design width and depth, respectively, for the original facility

b_1, d_1 = simulated column design width and depth, respectively, for the replacement facility

p_0 = simulated shear wall percentage steel for the original facility

p_1 = simulated shear wall percentage steel for the replacement facility.

Available data indicate that 82 percent of the replacement cost can be attributed to structural costs and the remaining 18 percent to nonstructural costs.¹⁵ Thus, the incremental cost IC for the i^{th} story may be written as follows:

$$IC = RF \cdot AR_i \cdot CSS/0.82 \quad [Eq A49]$$

The total structural cost then becomes

$$SC_i = 0.82 \cdot IC + AR_i \cdot CSS \quad [Eq A50]$$

and the total nonstructural cost becomes

$$NSC_i = 0.18 \cdot IC + AR_i \cdot CNS \quad [Eq A51]$$

¹⁵ R. V. Whitman, *Methodology and Pilot Application*, R74-15, Report No. 10, Structures Publication 385 (Department of Civil Engineering, MIT, July 1974).

Again, the total structural and nonstructural cost for a given facility is merely the summation over the floor levels.

Summary

Perhaps the most useful cost information is provided by the average of the floor level costs. The computer program not only provides this information but also the cost as a percentage of the original facility cost, as used in the example in Chapter 4. Knowing the original facility cost provides a basis for comparing costs to repair, strengthen, or replace over a variety of different facility configurations.

APPENDIX B:

MODEL CALIBRATION

Damage/Repair Algorithm

Four constants must be determined for calibration of the algorithm for damage/repair--two for DF_{SNS} and two for DF_{NS} . In the expression for DF_{SNS} (Eq A36), c_1 represents the building's ultimate ductility and c_2 is an "economic factor." Section 3.3 of the American Concrete Institute (ACI) report entitled *Response of Multistory Concrete Structural Lateral Forces*¹⁶ indicates that a recommended minimum ductility factor for reinforced concrete buildings in earthquake areas is 4.0 to 6.0. For estimating damage, then, using an ultimate ductility factor of 4.0 would be conservative. It should be noted, however, that determination of a building system's ultimate ductility is difficult, and any number assigned to broad classes of buildings may be subject to considerable error.

An MIT¹⁷ study of four specially designed reinforced concrete buildings to determine a maximum ductility factor for buildings designed for several UBC requirements and a "super zone" (twice UBC Zone 3) subject to a 0.27 g ground acceleration, obtained lower values for maximum ductility in most cases. Table B1 presents a portion of the MIT data. These data seem to indicate, for example, that a six-story concrete moment-resisting frame building designed for UBC 3 would have a ductility factor of 3.8 when subjected to a 0.27 g peak ground acceleration. Moving from UBC 0 design to UBC 2 design, in general, decreases the maximum ductility inherent in the building. The last two columns

¹⁶ *Response of Multistory Concrete Structures to Lateral Forces*, ACI SP-36 (American Concrete Institute, 1973).

¹⁷ R. V. Whitman, *Methodology and Pilot Application*, R74-15, Report No. 10, Structures Publication 385 (Department of Civil Engineering, MIT, July 1974).



Table B1

Maximum Ductility Factors Caused by Ground Motion
With 0.27 g Peak Acceleration*

Building**	Stories	Maximum Ductility Factor μ_{max}					
		UBC 0	UBC 2	UBC 3	S	$\bar{\mu}_{max}$	σ_{max}
CMRF	6	[4.6] [†]	[4.6]	[3.8]	2.8	4.3	0.5
CMRF	11	[3.5]	[3.5]	2.7	1.2	3.2	0.5
CSW	11	[8.6]	[6.5]	2.2	1.7	5.8	3.3
CSW	17	[3.3]	[3.1]	2.0	1.0	-	-

* From R. V. Whitman, *Methodology and Pilot Application*, R74-15, Report No. 10, Structures Publication 385 (Department of Civil Engineering, MIT, July 1974).

** CSW = Concrete Shear Wall.
CMRF = Concrete Moment-Resisting Frame.

†  = probable partial or total collapse.
 = possible partial or total collapse.

of Table B1 give the mean and standard deviations of the maximum ductility ratios of the concrete moment-resisting frame buildings of six and 11 stories and the concrete shear wall building of 11 stories (configurations in Table B1 corresponding to configurations developed in the model) over the UBC 0, UBC 2, and UBC 3 design strategies. Since c_2 will be determined over the number of building stories without reference to a moment-resisting frame or a shear wall configuration, to be consistent, c_1 must also be determined on this basis. For the six-story concrete moment-resisting frame, the mean and standard deviation over the first three design strategies are 4.3 and 0.5, respectively. For the 11-story concrete moment-resisting and shear wall configurations, the mean and standard deviation of the maximum ductility ratio were calculated from the values in the table to be 4.5 and 2.5, respectively. If ultimate ductility is assumed to correlate with "probable partial or total collapse" of Table B1, then both 4.3 and 4.5 are low, since both are weighted toward "possible partial or total collapse"--the former more so than the latter. In addition, the data to be used in determining c_2 are probably weighted toward the lower design strategy requirements (UBC 0, UBC 2), also implying an increased mean maximum ductility factor. For simplicity, a value of ultimate ductility of 5.0 has been selected for all building configurations of Table B1. The uncertainties in the data do not allow a more refined estimate.

A preliminary determination of c_2 was made from the data on the San Fernando earthquake compiled at MIT,¹⁸ which provides perhaps the most extensive data base available on damage to structures from seismic activity. From Blume's work,¹⁹ a structure's ductility is expressed in terms of its design capacity and the demand placed on it (Eq A34). The damage factor can be expressed in terms of the ratio of the demand on

¹⁸ R. V. Whitman, *Damage Probability Matrices for Prototype Buildings*, R73-57, Report No. 8, Structures Publication 380 (Department of Civil Engineering, MIT, October 1973).

¹⁹ *Effects Prediction Guidelines for Structures Subjected to Ground Motion*, JAB-99-115, UC-11 (URS/John A. Blume and Associates, Engineers, July 1975).

the structure to the structure's design capacity. Here, μ is assumed to represent the structure ductility at demand D_s for a given capacity C_s . The expression for the damage factor can be written as follows:

$$\log (DF_{SNS}) = c_2 \log \left(\frac{\mu-1}{c_1-1} \right) \quad 1 < \mu \leq c_1 \quad [\text{Eq B1}]$$

If c_1 and μ are known in terms of the demand to capacity ratio, then a relationship between the damage factor for various demands and c_2 can be developed. If the damage factor can be correlated with a demand, the value of c_2 can be determined.

The MIT data base on the San Fernando earthquake is expressed in terms of damage probability matrices. Damage probability matrices (Figure B1) express the damage state of a structure in terms of the percentage of structures in that state (P_{DSI}) for a particular building configuration and a particular demand level expressed in terms of MMI. The damage state can be correlated with the damage ratio central value, as in Table B2. The MMI can be correlated with the nominal peak ground acceleration, as in Table B3. Thus, the damage a structural configuration may be expected to experience (defined by the percentage of structures in a particular damage state) can be related to a value of nominal peak ground acceleration (a reasonable measure for defining seismic activity). A particular damage probability matrix can be summarized at a given intensity by computing a mean damage ratio (MDR). This weighted average over the damage states is defined as follows:

$$mDR_I = \frac{1}{n_I} \sum_{i=1}^{n_I} DR_{iI} \quad [\text{Eq B2}]$$

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

DR_{iI} = damage ratio (cost of repair to cost of replacement) for the i^{th} building in a particular category subject to ground motion intensity I .

This number provides a measure of the ratio of cost to repair to cost

DAMAGE STATE	CENTRAL DAMAGE RATIO, %	MM INTENSITY				
		VI	VII	VIII	IX	X
0 - NONE	0					
1 2 - LIGHT	0.3		P_{DSI}			
3 4 5 - MODERATE	5					
6 - HEAVY	30					
7 - TOTAL	100					
8 - COLLAPSE	100					

Figure B1. Form of damage probability matrix.

Table B2

Correlation of Damage State to Damage Ratio Central Value

<u>Damage State</u>	<u>Damage Ratio* Central Value, %</u>
0	0
1	0.1
2	0.5
3	2.0
4	5.0
5	10.0
6	30.0
7	100.0
8	100.0

$$\text{*Damage Ratio} = \frac{\text{cost of repair}}{\text{cost of replacement}}$$

Table B3

Correlation of MMI to Nominal Peak Ground Acceleration

<u>MMI</u>	<u>Nominal Peak Ground Acceleration, g</u>
IV	0.007
V	0.015
VI	0.030
VI.5	0.050
VII	0.090
VIII	0.200
IX	0.500
X	1.020

of replacement for the particular building configurations that are used to develop the matrix. For computing c_2 , the mean damage ratio is taken to be the damage factor DF_{SNS} .

The capacity of the structure is assumed to be 1968 SEAOC Zone 3 design. Then, the base shear for the design (V) may be expressed

$$V = KCZW \quad [Eq B3]$$

where K = structural system type coefficient

C = structural system dynamic coefficient

Z = seismic zone factor

W = total weight of structure.

Its capacity (U) can be expressed as

$$U = 1.4V = 1.4KCZW \quad [Eq B4]$$

To use the information from the MIT study, it is necessary to correlate response spectra with MMI. For 3 percent damping, Figure B2 correlates the elastic response spectra bounds with MMI based on the correlation of MMI with nominal peak ground acceleration provided in Table B3. A response spectrum approach is used to establish the demand on the structure for given peak nominal ground acceleration values. For reference purposes, the 1968 SEAOC Zone 3 capacity and the 1.0 g response spectrum demand values are provided in Table B4 over the four-, seven-, and ten-story configurations, two dimensional ratios, and three structural systems. The capacity of the facility can be read from the table and the demand scaled from the value corresponding to a 1.0 g response spectrum.

Table B5 presents the required structure ductility based on the 1968 SEAOC Zone 3 capacity and the response spectrum demand determined for MMIs of VI, VII, and VIII. The variation in correlation of MMI with nominal peak ground acceleration, the selection of 3 percent damping for spectra amplification factor computation, and the accuracy in

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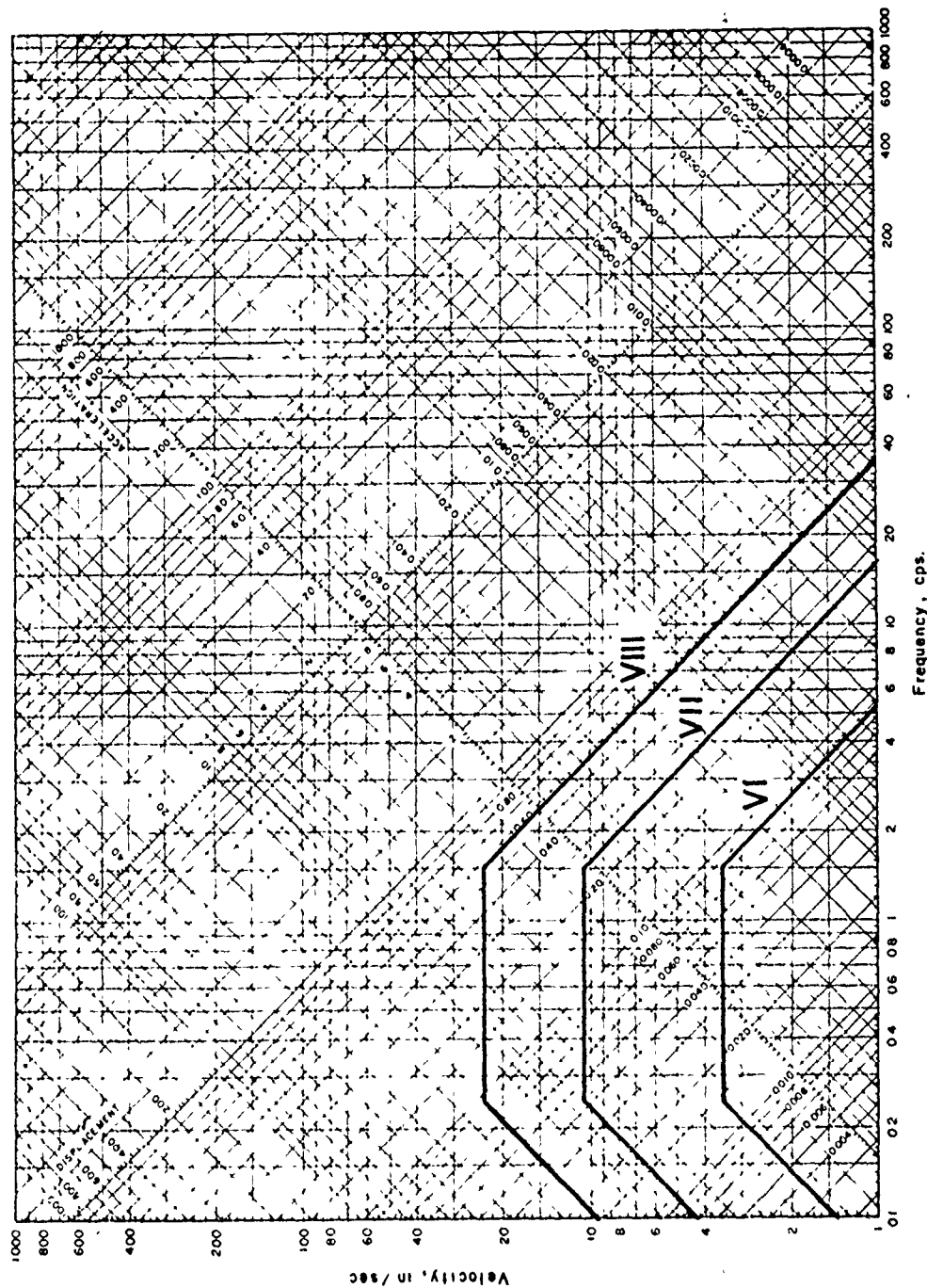


Figure B2. Correlation of elastic response spectra bounds with MMI. SI conversion factor: 1 in./sec = 25.4 mm/sec.

Table B4
Capacity Demand for Various Structural Configurations
Four Story

K	T	Dimensional Ratio	Capacity $C_s = 1.4KCZW$	Demand * $D_s = CS_aW$
0.67	.4	1.0	70,656	1,032,300
0.67	.4	1.6	113,049	1,651,680
0.80	.843	1.0	71,130	1,008,000
0.80	.667	1.6	123,081	1,612,800
1.33	.843	1.0	127,122	825,600
1.33	.667	1.6	219,966	1,320,960

Seven Story

0.67	.7	1.0	101,415	1,722,240
0.67	.7	1.6	162,267	2,755,584
0.80	1.476	1.0	103,293	1,686,300
0.80	1.167	1.6	178,737	2,698,080
1.33	1.476	1.0	186,444	1,383,960
1.33	1.167	1.6	322,617	2,214,336

Ten Story

0.67	1.0	1.0	128,037	2,416,050
0.67	1.0	1.6	204,858	3,865,680
0.80	2.108	1.0	131,022	2,370,000
0.80	1.667	1.6	226,716	3,792,000
1.33	2.108	1.0	237,426	1,929,300
1.33	1.667	1.6	410,835	3,086,880

* In the response spectrum approach, the ratio of C to the response spectrum acceleration (C/S_a) is defined as follows:

<u>k</u>	Stories		
	Four	Seven	Ten
0.67	0.950	0.897	0.885
0.80	0.840	0.803	0.790
1.33	0.640	0.607	0.590

Table B5

Model Ductility Based on Capacity and Demand

Freq Hz	C _s	D _s			μ		
		MMI	MMI	MMI	VI	VII	VIII
		VI	VII	VIII			
Four Story							
2.50	70,656	89,810	269,430	598,734	1.31	7.77	36.40
2.50	113,049	143,696	431,088	957,974	1.31	7.77	36.40
1.19	71,130	67,536	201,600	443,520	0.95	4.52	19.94
1.50	123,081	140,314	420,941	935,424	1.15	6.35	29.38
1.19	127,122	55,315	165,120	363,264	0.59	1.34	4.58
1.50	219,966	114,924	344,771	766,157	0.64	1.73	6.57
Seven Story							
1.43	101,415	139,501	421,949	930,010	1.45	9.16	42.55
1.43	162,267	223,202	675,118	1,488,015	1.45	9.16	42.55
0.68	103,293	65,766	202,356	455,301	.70	3.11	10.21
0.86	178,737	129,508	404,712	863,386	.76	3.06	12.17
0.68	186,444	53,974	166,075	373,669	.54	1.04	2.51
0.86	322,617	106,288	332,150	708,588	.55	1.03	2.91
Ten Story							
1.0	128,037	137,715	422,809	918,099	1.08	5.95	26.21
1.0	204,858	220,344	676,494	1,468,958	1.08	5.95	26.21
0.47	131,022	63,990	189,600	450,300	.62	1.55	6.41
0.60	226,716	128,928	398,160	872,160	.66	2.04	7.90
0.47	237,426	52,091	154,344	366,567	.52	.71	1.69
0.60	410,835	104,954	324,122	709,982	.53	.81	1.99

reading values from the elastic bound response spectrum plots may result in as much as 25 percent error in some of the computed demand values in the table. This is within the accuracy of the model for damage/repair.

The DF_{SNS} , which is taken equivalent to the mean damage ratio of the MIT study, is presented in Table B6 for five to seven story and eight to 13 story buildings for MMIs of VI, VII, and VIII. Table B7 presents the overall damage probability matrices from the MIT study. The structures making up the data sample were constructed prior to 1933 or after 1947. The post-1947 data are presented along with the combination pre-1933 and post-1947 data. In Table B8 the calibration constant c_2 is presented for the four-, seven-, and ten-story buildings over the building configuration and MMIs of VI, VII, and VIII. Blanks occur in the table when the computed ductility is either greater than 5.0 or less than 1.0. Certain cases examined showed high ductilities but very low damage ratios. This is unreasonable and biases the values of c_2 to the high side. Values of c_2 less than 5 are the only ones considered.

Based on averages over each story configuration assuming c_2 to be independent of MMI, the following values of c_2 and their standard deviations are selected:

Four Story	$c_2 = 2.66$	$(\sigma_{c2} = 0.68)$
Seven Story	$c_2 = 2.78$	$(\sigma_{c2} = 1.67)$
Ten Story	$c_2 = 2.18$	$(\sigma_{c2} = 0.85)$

For the portion of the damage/repair algorithm related to relative story displacement (Eq A37), c_4 is taken to be 1.0, i.e., nonstructural damage related to the relative story displacement is a linear function of the story displacement. There are no data and/or guidance available that indicate otherwise.

To complete calibration of this portion of the algorithm, c_3 is determined from some sparse data relating relative story displacement

Table B6

Mean Damage ratio (DF_{SNS}) for Concrete Buildings of Five to Seven
and Eight to 13 Stories for MMIs of VI, VII, and VIII

Five to Seven Story

<u>MMI</u>	DF_{SNS}	
	<u>Post-1947</u>	<u>All</u> *
VI	.0002	.0002
VII	.0105	.0268
VIII	.0267	.0267

Eight to 13 Story

<u>MMI</u>	DF_{SNS}	
	<u>Post-1947</u>	<u>All</u>
VI	.0015	.0011
VII	.0043	.0162
VIII	.0963	.0963

* Includes all data available in the study regardless of construction era.

Table B7

[illegible]

- * R. V. Whitman, et al., *Damage Statistics for High-Rise Buildings in the Vicinity of the San Fernando Earthquake*, R73-24, Report No. 7, Structures Publication 363 (Department of Civil Engineering, MIT, April 1973).
- ** Data for damage states expressed in percentag^r. MDRs and standard deviations are not expressed in percentage.
- † Mean damage ratio.

Table B8

c₂ Computed on the Basis of MIT San Fernando
Earthquake Data and DF_{SNS}

	Post-1947	All	Post-1947	All	Post-1947
	<u>VI</u>	<u>VI</u>	<u>VII</u>	<u>VII</u>	<u>VIII</u>
Four Story	3.33	3.33	-	-	-
	3.33	3.33	-	-	-
	-	-	35.64	28.31	-
	2.59	2.59	-	-	-
	-	-	1.85	1.47	32.66
	-	-	2.68	2.13	-
	Post-1947	All	Post-1947	All	Post-1947
	<u>VI</u>	<u>VI</u>	<u>VII</u>	<u>VII</u>	<u>VIII</u>
Seven Story	3.90	3.90	-	-	-
	3.90	3.90	-	-	-
	-	-	4.40	3.49	-
	-	-	6.87	5.45	-
	-	-	-	-	3.72
	-	-	.93	.74	4.90
	Post-1947	All	Post-1947	All	Post-1947
	<u>VI</u>	<u>VI</u>	<u>VII</u>	<u>VII</u>	<u>VIII</u>
Ten Story	1.66	1.74	-	-	-
	1.66	1.74	-	-	-
	-	-	2.75	2.08	-
	-	-	4.05	3.06	-
	-	-	-	-	1.33
	-	-	-	-	1.68

to a mean damage ratio from the San Fernando earthquake. Figure B3 presents these data. It should be noted here that the data in Figure B3 include both structural and nonstructural damage. However, for purposes of this model, the structural damage is assumed to be insignificant so that the damage ratio is very nearly proportional and on the same order of magnitude as DF_{NS} . In reference to Figure B3, the MIT study states, "the results appear to fall into two groups: One group with larger damage from two buildings with many stiff and brittle partitions not isolated from the structural frame, and a second group with smaller damage from buildings in which there were either few partitions or with flexible and/or isolated partitions."²⁰ The data are weighted heavily toward nonstructural damage. By fitting a straight line to the data, DF_{NS} may be expressed in terms of the relative story displacement as follows:

$$DF_{NS} = 0.0648\delta + 0.0016 \quad [\text{Eq B5}]$$

where δ = relative story displacement in feet.
Thus, solving for δ , realizing that

$$D_r = 0.06h \quad [\text{Eq B6}]$$

where h is the story height in feet, and substituting into Eq A37 yields

$$c_3 = \frac{DF_{NS} - 0.0016}{(.0648)DF_{NS}D_r} \quad [\text{Eq B7}]$$

Table B9 tabulates c_3 for various values of DF_{NS} for a building with story height of 10.0 ft (3.0 m). From the values in this table,

²⁰ R. V. Whitman, *Damage Probability Matrices for Prototype Buildings*, R73-57, Report No. 8, Structures Publication 380 (Department of Civil Engineering, MIT, October 1973).

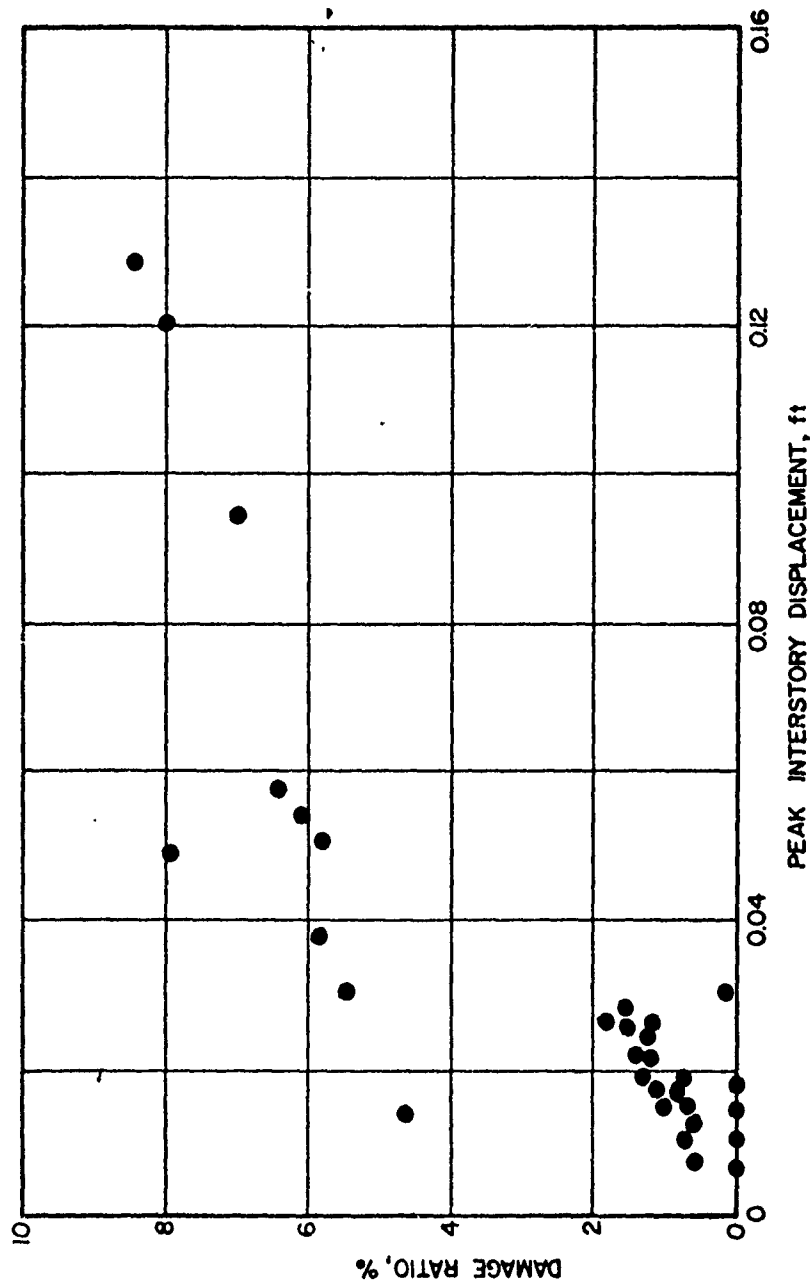


Table B9
Correlation of DF_{NS} With c_3

<u>DF_{NS}</u>	<u>c_3</u>
0.0100	21.60
0.0500	24.90
0.1000	25.31
1.0000	25.68

$\bar{c}_3 = 24.37$, $\sigma_{c_3} = 1.88$, and $COV(c_3) = 0.08$.
 c_3 is taken as \bar{c}_3 or

$$c_3 = 24.37. \quad [Eq\ B8]$$

This completes calibration of the damage/repair algorithm.

Strengthening Algorithm

For the strengthening algorithm, calibration requires the determination of two constants-- c_5 and c_6 --that translate model behavior into strengthening costs (Eqs A43 and A44). The ratio of nonstructural to structural strengthening can be taken as the average of the data on the four facilities shown in Table B10.

For Letterman, Hays, Oakland Naval, and Charleston Naval Hospitals several strengthening schemes with different costs were proposed. Table B11 provides the data based on crude estimates of costs for the four hospital configurations with one workable strengthening scheme for each. The approximate ratio of the capacity of the strengthened facility to the original facility is computed along with a ratio of strengthening cost to initial facility costs. The figures are very approximate based on the best information that could be derived from the reports associated with each of these facility strengthening proposals.²¹ The capacity of the hospitals is often different in two directions. The capacity is taken to be the minimum in this study. The facility capacity ratios do not correlate well with the cost ratios, due in part to the peculiarity of each strengthening scheme.

²¹ *Seismic Safety Investigation of Charleston Naval Hospital at Charleston, South Carolina* (John A. Blume and Associates, November 1974); *Seismic Safety Investigation of Oakland Naval Hospital at Oakland, California* (John A. Blume and Associates, November 1974); P. J. Richter, *Concepts for Strengthening Letterman and Hays Army Hospitals--Task 4C, Aseismic Design Criteria of Existing Hospital Facilities*, Memorandum No. 7338-4C-3241 (Agabian Associates, June 1974).

Table B10

Ratio of Nonstructural Costs to Structural Costs for
Strengthening of Four Critical Facilities

<u>Hospital</u>	<u>CNS/CS</u>
Hays	0.154
Letterman	0.067
Oakland Naval	0.024
Charleston Naval	0.036

Table B11
Strengthening Cost Data

<u>Hospital</u>	<u>Cost to Strengthen (10⁻⁶)</u>	<u>Cost Ratio</u>	<u>Facility Capacity Ratio</u>
Letterman	11.9	.74	6.5
Hays	4.6	.29	5.9
Oakland	10.5	.64	3.6
Charleston	.8	.06	2.8

To arrive at calibration constants for the model, two sets of data from Table B11 are considered. Letterman, Hays, and Oakland are considered as one case for arriving at strengthening costs for the four- and seven-story models, and Letterman and Oakland as a second case for arriving at strengthening costs for ten-story models.* This division was used because the Letterman and Oakland Hospitals correlate with ten-story models, whereas Hays Hospital is better represented by a seven-story model in the current study. However, it would be dangerous to conclude on the basis of the sparse data in Table B11 that low-level buildings have strengthening costs significantly less than those for high-rise buildings (particularly since Hays Hospital is only two to three stories shorter than Letterman or Oakland). The simulation model does, however, seem to indicate a trend in this direction; thus, the data for low-level structures provide a weighting of strengthening costs on all models. A weighted average is therefore the most acceptable alternative for the low-level structures at this time.

The cost to strengthen the ten-story facility model from the simulation is taken to be 69 percent with a 5.1 facility capacity ratio (an average of the values in Table B11 for Letterman and Oakland Hospitals). The cost to strengthen the four- and seven-story facility model from the simulation is taken to be 56 percent with a 5.3 facility capacity ratio.

To calibrate the model for strengthening for ten-story configurations, all ten-story cost data for which k equals 1.33 are averaged for arbitrary c_5 and c_6 . The constants c_5 and c_6 are scaled to the above percentage strengthening cost for the facility capacity ratio. For four- and seven-story configurations, all seven- and ten-story cost data for which k equals 1.33 are averaged for arbitrary c_5 and c_6 with

* Charleston Naval Hospital was not considered further in this study because its comparatively low facility capacity ratio indicates a special case of strengthening not compatible with the other three cases. In fact, significant structural strengthening was not recommended in the case of the Charleston Naval Hospital.

c_5 and c_6 scaled as in the above case. Table B12 provides the strengthening coefficients.

Replacement Algorithm

The replacement algorithm calibration requires the determination of constants c_7 and c_8 in Eq A48. Data for calibration of the model are taken from two sources. Appendix B of a report of the SEAOC Ad Hoc Committee on Costs of Design for Earthquakes²² states that the cost for design and construction of a reinforced concrete structure for UBC Zone 2 is 102 percent of the cost for design and construction for UBC Zone 0. For UBC Zone 3, this publication gives the cost increase as 5 percent. A second source of data computes cost figures from the design of a six-story and an 11-story reinforced concrete apartment building for UBC Zones 0, 1, 2, 3, and S (Zone S requirements are twice UBC Zone 3 requirements).²³

A linear extrapolation of these data for a critical facility for 1.0 g response spectrum acceleration using the SEAOC and MIT data yields an average increase in replacement costs of 74.8 and 30.3 percent, respectively. The 74.8 percent increase is probably excessive, particularly in shear wall buildings where the configuration may remain basically the same at low and high levels, but the percentage steel in the shear wall is increased. For this study, the four- and seven-story configurations are based on data for the six-story concrete building in the MIT study, and the ten-story configuration is based on the data for the 11-story concrete building.

For four-story configurations, extrapolation of the MIT data based on a Zone 3 requirement and capacity and demand values at SEAOC Zone 3 and a 1.0 g response spectrum indicated replacement costs to be

²² *Percent Increase in Design Inspection and Construction Cost*, Report of the Ad Hoc Committee on Costs of Design for Earthquakes (SEAOC).

²³ R. V. Whitman, *Methodology and Pilot Application*, R74-15, Report No. 10, Structures Publication 385 (Department of Civil Engineering, MIT, July 1974).

Table B12

Simulation Model Coefficients

	<u>Four Story</u>	<u>Seven Story</u>	<u>Ten Story</u>
c_1	5.00	5.00	5.00
c_2	2.66	3.38	2.18
c_3	24.37	24.37	24.37
c_4	1.00	1.00	1.00
c_5	1.812	1.812	1.78
c_6	0.0317	0.0317	0.0312
c_7	0.00998	0.00349	0.00155
c_8	76.109	26.650	11.801

on the order of 124 percent. A similar consideration of seven- and ten-story facilities provided replacement costs of 136 percent and 131 percent, respectively. All three structural configurations (K equals 0.67, 0.80, and 1.33) were considered in computing these costs. c_7 and c_8 were determined by equating the percentage increase in costs to

$$c_7 \left\{ (b_1 d_1 - b_0 d_0) \left(\frac{CSS}{CSS + CNS} \right) + \frac{c_8}{c_7} (p_1 - p_0) \left(\frac{CSS}{CSS + CNS} \right) \right\}$$

The ratio of c_8 to c_7 was determined by requiring the wall reinforcement steel quantity to be equal to the frame reinforcement steel quantity. This provided realistic percentages of shear wall steel reinforcement. Table B12 presents the values for c_7 and c_8 .

It should also be noted here that structural costs (CSS) are taken to be 82 percent of the total cost increase (TCIN). The data of Figure B4 provide some support for this percentage. Thus the structural cost increase is .82 TCIN and the nonstructural cost increase is .18 TCIN, where

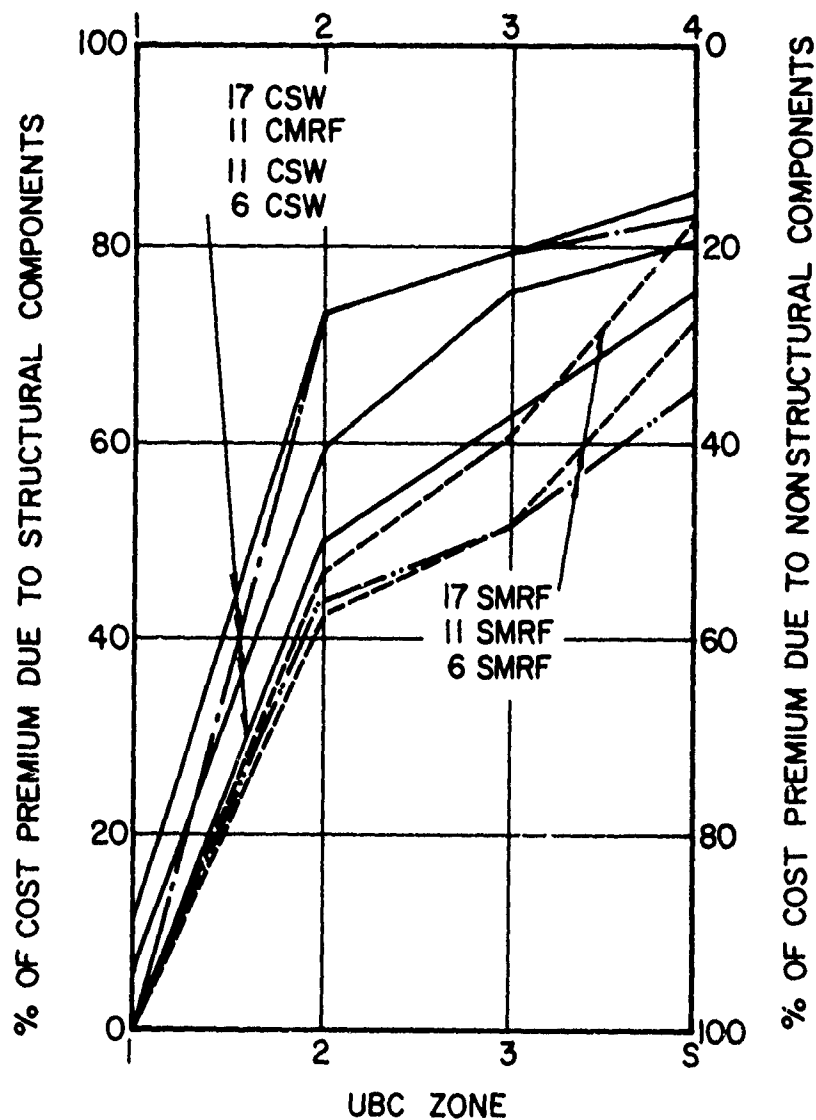
$$TCIN = [c_7(b_1 d_1 - b_0 d_0) + c_8(p_1 - p_0)] / 0.82 \quad [Eq B10]$$

The model is calibrated for replacement data.

A final computation for the costs for a critical facility should be presented. Five case studies of Veterans Administration hospital construction²⁴ give the ratio of structural to nonstructural costs as 25.2 percent. The total nonstructural cost is given by

$$CNS = 0.80 TC = 0.80 (41.03) = \$32.82/\text{sq ft} \\ (\$353.27/\text{m}^2) \quad [Eq B11]$$

²⁴ *Feasibility Study--V.A. Hospital Building System*, Research Study Report, Project Number 99-R003 (Building Systems Development and Stone, Marraccini and Patterson, October 1968).



CSW - Concrete Shear Wall
 CMRF - Concrete Moment Resisting Frame
 SMF, SMRF - Steel Moment Resisting Frame

Figure B4. Relative cost premium for structural and nonstructural components. From R. V. Whitman, *Methodology and Pilot Application*, R74-15, Report No. 10, Structures Publication 385 (Department of Civil Engineering, MIT, July 1974).

and the total structural cost by

$$CS = 0.20 \quad TC = 0.20 (41.03) = \$8.21/\text{sq ft } (\$88.37/\text{m}^2) \quad [\text{Eq B12}]$$

The costs are averaged over the five hospitals.

Summary

The calibration data presented above for the three algorithms can be summarized as follows:

1. Damage/repair data consider only story height breakdown (five to seven stories and eight to thirteen stories) and building material (steel or concrete). Building system and ratios of relative siffnesses are not a part of the classification. For nonstructural damage/repair data, there is no breakdown at all.

2. Strengthening data considers building system and story height; however, the available data are for a K of 1.33 only.

3. Replacement data provide no breakdown according to building configuration. Extrapolation of the data for various story height configurations results in different calibration constants because of the differing ratios of structure demand to capacity.

The model contains more detail than can be provided by the available data. The calibration process is kept simple so that any uniformity in variation of the costs can be identified over the wall to frame stiffness parameter. Observations such as general decrease in strengthening costs with increase of wall to frame stiffness are a product of this effort. As a result of this averaging of the calibration constants over the wall to frame stiffness configuration, the cost information will be averaged over the wall to frame stiffness configuration also. Thus, calibration to 131 percent replacement cost for a ten-story building will show a variation from 138 percent replacement cost for a frame structure down to a 109 percent replacement cost for a light shear wall structure. This variation over structural configuration leaves the model well within the accuracy of the available data.

SYMBOLS

AR_i	- area of the i^{th} floor.
A^S	- area of shear wall.
A_S	- area of column tension steel.
A'_S	- area of column compression steel.
b	- column width.
$c_i (i=1, 8)$	- model calibration constants.
C	- structural system dynamic coefficient.
C'_j, C'_i	- cost coefficients.
C_S	- structure capacity in terms of base shear force.
CNS	- total nonstructural construction costs per square foot.
$COV(x)$	- coefficient of variation of x .
(C/S_a)	- response spectrum structural system dynamic coefficient.
CSS	- total structural construction costs per square foot.
d_i	- column effective depth.
D_r	- story drift design requirement.
D_s	- demand on the structure in terms of yield base shear force.
d_x, d_N	- structure story displacement at level x (at level N).
DF	- total damage factor.

DF_{NS}, DF_{NS_i}	- damage factor for nonstructural damage (for the i^{th} story).
DF_{SNS}	- damage factor for structural and nonstructural damage.
DR_{iI}	- damage ratio (cost of repair/cost of replacement) for the i^{th} structure in a particular category subject to ground motion intensity I.
e	- column eccentricity ratio.
E	- modulus of elasticity.
E, E_i	- an event.
f_c^i	- yield strength of concrete.
f_y	- yield strength of steel.
F^f	- portion of design lateral force acting on the frame.
F_i	- total external lateral load.
F^S	- portion of design lateral force acting on the shear wall.
F_x	- design lateral force at height x.
F, F_i	- structure overturning moment forces.
h	- story height
h_x	- structural height from ground to x.
$H(w)$	- a function of the stiffness of the elements of the structure and the absolute lateral displacements.
I_c	- moment of inertia of structure model exterior column.

I^f	- moment of inertia of structure model frame.
I^s	- moment of inertia of structure model shear wall.
IC	- incremental cost.
k	- an economic factor (Blume damage model).
K	- structural system type coefficient.
K_i	- existing structure stiffness matrix.
\bar{K}_i	- average stiffness of the original simulated design.
K_m, K_p	- interaction diagram column design parameters.
K_s	- strengthened structure stiffness matrix.
K_{ss}	- strengthening parameter stiffness matrix.
$K_{ss}(i,i)$	- i^{th} diagonal term of the strengthening parameter stiffness matrix.
ℓ	- length of dimension.
m_{DRI}	- mean damage ratio at MMI I.
M	- bending moment.
M_i^f	- moment in frame at level i.
M_i^s	- moment in shear wall at level i.
M_u	- ultimate moment capacity of a column.
n_I	- total number of buildings in a particular category subject to ground motion intensity I.
NSC_i	- nonstructural damage/repair costs for the i^{th} level.
p	- percentage of steel in one column face.

p_s	- percent steel in shear wall
p_{t_m}	- column compression and bending design parameter.
p_{DSI}	- percentage of structures in a particular damage state.
p_i^f	- external force on plane at i^{th} level.
p_i^s	- external force on shear wall at i^{th} level.
$P(E)$	- probability of occurrence of event E.
P_u	- ultimate axial force capacity of a column.
R	- column reduction factor.
RF_{SNS}	- replacement factor for structural and nonstructural elements.
S_a	- spectrum acceleration level.
SC_i	- structural damage/repair costs for the i^{th} level.
SF_{NS}	- strengthening factor for nonstructural elements.
SF_S	- strengthening factor for structural elements.
t	- effective column depth.
T	- convergence factor.
TC	- total structure construction cost.
$TCIN$	- total cost increase.
U	- structure capacity.
V	- structure design base shear.
V_b	- structure shear at base.
v^f	portion of shear in frame.
v^s	- portion of shear in shear wall.

V_u	- ultimate column shear resistance.
V_y	- structure yield base shear.
w_i	- story displacement at the i^{th} level.
w_1, w_2	- weighting factors for determination of DF
W, W_i	- total weight of structure including dead load and live load (weight of i^{th} story).
Z	- seismic zone factor.
Δ, Δ_i	- relative story displacement (for the i^{th} story).
Δ	- relative measure of structure displacement before collapse (single degree of freedom model).
Δ_e	- relative measure of structure displacement at elastic limit (single degree of freedom model).
δ	- relative story displacement in feet.
δ_i	- dimension factor for overturning moment.
ϵ_u	- ultimate story strain.
μ	- structure ductility.
μ_u	- ultimate structure ductility.
σ_x	- standard deviation of x .
ϕ_s, ϕ_m	- capacity reduction factors.

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