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# ASSESSMENT OF DAMAGEABILITY FOR EXISTING BUILDINGS IN A NATURAL HAZARDS ENVIRONMENT

## VOLUME I: METHODOLOGY

TECHNICAL REPORT NO. 80-1332-1 • PREPARED FOR THE NATIONAL SCIENCE FOUNDATION, WASHINGTON, D.C.

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# ASSESSMENT OF DAMAGEABILITY FOR EXISTING BUILDINGS IN A NATURAL HAZARDS ENVIRONMENT

## VOLUME I: METHODOLOGY

Technical Report No. 80-1332-1

Prepared for

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September 1980

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## SUMMARY

This report documents a research study undertaken to develop a computer tool for local building and safety officials so that they may independently assess the damageability, and hence the potential safety, of individual buildings exposed to earthquake, severe wind, and tornado forces. The computer program, named DAMAGE, offers a number of features which have been designed with the needs of the building official in mind. They include:

- An integrated program which combines the calculation of environmental forces, with dynamic response analysis and damage modeling to evaluate both the structural and nonstructural damage potential of existing as well as newly designed buildings on a story by story basis.

- An interpretive format keyed to interstory drift as (1) an index related to earthquake and wind intensity, (2) a measure of static and dynamic response for individual buildings, and (3) an index related to reported damage experience.

- Documentation containing user-oriented instructions and guidelines with examples developed during actual user testing by a local building official having no prior involvement in the development or use of the program.

Volume I of the report describes the development of interstory drift as a basis for measuring building performance in earthquake and wind environments. It also describes the nature of the computer program DAMAGE, including input requirements, computational options and output quantities. One full section of Volume I is devoted to user testing, including the planning and conduct of the test as well as the incorpora-

tion of user feedback in the program and documentation. Numerical results from the test cases are appended to this volume.

Volume II consists of the Users Manual for DAMAGE. This manual contains step-by-step instructions for the preparation of input necessary to execute the program. Examples of input coding sheets, card sequence and an operational flow diagram are included.

The goals of this effort have been two-fold: First, to provide a computer tool which non computer-oriented personnel in local building and safety departments can learn to use. Second, and of equal importance, has been the goal to relate the input, methods, and output of the program to the professional experience and occupational needs of the intended user.

# 1. INTRODUCTION

## 1.1 Background and Purpose

Reference [1]* documents the development of a methodology and computer program for estimating the damage by floor for individual buildings subjected to earthquake and wind loads. The original computer program provides a very flexible tool for the analysis of these natural forces and their effect on specific buildings, under specific site conditions. Multiple options for modeling the earthquake and wind environment, site modifications, structural configurations and types of damage are offered by the program. Understandably, this flexibility has proved to be somewhat of a deterrent to practical use, because of the large amount of input required to exercise the various options. This has been especially true for local building and safety departments where the needs tend to be more utilitarian. The present report documents a research study to investigate the tradeoffs between modeling detail and flexibility on one hand, and utility on the other, as they affect the primary output of the program which is structural response in an earthquake or wind environment and corresponding measures of damageability. Damageability is herein considered to be the potential of a building to suffer damage from the natural hazards (or forces) under consideration.

There is a subtle but important distinction implied here between "assessed damageability" and what might be termed "predicted damage," even though the terms are used interchangably in much of the current literature. The distinction is important from the standpoint of conveying the proper meaning of the term damageability as it is used in this report, and thereby avoiding confusion which might otherwise result. The following working definitions have been adopted:

*Numbers shown in square brackets correspond to references listed at the end of this report.

- Assessed damagebility is defined as a relative measure of building performance, under earthquake or wind loading of a given intensity or severity, depending on the design characteristics of the building.

- Predicted damage is defined as an absolute measure of physical damage to a particular building resulting from a particular earthquake or wind event.

The difference between these two measures of damage is similar to the difference between earthquake hazard assessment and earthquake prediction. The latter is specific in terms of time, location and magnitude, whereas the former tends to reflect statistical averages and is therefore more relativistic in nature. It seems appropriate to use statistical averages either as a measure of aggregate damage or loss for a region during a period of time, or for assessing the damageability of a building to a generic event of a given intensity.

Clearly, this rationalization may be carried too far. For example, a building and safety official who must decide whether or not to condemn an existing building or approve the design of a new one, cannot base his decision on information which is nonspecific to the building. Furthermore, such information gives no indication of what alternatives for retrofit or redesign might be considered to make the building more resistant to hazardous loading conditions. While it obviously would be desirable to be able to predict the damage to a particular building as the result of a future earthquake, in general not enough is known either of the building or "the future earthquake" to make this possible.

We are thus faced with the dual challenge of not only facilitating meaningful assessments of a particular building's damageability, but at the same time having the methodology be sufficiently simple and transparent for general use. The specific goals of the project are to pro-

vide a computer tool which local building and safety officials without computer experience can learn to use, and to relate the required input, methods, and output of the program to the experience and needs of the intended user.

## 1.2 Objectives and Scope

Having established the purpose and goals of the project, it is important to delineate the specific objectives which have guided the technical approach, and to define the scope of the work so that its uses and limitations may be properly understood.

The primary objectives were:

(1) To review the current literature for methods and/or suggestions of methods which might be used to model damageability.

(2) To develop such models suitable for interfacing with an existing hazard loading and structural response analysis computer program.

(3) To develop an integrated computer program which includes modeling capabilities for (a) earthquake, wind and tornado-induced lateral building loads, (b) structural static and dynamic response analysis, and (c) structural and nonstructural damageability assessment.

(4) To provide adequate documentation for the intended user (local building and safety engineer) to understand the program and use it effectively, and

(5) To have the program and its documentation user-tested on the basis of realistic applications.

The first two objectives are detailed more specifically in terms of guidelines for the development of damageability models. It was felt that the models should satisfy the following criteria:

- They should have a theoretical basis which enables the "damages" computed for a particular building to be traced back to the characteristics of the structure and load conditions so that (a) the results may be evaluated on the basis of the user's knowledge and experience, and (b) remedial alternatives may be inferred.

- The models should be grounded in and relate directly to actual damage experience.

- The models should be formulated as simply as possible, in terms of parameters that are familiar to the user and have a high degree of correspondence with observable quantities for which data are available.

These guidelines were followed in an effort to achieve the degree of reliability and transparency sought in the models.

The scope of the effort was necessarily limited to ensure its integrity. The following comments are offered to help the reader grasp the intended significance of the work.

All of the models and the analysis are two-dimensional. This includes earthquake and wind loading models, structural models, and damageability models to the extent that damages, which of course reflect 3D behavior,

are related to 2D response variables. No attempt has been made to correlate damage experience with 3D effects.

The damageability models, expressed in terms of a damage ratio vs. interstory drift, are considered to be primarily representative of high-rise buildings, i.e., five stories and above. While nothing prevents the methodology from being applied to low-rise buildings, it is felt that a response index other than average interstory drift, e.g., total base shear, may be more appropriate for low-rise buildings in the sense that it appears to be more readily observable in strong motion records for short-period buildings, to the extent that measured floor accelerations can be used to estimate base shear. This depends significantly on the nature of building response, where for short period buildings the floor accelerations tend to be dominated more by the fundamental mode.

While the basic form of the models and the apparent nature of the analysis is "deterministic," i.e., the model parameters as well as input and output variables are treated deterministically, the underlying data, theory, and analysis techniques are probabilistic and statistical, so that a valid probabilistic interpretation of the results can be made. In short, simplifying assumptions have been made in considering key parameters to have lognormal distributions so that median values may be treated deterministically for convenience and simplicity, while maintaining a realistic perspective on uncertainty through coefficients of variation which are statistically derived from the data. The approach here has been to associate all of the uncertainty in the loads (given a load intensity), structural models and response, and the damage models themselves with the damage models. The degree of scatter or coefficient of variation is quite large - typically a factor of three. However, from the standpoint of quantifying uncertainty, considering the difficulty of characterizing damage by a single parameter such as damage

ratio, this is probably as accurate as one can be. The trends evidenced by central tendencies (median values) are believed to be meaningful, large uncertainties not withstanding.

# 2. MODELING DAMAGEABILITY

## 2.1 Discussion of Current Methods

Various methods of estimating building damage due to earthquakes have been presented in the literature. Generally, these methods fall into one of three groups: empirical, theoretical and subjective.

### 2.1.1 Empirical Methods

Empirical methods are based entirely on statistical observations of building damage from past earthquakes. Normally, these relationships relate percent damage to some measure of earthquake ground motion, such as Modified Mercalli Intensity, MMI, for different categories of structures. It must be noted, however, that these empirical relationships are only valid when attempting to predict how an "average" structure will respond to a given level of ground motion. They normally do not consider such factors as load-deformation characteristics or energy dissipation which are often evaluated on a building specific level.

Specific advantages are obvious, however, in an empirical approach. Very often these relationships are based on large data samples. By using a non-instrumentally recorded earthquake intensity index such as MMI, one can conceivably assign an earthquake intensity measure to every building damaged during an earthquake. This leads to a substantial data base; one much larger than if only buildings with triggered accelerographs are used. A second advantage which results from using an empirical approach is that the relationships are generally easier to apply. In most cases, all that is required to evaluate the damageability of a building due to earthquake is: (1) an estimate of the expected level of earthquake ground motion; (2) some knowledge of the local soil condition; and (3) if different damage relationships are used for different types of structures, knowledge of structure type. The majority of the relationships which have been developed often do not consider (2).

Empirical relationships have been used primarily in macro economic and insurance related studies. Normally, in these types of studies, the aggregate effects of earthquake losses from a large population of structures are evaluated. This often entails a large number of computations. The empirical approach, both from the standpoint of accuracy and complexity, is well suited for these types of analyses. Some notable studies which have incorporated these types of relationships include: Seed, et al., 1970; J.H. Wiggins Company, 1975; Whitman, et al., 1977; Hafer and Kintzer, 1977; Earthquake Engineering Systems, 19 ; Wiggins, et al., 1978; and Sauter, 1979. Information each on each relationship is provided in Table 2-1.

### 2.1.2 Theoretical Methods

Theoretical methods for building damage assessment are typically based on detailed structural models. Structural models incorporating beam and column elements, diaphragms and shear walls are used to idealize the building mathematically. Seismic input loads, often expressed in terms of peak accelerations or peak velocities, are then calculated and imposed on the structural model. Responses in terms of stresses and deformations are calculated which are then related to different measures of damage. The theoretical approach assumes that one can relate damage, for a specific structure, to key response parameters. This approach is quite different from the one presented in Section 2.1.1 which allowed only a generic assessment of damage.

The theoretical approach has several advantages. First, it allows one to relate damage to factors which actually influence the response of the structure, such as the rigidity of the system or the energy dissipation which may occur during an earthquake. Secondly, because the structure is made up of a number of subsystems, one has the flexibility to perform a sensitivity analysis to evaluate the actual contributions of each subsystem to the overall reliability of the structure. This allows one

Table 2-1. Empirical Earthquake Damage Relationships for Buildings

| DATA SOURCE | UNITS OF DAMAGE | INDEPENDENT VARIABLE(S) | BUILDING CLASSIFICATIONS | REFERENCE | COMMENTS |
|---|---|---|---|---|---|
| OBSERVATIONS OF BUILDING DAMAGE DURING THE 1967 CARACUS EARTHQUAKE | NUMBER OF BUILDINGS SUFFERING STRUCTURAL DAMAGE DIVIDED BY THE TOTAL NUMBER OF BUILDINGS | DEPTH TO ROCK AND STORY HEIGHT | MULTI-STORY BUILDINGS WITH THE FOLLOWING STORY HEIGHT RANGES<br>• N<14<br>• 10<N<14<br>• 5<N<9<br>WHERE N IS THE NUMBER OF STORIES | SEED, ET AL., (1970) [2]* | THE REPORT DEMONSTRATED THAT DAMAGE PATTERNS DURING THAT EARTHQUAKE WERE CLEARLY RELATED TO SOIL CONDITIONS AND IN EFFECT WERE ATTRIBUTED TO THE "TUNING" OF THE SOIL AND THE BUILDING. |
| DAMAGE ESTIMATES FROM THE 1971 SAN FERNANDO EARTHQUAKE MODIFIED BY EXPERT OPINION | REPAIR COST DIVIDED BY MARKET COST | MODIFIED MERCALLI INTENSITY | • COMMERCIAL - INDUSTRIAL<br>• RESIDENTIAL<br>FOUR DIFFERENT RELATIVE STRENGTHS OF CONSTRUCTION OR QUALITY FACTORS WERE PRESENTED FOR EACH CLASSIFICATION | WIGGINS, ET AL., (1974)<br>J.H. WIGGINS COMPANY (1975)<br>WIGGINS, ET AL., (1978)<br>EGUCHI AND WIGGINS (1979)<br>[3, 4, 5, 6] | THE RELATIONSHIPS ARE BI-LINEAR ON LOG-LINEAR PAPER, WITH THE STEEPER SLOPE OCCURRING AT THE LOWER MMI LEVELS. |
| DAMAGE ESTIMATES FROM THE 1971 SAN FERNANDO EARTHQUAKE | REPAIR COST DIVIDED BY REPLACEMENT COST | VARIOUS STUDIES WERE CONDUCTED USING THE FOLLOWING AS DEPENDENT VARIABLES:<br>• AVERAGE PEAK ACCELERATION<br>• AVERAGE PEAK VELOCITY<br>• HOUSNER INTENSITY<br>• ARIAS INTENSITY<br>IN ADDITION, CORRELATIONS BETWEEN DAMAGE AND ELASTIC BASE SHEAR RATIO AND MEAN STRUCTURAL RESPONSE WERE EVALUATED | • CONCRETE<br>• STEEL<br>• BRICK MASONRY<br>THE ABOVE GROUPS WERE FURTHER BROKEN DOWN INTO AGE GROUPS (I.E., PRE-1933 AND POST-1947) | WHITMAN, ET AL., (1974) [7] | OUT OF THE FOUR GROUND MOTION PARAMETERS DESCRIBED, CORRELATIONS USING PEAK VELOCITY AND HOUSNER INTENSITY WERE FOUND TO BE THE BEST. CORRELATIONS USING THE RESPONSE MEASURES WERE FOUND TO BE EQUALLY AS GOOD. |

*References numbered in square brackets are listed at the end of this report.

Table 2-1. Empirical Earthquake Damage Relationships for Buildings (continued)

| DATA SOURCE | UNITS OF DAMAGE | INDEPENDENT VARIABLE(S) | BUILDING CLASSIFICATIONS | REFERENCE | COMMENTS |
|---|---|---|---|---|---|
| DAMAGE ESTIMATES FROM THE 1971 SAN FERNANDO EARTHQUAKE | REPAIR COST DIVIDED BY REPLACEMENT COST | ENGINEERING INTENSITY (EI) WHICH IS DEVELOPED FROM A 5% DAMPED SPECTRA AND FOR VARIOUS PERIOD BANDS | • LOW-RISE STRUCTURES<br>• HIGH-RISE STRUCTURES | HAFEN AND KINTZER (1977) [8] | IT WAS FOUND THAT THE CORRELATION BETWEEN GROUND MOTIONS AND DAMAGE FOR LOW-RISE BUILDINGS WAS THE HIGHEST WHEN THE GROUND MOTION PARAMETER WAS THE FIRST DIGIT OF THE THREE-DIGIT EI. FOR HIGH-RISE BUILDINGS, THE CORRELATION WAS HIGHEST WHEN THE GROUND MOTION PARAMETER WAS THE SECOND DIGIT OF THE THREE-DIGIT EI. |
| 40 BUILDINGS IN LOS ANGELES WHICH EXPERIENCED DAMAGE DURING THE 1971 SAN FERNANDO EARTHQUAKE AND ALSO PRODUCED STRONG MOTION RECORDS | REPAIR COST DIVIDED BY REPLACEMENT COST | VARIOUS CORRELATIONS WERE DEVELOPED USING THE FOLLOWING DEPENDENT VARIABLES:<br>• SPECTRAL ACCELERATION<br>• SPECTRAL VELOCITY<br>• SPECTRAL DISPLACEMENT<br>• CALCULATED INTER-STORY DISPLACEMENT | • CONCRETE<br>• STEEL<br>ALL BUILDINGS GREATER THAN OR EQUAL TO FIVE STORIES | WONG, (1975) [9] | CORRELATIONS USING SPECTRAL VELOCITY AND SPECTRAL ACCELERATION APPEAR TO BE THE BEST DAMAGE INDICATORS. INTER-STORY DISPLACEMENT ALSO PROVIDED ENCOURAGING RESULTS, ALTHOUGH IT WAS NOT CONSIDERED IN THE DEVELOPMENT OF THE FINAL MOTION-DAMAGE RELATIONSHIPS. |
| BUILDING DAMAGE DATA FROM FIVE CALIFORNIA EARTHQUAKES (NOT INCLUDING THE 1971 SAN FERNANDO EARTHQUAKE) AND FROM DATA RECORDED IN SIX INTERNATIONAL COUNTRIES | REPAIR COST DIVIDED BY REPLACEMENT COST | MODIFIED MERCALLI INTENSITY | • MASONRY<br>FOUR CLASSES OF MASONRY WERE DEFINED, EACH REFLECTING A DIFFERENT QUALITY LEVEL | EARTHQUAKE ENGINEERING SYSTEMS, (1978) [10] | A POSTULATED DISTRIBUTION OF BUILDING DAMAGE BY COMPONENT IS GIVEN:<br>STRUCTURAL DAMAGE 40%<br>ARCHITECTURAL DAMAGE 50%<br>MECHANICAL DAMAGE 7%<br>ELECTRICAL DAMAGE 3% |
| EARTHQUAKE DAMAGE DATA RECORDED FROM THE 1971 SAN FERNANDO, 1933 COMPTON, AND 1952 KERN COUNTY EARTHQUAKES | REPAIR COST DIVIDED BY REPLACEMENT COST | MODIFIED MERCALLI INTENSITY | • 1 AND 2 STORY BUILDINGS<br>• LIGHT INDUSTRIAL | BLUME, ET AL., (1975) [11] | RESULTS PRESENTED IN TERMS OF BOTH MEAN AND MEDIAN VALUES |

Table 2-1. Empirical Earthquake Damage Relationships for Buildings (continued)

| DATA SOURCE | UNITS OF DAMAGE | INDEPENDENT VARIABLE(S) | BUILDING CLASSIFICATIONS | REFERENCE | COMMENTS |
|---|---|---|---|---|---|
| | REPAIR COSTS DIVIDED BY REPLACEMENT COST | MODIFIED MERCALLI INTENSITY | • ADOBE<br>• UNREINFORCED MASONRY - LOW QUALITY<br>• REINFORCED CONCRETE FRAMES - WITH AND WITHOUT SEISMIC DESIGN<br>• STEEL FRAMES - WITH AND WITHOUT SEISMIC DESIGN<br>• REINFORCED MASONRY - MEDIUM QUALITY - WITHOUT SEISMIC DESIGN<br>• REINFORCED MASONRY - HIGH QUALITY - WITH SEISMIC DESIGN<br>• SHEAR WALLS - WITH SEISMIC DESIGN<br>• WOODEN FRAME DWELLINGS | SAUTER, (1979) [12] | |

to isolate vulnerable parts of the structure, which is a necessary step in any damage reduction plan. Finally, these theoretical approaches normally make use of quantitative seismic input parameters, such as peak acceleration and peak velocity, to represent the seismic environment. These parameters are better indicators of earthquake ground motion then that afforded by the qualitative Modified Mercalli Intensity scale, which is often used in empirical approaches.

The damage functions which are used in these theoretical methods are, in most cases, analytical expressions relating percent damage to some structural response parameter, such as interstory drift. The basic assumption in these methods is that failure or some level of damage results when a computed "load", such as interstory drift, exceeds some specified load, which is often part of a "resistance" curve. The verification of this assumption is often left to an analytical solution, which is sometimes modified by subjective judgment. The development of these damage functions represents the most important step in the theoretical approach. The functions themselves should produce believable results, and should incorporate whatever historical observation data are available. Some notable studies which reflect primarily on the theoretical approach include: Blume et al., 1975; Culver et al., 1975; Czarnecki, 1978. Information on each approach is provided in Table 2-2.

### 2.1.3 Subjective Approach

When there is a lack of quantitative knowledge concerning the relationship between building damage and some measure of earthquake intensity, or if an analytical solution to the problem is beyond the present state-of-the-art, one has to rely on qualitative or subjective methods of predicting damage. Personal experience in observing earthquake damage is invaluable. Recently, an approach was developed by Whitman at MIT which organized the opinions of a number of earthquake engineering experts regarding the issue of expected damage to buildings during

Table 2-2. Theoretical Methods for Earthquake Damage Assessment

| UNITS OF DAMAGE | INPUT PARAMETERS | BUILDING CLASSIFICATIONS | PROCEDURE | REFERENCE | COMMENTS |
|---|---|---|---|---|---|
| REPAIR COST DIVIDED BY REPLACEMENT COST | • MEDIAN PSEUDO VELOCITY RESPONSE SPECTRUM<br>• GEOMETRIC STANDARD DEVIATION<br>• BUILDING PERIOD<br>• FIRST YIELD CAPACITY IN TERMS OF A SPECTRAL VALUE<br>• ULTIMATE DUCTILITY<br>• LOWER SPECTRAL BOUND ASSOCIATED WITH NO DAMAGE<br>• SLOPE OF THE IN-ELASTIC REGION OF THE P-Δ CURVE<br>• ECONOMIC SCALE FACTOR<br>• COEFFICIENT OF VARIATION FOR CAPACITY | DEPENDS UPON INPUT PARAMETERS | A MEAN DAMAGE FACTOR CURVE IS GENERATED FOR EACH BUILDING TYPE. THE CURVE RELATES DAMAGE TO A NORMALIZED RATIO OF MEDIAN DEMAND OVER MEAN CAPACITY FOR SPECIFIED VALUES OF THE OTHER CAPACITY PARAMETERS. THE DEMAND/CAPACITY RELATIONSHIP IS BASED ON EQUATING THE ENERGY ABSORBED BY THE ELASTIC CAPACITY MODEL WITH AN ASSUMED EQUIVALENT ELASTIC MODEL. AN EXPRESSION FOR DUCTILITY IS DERIVED WHICH IS USED TO COMPUTE THE DAMAGE FACTOR. | BLUME, ET AL., (1975) [11]* | THIS METHOD IS REFERRED TO AS THE SPECTRAL MATRIX METHOD (SMM). THE SMM ALSO MAKES USE OF PROBABILISTIC FORMULATIONS FOR INDIVIDUAL BUILDING DEMAND AND CAPACITY. |
| REPAIR COST DIVIDED BY REPLACEMENT COST | • CONSTRUCTION TYPE<br>• AGE<br>• NUMBER OF STORIES<br>• BUILDING USE<br>• MEDIAN PSUEDO-RESPONSE SPECTRA VALUES | • STEEL OR REINFORCED CONCRETE; FIRE-RESISTIVE; NO HEIGHT LIMITATIONS<br>• STEEL OR REINFORCED CONCRETE; LESS FIRE-RESISTIVE; HEIGHT LIMITATION<br>• MASONRY OR CONCRETE WALLS; WOOD FLOORS AND ROOF<br>• LIGHT STEEL, INCOMBUSTIBLE (SERVICE STATIONS)<br>• WOOD FRAME, LIMITED AREA AND HEIGHT | THREE DAMAGE FACTORS ARE DETERMINED FOR EACH BUILDING: THE FIRST IS AN ELASTIC FACTOR WHICH IS BASED ON THE UNDAMAGED PERIOD OF THE BUILDING; THE SECOND FACTOR IS BASED ON A LENGTHENED PERIOD DUE TO YIELDING AND REPRESENTS AN IN-ELASTIC OR ULTIMATE CAPACITY; AND THE THIRD FACTOR IS CALCULATED FROM A WEIGHTED COMBINATION OF THE INDIVIDUAL BUILDING DAMAGE FACTORS. DAMAGES ARE THEN COMPUTED FOR THE INITIAL AND FINAL STATES BY INPUTING THE CORRESPONDING SPECTRAL ACCELERATIONS AT 5% DAMPING. | BLUME, ET AL., (1975) [11] | THIS METHOD IS REFERRED TO AS THE SEISMIC ELEMENT METHOD (SEM). THE SEM IS AN ATTEMPT TO RECONCILE THEORY WITH HISTORICAL STRUCTURAL PERFORMANCE. |

*References numbered in square brackets are listed at the end of this report.

Table 2-2. Theoretical Methods for Earthquake Damage Assessment (continued)

| UNITS OF DAMAGE | INPUT PARAMETERS | BUILDING CLASSIFICATIONS | PROCEDURE | REFERENCE | COMMENTS |
|---|---|---|---|---|---|
| REPAIR COST DIVIDED BY REPLACEMENT COST | INPUT IS PREPARED FOR FOUR DATA GROUPS:<br>1. LOADS<br>2. STRUCTURAL DATA<br>3. RESPONSE DATA<br>4. DAMAGE DATA<br>AMOUNT OF INPUT FOR EACH GROUP VARIES DEPENDING ON WHICH STRUCTURAL MODEL IS CONSTRUCTED, I.E., DETAILED FRAME MODEL, STORY STIFFNESS MODEL, EMPIRICAL MODEL, AND ALSO WHICH HAZARDS ARE SELECTED, I.E.,EARTHQUAKE, WIND AND TORNADO. | DEPENDS UPON WHICH STRUCTURAL MODEL IS SELECTED AND WHICH HAZARDS ARE ANALYZED. | INTERSTORY DRIFTS ARE COMPUTED FROM ONE OF THREE STRUCTURAL MODELS: (1) A DETAILED FRAME MODEL WHICH GENERATES A STIFFNESS MATRIX AND COMPUTES MODAL DEFLECTIONS FROM A DETAILED MULTI-DEGREE OF FREEDOM STRUCTURAL MODEL INCORPORATING BEAM AND COLUMN FRAMING ELEMENTS, RIGID DIAPHRAGMS AND CONCRETE OR MASONRY SHEAR WALLS: (2) A STORY STIFFNESS MODEL WHICH GENERATES THE SAME INFORMATION AS IN (1) BUT FROM USER INPUT STORY STIFFNESS DATA; AND (3) AN EMPIRICAL MODEL WHICH COMPUTES MODAL DEFLECTIONS FROM A LINEAR MODE SHAPE MODEL AND A USER INPUT FUNDAMENTAL PERIOD. A DAMAGE FUNCTION WHICH RELATES PERCENT DAMAGE TO INTERSTORY DRIFT IS THEN USED TO COMPUTE DAMAGE. | CULVER, ET AL., (1975) [1] | THIS METHOD REQUIRES THE INPUT OF A LARGE NUMBER OF PARAMETERS, RANGING FROM THOSE NEEDED TO REPRESENT THE HAZARD TO THOSE NEEDED TO IDENTIFY THE QUALITY OF THE SUBJECT BUILDING. GUIDELINES FOR EVALUATING BUILDING QUALITY ARE PROVIDED. THE COMPUTER PROGRAM OFFERS A WIDE RANGE OF OPTIONS, INCLUDING FOUR DIFFERENT METHODS OF INCORPORATING SOIL EFFECTS. THE PROGRAM INCORPORATES THE EFFECTS OF EXPERT JUDGMENT BY PROVIDING GUIDELINES FOR SELECTING IMPORTANT DAMAGE PARAMETERS. |
| REPAIR COST DIVIDED BY REPLACEMENT COST | DETAILED STRUCTURAL MEMBER PROPERTIES AND MATERIAL PROPERTIES. | • STEEL MOMENT - RESISTING FRAME<br>• CONCRETE MOMENT - RESISTING FRAME<br>• STEEL BRACED FRAME<br>• CONCRETE SHEAR WALL | STRUCTURAL RESPONSE IS COMPUTED FROM A NONLINEAR DYNAMIC ANALYSIS. STATIC STRESS-STRAIN RELATIONSHIPS ARE USED IN COMPUTING RATIOS OF INELASTIC STRAIN ENERGY (BASED ON COMPUTED MAXIMUM INTERSTORY DISPLACEMENTS) TO TOTAL AVAILABLE INELASTIC STRAIN ENERGY, ON A MEMBER-BY-MEMBER BASIS. THESE RATIOS ARE AVERAGED OVER THE ENTIRE BUILDING TO OBTAIN AN AVERAGE INELASTIC STRAIN ENERGY RATIO WHICH IS EQUATED TO THE DAMAGE RATIO, REPAIR COST DIVIDED BY REPLACEMENT COST. | CZARNECKI (1973) [13] | THIS METHOD REQUIRES MUCH DETAILED ANALYSIS WHICH IS DIFFICULT TO JUSTIFY IN A PRACTICAL SENSE. DAMAGE MODELS DEPEND ON IDEALIZATIONS WHICH MAY NOT ALWAYS APPLY, E.G., STATIC STRESS-STRAIN RELATIONSHIPS. |

earthquakes. Damage probability matrices were developed which presented probability density functions of earthquake damage as a function of specific levels of ground motion. The utility of this approach can be seen when one realizes their potential for interpolating and extrapolating limited but actual data bases.

### 2.1.4 Conclusions

It is fair to say that all of the three groups of damage models discussed above overlap to some degree. None is completely separate from the others. For example, subjective judgment plays an important role in theoretical modeling; empirical data are required to calibrate a theoretical model, and of course underlie the judgment used in a subjective approach. So to say that what is needed is a combination of the three really begs the question. Yet, after studying many of the respectable contributions to the current literature, one is still tempted by such a desire. Perhaps it is partly because much of the power of these existing methods resides with the respective authors who have themselves combined theoretical techniques with empirical data using their own experience and judgment. While the most reliable estimates of damageability may very well be made by some of these individuals, others attempting to use their methods may not be successful. For this reason, we were motivated to try again.

Recalling the three criteria posed in Section 1.2 to guide the selection and/or development of damageability models for the specific purpose of this project, the following comments can be made of existing models in general:

- All of the empirical models reviewed above have one or both of two shortcomings: (1) the independent variable which serves as the "input" to the damage model relates to structural excitation rather than response, thereby precluding the use of

a structural model for purposes of tracing the causal nature of damage, and (2) available data often span a very limited range of the independent variable so that significant trends cannot be established.

- The theoretical models reviewed above seem to have one or both of two shortcomings: (1) they are too complex to be transparent without the aid of detailed sensitivity analysis, and (2) they employ too many parameters (with widely varying degrees of correspondence to actual data) to be specified for particular applications without the aid of considerable expertise.

- The subjective approach alone does not meet the basic requirement of objectivity.

Having made these specific observations, we shall move on to try to develop a logical framework in which empirical modeling, theoretical modeling and subjective judgment can be brought together in new ways which ease the mathematical treatment of these inhomogeneous bodies of information, as well as improve the basis for physical understanding.

## 2.2 Interpretation of Damage Data

Physical damage to buildings from earthquake and wind forces is difficult to measure. Depending on how the damage information is to be used, different measures of damage may be appropriate. Economic uses suggest a measure of cost. Engineering uses (such as design improvement) suggest a measure of strength or resistance capacity. Life safety may relate in part to strength, as well as to other measures of damage such as percent of glass broken, architectural items torn loose, etc.

The difficulty of measuring damage only begins with the definition of measures or "yardsticks." The damage corresponding to these measures must be observable in order to be meaningful.

For example, strength is not directly observable, except by destructive testing. Its use as a meaningful measure of building damage (or damageability) is therefore impractical. Conversely, repair cost as a fraction of replacement cost is observable, but does not relate directly to loss of strength. Yet, structural damage as well as nonstructural damage has been measured in terms of cost. The damage ratio, defined as the ratio of repair cost to replacement cost, has been widely accepted as an objective measure of physical damage for both earthquake and wind [7, 11, 14]. The engineering profession has learned to interpret this measure of damage through experience, i.e., by comparing subjective observations of physical damage with actuarial data on repair cost. Documented case studies such as those reported in Reference [15] are essential to this process of association. Considering the diversity of the different types and degrees of damage, all of which are measured in terms of repair cost versus replacement cost, one can easily appreciate the variability inherent in the empirical relationships which form the basis of current damage models. This variability implies a corresponding degree of uncertainty in estimated values of damage or damageability.

Damageability has been defined as a relative measure of damage potential with respect to buildings of different design, subjected to natural forces of prescribed intensity. The need to discriminate among buildings of different design, however, is in conflict with the need to relate damageability to actual damage experience for purposes of interpretation and understanding. Sufficient damage data do not exist to allow for meaningful statistical analysis of damage to buildings within the very narrow categories which would be required to discriminate among buildings of significantly different design.

One way to resolve this conflict is to find a supplementary source of information. The most desirable kind of information would be that which relates directly to existing damage data. Component test data offer one possibility. In considering this alternative, it was recognized that such data express either damage or capacity as a function of applied load, or some measure of structural response such as deformation. When components comprise structural systems, the loads acting on them become internal loads to the system, and are therefore related to the response of the structural system to external forces. At the present time, most damage data for buildings are expressed as a function of the intensity of external loading, rather than a suitable measure of structural response. In the case of earthquake, for example, damage is most often given as a function of Modified Mercalli Intensity, or MMI. In the case of wind, damage is expressed as a function of either wind velocity or wind pressure at a standard height above ground. It has been found, particularly in the case of earthquakes, that observed damage correlates reasonably well with interstory displacement. It is plausible that wind damage resulting from wind-induced motion of a building would also correlate with interstory displacement. On the basis of this reasoning, studies were undertaken to reinterpret existing damage data in terms of interstory displacement.

### 2.2.1 Earthquake Damage Data

Several attempts have been made to correlate earthquake damage with interstory displacement [9, 13, 16]. In some cases, strong motion instruments located at the basement, mid-height, and top levels of modern high-rise buildings provide direct measurements of interstory displacement, through the doubly-integrated accelerograms. Alternatively, interstory displacements can be estimated from simple theoretical models which assume predominant response in the fundamental mode whose period, T, is proportional to the number of stories, N, and whose mode shape, $\phi$, is linear. A uniform mass distribution is also assum-

ed. The equation relating interstory displacement, d, or interstory drift, $\Delta = d/h$ ,to spectral velocity, $S_v$, is found to be

$$\Delta = \frac{d}{h} = \Gamma\left(\frac{T}{2\pi Nh}\right) S_v \tag{2-1}$$

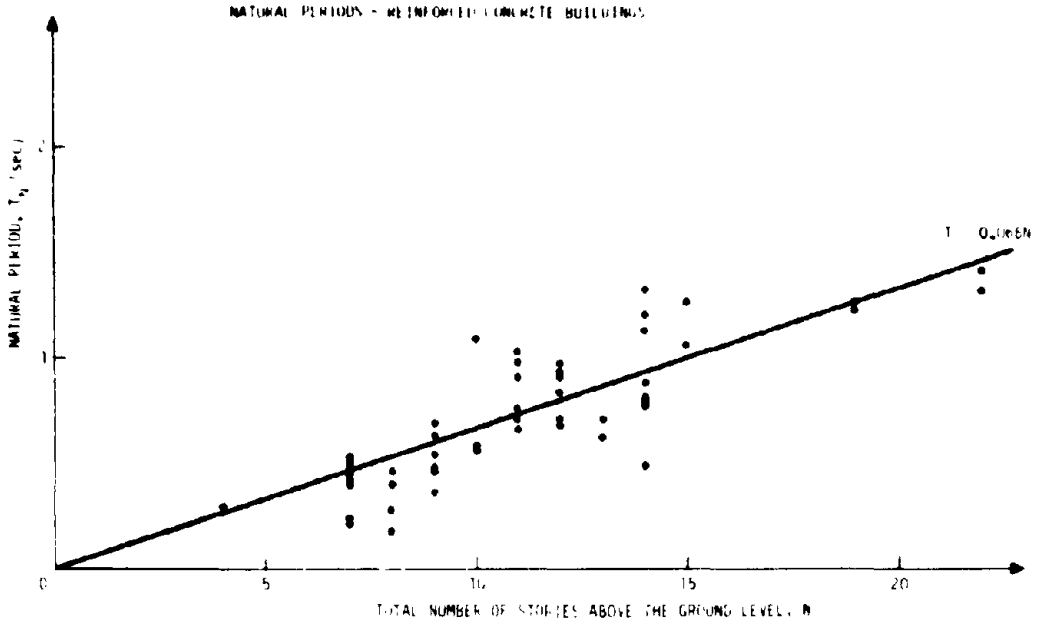
where h is the average story height and $\Gamma$ is the modal participation factor

$$\Gamma = \frac{\sum_{i=1}^{N} \phi_i}{\sum_{i=1}^{N} \phi_i^2} = 1.5 \tag{2-2}$$

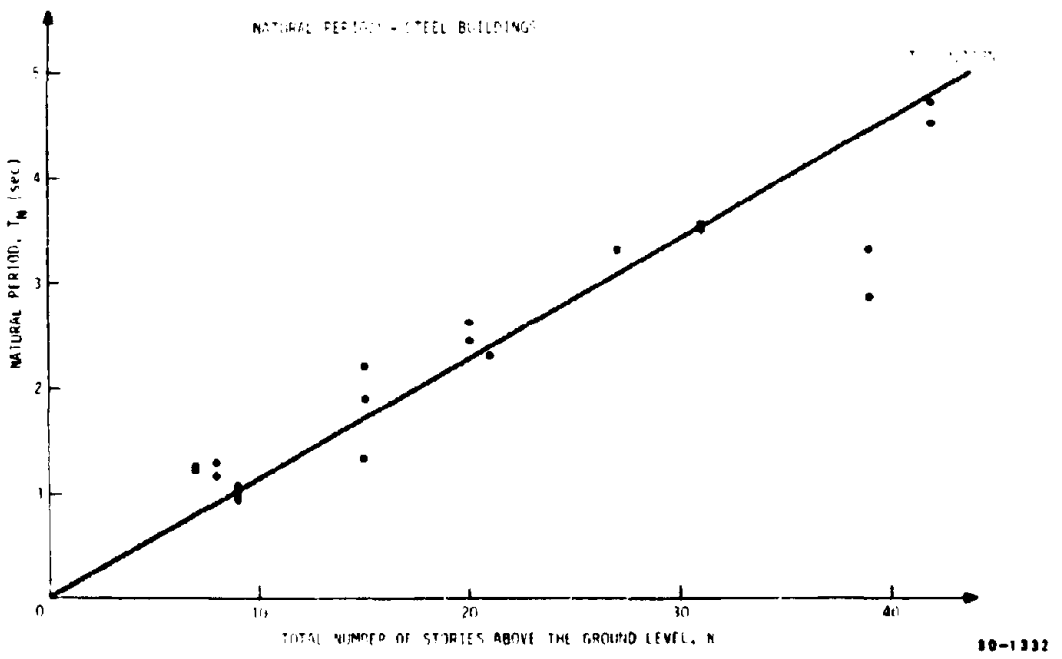
for high-rise buildings. Empirical studies have shown that the ratio T/N tends to be constant, although the constant may vary among different types of buildings. Figures 2-1 and 2-2 show these relationships for steel and reinforced concrete buildings, for pre and during-earthquake conditions, respectively. The appropriate periods to use in the present case are those measured during the earthquake. Values of T/N chosen for this study are T/N = 0.1 for reinforced concrete and T/N = 0.16 for steel buildings.

A correlation study was made using data from the 1971 San Fernando earthquake to assess the validity of Equation (2-1), and to determine empirical values for $\Gamma$ . Figures 2-3 and 2-4 show values of $\Delta/\Gamma$ computed from Equation (2-1), plotted against "measured" values of interstory drift, $\Delta_m$ . Most of these measured values were determined by subtracting the recorded peak displacements at the base of the building, $D_B$, from those measured at the top, $D_T$, and dividing by the height of the building above grade, $H = (L_T-1)h$,*

*In cases where the difference $D_T - D_B$ was small ($\Delta_m < 0.0008$), the actual time-history records were correlated in time to determine $D_T - D_B$.

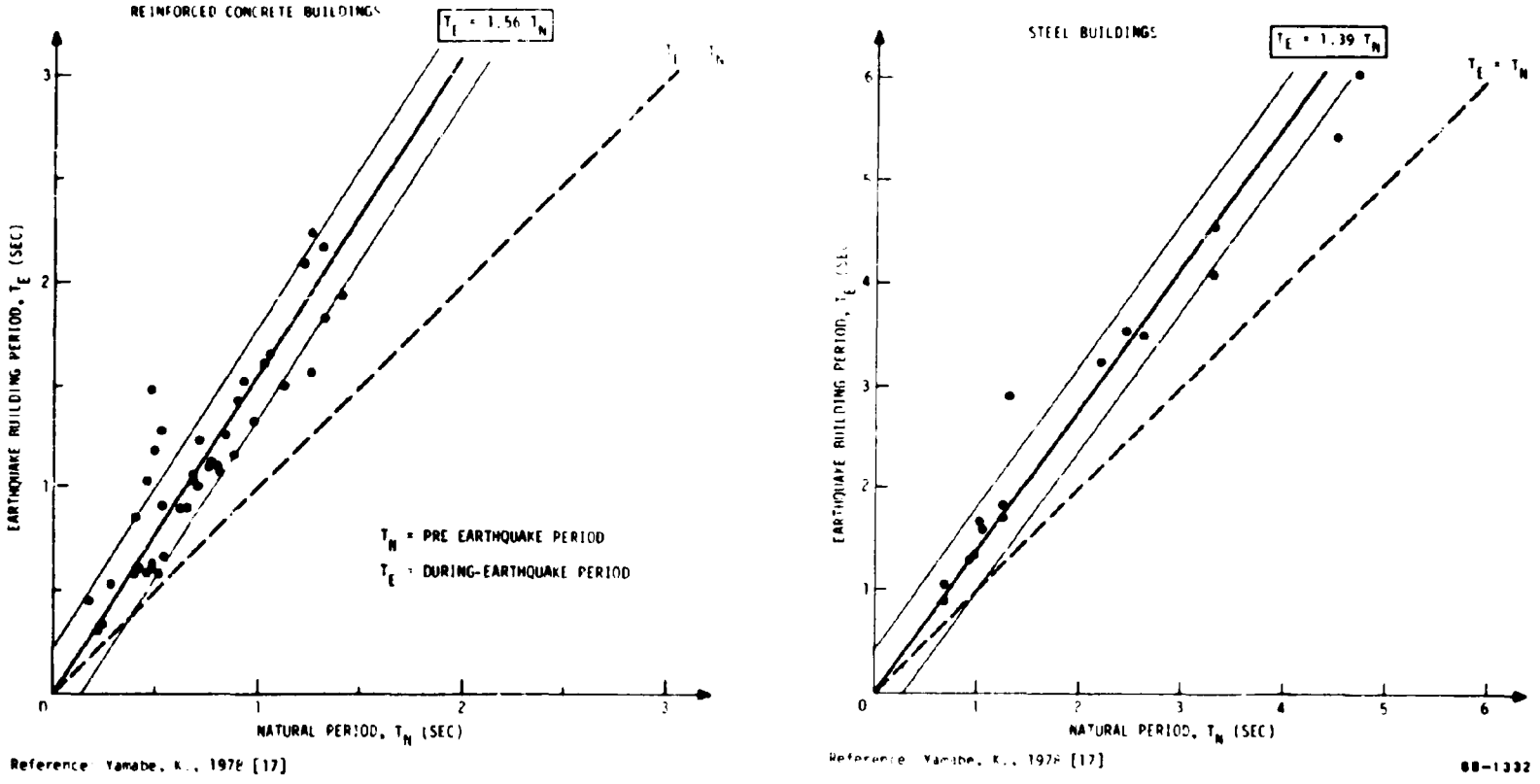



(A) REINFORCED-CONCRETE BUILDINGS




(B) STEEL BUILDINGS

Figure 2-1. Relationship between Natural Period and Total Number of Stories above the Ground Level - Pre Earthquake Condition




(A) REINFORCED-CONCRETE BUILDINGS

(B) STEEL BUILDINGS

Figure 2-2. Relationship Between Building Periods Measured Before and During an Earthquake

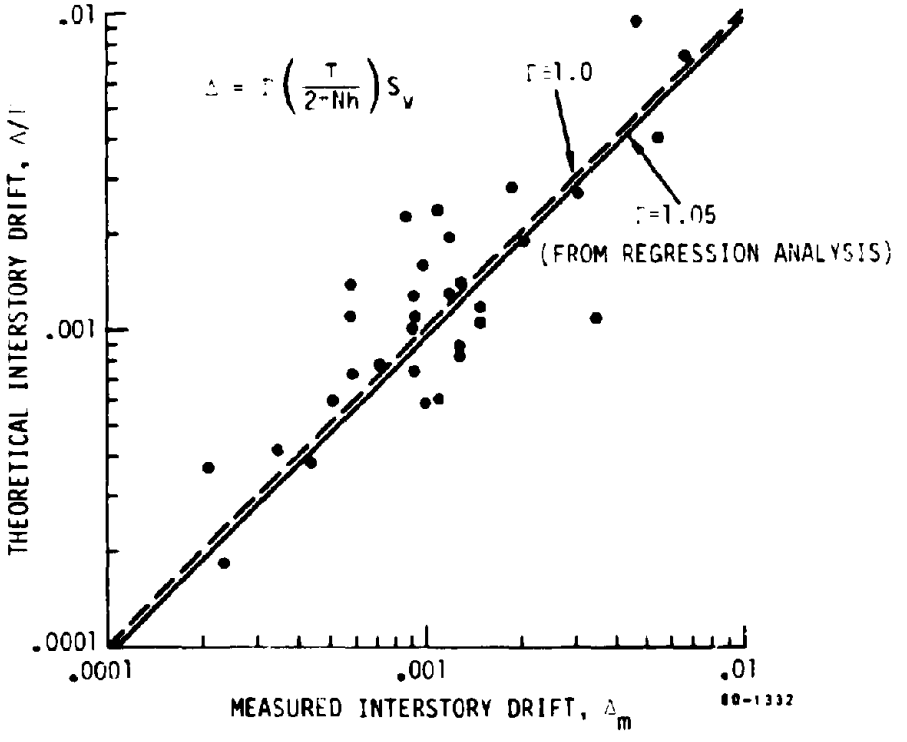



Figure 2-3. Correlation of Predicted and Measured Values of Interstory Drift for Concrete Buildings

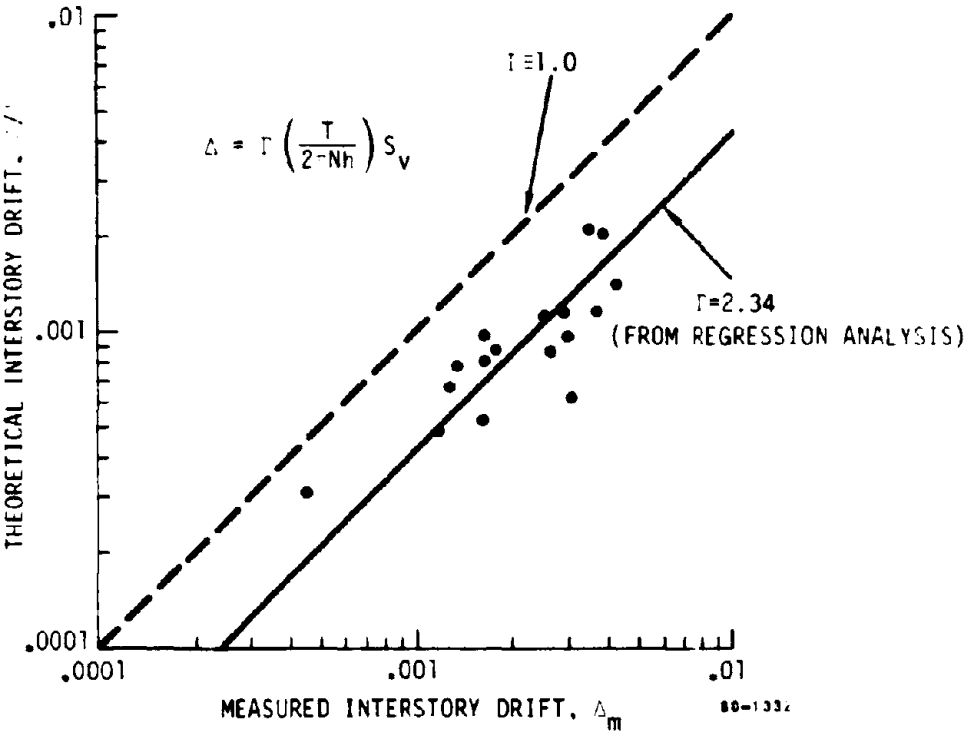



Figure 2-4. Correlation of Predicted and Measured Values of Interstory Drift for Steel Buildings

$$\Delta_m = \frac{D_T - D_B}{H} \qquad (2\text{-}3)$$

where $L_T$ is the number of the "top level" containing the strong motion instrument. Values of $\Gamma = 1.05$ for reinforced concrete and $\Gamma = 2.34$ for steel were determined from this correlation study, based on the data listed in Tables 2-3 and 2-4, respectively.

Earthquake damage statistics from the 1971 San Fernando Earthquake are reported in Referece [18]. Appendix H of [18] contains building values and damage costs for 402 buildings. Of these, 198 are classified as steel structures and 181 as reinforced concrete structures. In the present analysis, each group has been divided into two parts: pre-1933 and post-1933 buildings. The 1933 Long Beach earthquake marked the advent of a major change in building codes to require earthquake resistant design. As pointed out in [18], however, relatively few buildings were built between 1933 and 1947 during the Depression and World War II. The post-1933 category of the present study is therefore very similar, and for practical purposes the same as the post-1947 category in [18].

These data were screened to eliminate buildings of less than 5 stories, and buildings for which the total damage ratio (repair cost/replacement cost) was less than 0.1%. This damage level was taken to be the "threshold of observable damage," below which damage was considered negligible. A total of 118 buildings passed this screening, 59 steel and 59 concrete.

The observable damage threshold of 0.1% was selected with structural damage considerations in mind. According to the descriptive information which defines the MMI scale, structural damage should be negligible to buildings of good design and construction for MMI $\leq$ VII. Reference [18] reports a mean damage ratio of 0.5% for those post-1947 steel and reinforced concrete buildings which were judged to be in the MMI = VII

Table 2-3. Summary of 1971 San Fernando Earthquake Strong Motion Data for Reinforced Concrete Buildings (References [15, 17])

| BUILDING NO. | NO. STORIES N | EARTHQUAKE PERIOD $T_E$ (SEC) | STORY HEIGHT h (IN) | C.I.T. RECORD NO. | DIRECTION | BASE SPECTRAL VELOCITY $S_v$(IN/SEC) | TOP INSTRUMENT LEVEL $L_T$ | MEASURED DRIFT $\Delta_m$ | NORMALIZED THEORETICAL DRIFT $\Delta/\Gamma$ |
|---|---|---|---|---|---|---|---|---|---|
| 2 | 10 | 0.58 | (150)(1) | I128 | N | 6.0 | 11 | 0.00021 | 0.00037 |
| 3 | 10 | 0.93 | (150) | I131 | N40W | 5.5 | 10 | 0.00051 | 0.00060 |
| 4 | 10 | 1.70 | (150) | Q239 | S | 8.0 | 11 | 0.0013 | 0.0014 |
| 5 | 10 | 0.72 | (150) | H124 | W | 4.0 | 10 | 0.00024 | 0.00031 |
| 7 | 14 | 1.83 | 137 | P231 | W | 5.0 | 15 | 0.00075 | 0.00076 |
| 10 | 7 | N/A(2) | (150) | S261 | N31W | 6.0 | N/A | N/A | N/A |
| 12 | 16 | 1.55 | 168 | I134 | N54E | 8.5 | 16 | 0.0013 | 0.00063 |
| 17 | 17 | 1.12 | (150) | R244 | N53W | 14.0 | 20 | 0.0013 | 0.00088 |
| 18 | 17 | 1.00 | (150) | R251 | N37E | 10.0 | 18 | 0.0010 | 0.00059 |
| 22 | 9 | 0.98 | 159 | R253 | N30W | 18.0 | 11 | 0.00099 | 0.0016 |
| 23 | 8 | 1.03 | (150) | K159 | S30W | 12.0 | 6 | 0.00087 | 0.0026 |
| 28 | 11 | 1.61 | 135 | D068 | E | 11.0 | 12 | 0.0021 | 0.0019 |
| 31 | 19 | 2.24 | 116 | L166 | W | 7.0 | 21 | 0.00094 | 0.0011 |
| 32 | 12 | 0.80 | 122 | H118 | S45E | 4.0 | 12 | 0.00044 | 0.00038 |
| 34 | 7 | 1.18 | 113 | D062 | S52W | 17.0 | 8 | 0.0055 | 0.0040 |
| 36 | 16 | 1.75 | 114 | J148 | W | 8.0 | 17 | 0.0015 | 0.0012 |
| 37 | 7 | 0.58 | 179 | F098 | S37W | 10.0 | 7 | INDET.(3) | 0.0010 |

(1) Average story height of 150 inches assumed

(2) N/A - Not available

(3) INDET. - Indeterminate

Table 2-3. Summary of 1971 San Fernando Earthquake Strong Motion Data for Reinforced Concrete Buildings (References [15, 17]) (continued)

| BUILDING NO. | NO. STORIES N | EARTHQUAKE PERIOD $T_E$ (SEC) | STORY HEIGHT h (IN) | C.I.T. RECORD NO. | DIRECTION | BASE SPECTRAL VELOCITY $S_v$ (IN/SEC) | TOP INSTRUMENT LEVEL $L_T$ | MEASURED DRIFT $\Delta_m$ | NORMALIZED THEORETICAL DRIFT $\Delta/\Gamma$ |
|---|---|---|---|---|---|---|---|---|---|
| 38 | 7 | 0.58 | 130 | F089 | S37W | 6.0 | 8 | 0.0018 | 0.00061 |
| 39 | 10 | 1.59 | (150)[1] | O199 | N62W | 15.0 | 10 | 0.0019 | 0.0028 |
| 40 | 22 | 2.17 | (150) | Q236 | E | 7.0 | 23 | 0.00059 | 0.00073 |
| 41 | 7 | 1.48 | 113 | C048 | N | 32.0 | 8 | 0.0047 | 0.0095 |
| 42 | 9 | 0.90 | 157 | F095 | S02W | 12.0 | 9 | 0.00059 | 0.0014 |
| 46 | 8 | 0.53 | 129 | P214 | S89W | 10.0 | 7 | 0.0035 | 0.0011 |
| 50 | 11 | 1.01 | 144 | S258 | N29E | 8.0 | 13 | 0.00091 | 0.00074 |
| 51 | 7 | 1.04 | (150) | J145 | S | 15.0 | 8 | 0.0011 | 0.0024 |
| 52 | 14 | 1.09 | (150) | Q233 | S12W | 8.0 | 13 | 0.00149 | 0.0016 |
| 53 | 12 | 2.90 | 159 | H115 | N79W | 30.0 | 13 | 0.0067 | 0.0073 |
| 58 | 13 | 2.30 | 153 | N192 | N29E | 15.0 | 14 | 0.0031 | 0.0028 |
| 59 | 12 | 1.26 | 165 | P217 | S | 10.0 | 12 | 0.00098 | 0.0011 |
| 61 | 12 | 1.52 | 148 | E075 | N | 12.0 | 11 | 0.0012 | 0.0020 |
| 63 | 11 | 1.33 | 175 | D065 | S | 10.0 | 10 | 0.00091 | 0.0013 |
| 64 | 7 | 0.61 | 146 | E072 | N75W | 8.0 | 6 | 0.00059 | 0.0011 |
| 66 | 15 | 1.65 | 131 | S255 | N08E | 8.0 | 17 | 0.00091 | 0.0010 |
| 67 | 9 | 0.62 | 149 | F092 | S62E | 6.0 | 9 | INDET.[3] | 0.00044 |
| 68 | 19 | 2.08 | (150) | M180 | S | 3.4 | 19 | 0.00035 | 0.00040 |
| 69 | 9 | 1.03 | 171 | G108 | E | 12.0 | 10 | 0.0012 | 0.0013 |

Table 2-4. Summary of 1971 San Fernando Earthquake Strong Motion Data for Steel Frame Buildings (References [15, 17])

| BUILDING NO. | NO. STORIES N | EARTHQUAKE PERIOD $T_E$ (SEC) | STORY HEIGHT h (IN) | C.I.T. RECORD NO. | DIRECTION | BASE SPECTRAL VELOCITY $S_v$ (IN/SEC) | TOP INSTRUMENT LEVEL $L_T$ | MEASURED DRIFT $\Delta_m$ | NORMALIZED THEORETICAL DRIFT $\Delta/T$ |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 12 | 2.09 | (150)(1) | H121 | W | 12.0 | 12 | .00267 | .000672 |
| 8 | 27 | 4.52 | 165 | R249 | N44E | 18.0 | 29 | .00286 | .00114 |
| 9 | 20 | 3.52 | 162 | D059 | S44W | 12.0 | 21 | .00165 | .000816 |
| 11 | 7 | 1.82 | 170 | S267 | E | 7.0 | 8 | .00129 | .000671 |
| 13 | 15 | 3.52 | 166 | N188 | N54E | 10. | 17 | .00178 | .000887 |
| 14 | 20 | 4.12 | 154 | L174 | N54E | N/A(2) | 21 | .00439 | N/A |
| 19 | 39 | 4.57 | 152 | C054 | S38W | 20.0 | 39 | .00166 | .000964 |
| 20 | 15 | 2.90 | 152 | C051 | N54W | 10.0 | 17 | .00136 | .000797 |
| 21 | 32 | 3.39 | 122 | Q241 | N37E | 18.0 | 33 | .00300 | .000974 |
| 24 | 17 | 1.06 | 175 | K157 | S53E | 14.0 | 17 | .00450 | .000312 |
| 27 | 10 | 2.71 | 193 | M176 | N37E | 24.0 | 10 | .00346 | .00211 |
| 30 | 15 | 3.22 | 202 | F078 | S40W | 17.0 | 15 | .00253 | .001134 |
| 33 | 8 | 1.79 | (150) | N/A | N/A | N/A | N/A | N/A | N/A |
| 44 | 42 | 6.02 | 172 | G112 | N38E | 10.0 | 42 | .00163 | .000523 |
| 45 | 9 | 1.59 | (150) | E083 | E | 8.5 | 8 | .00307 | .000627 |
| 47 | 14 | 2.74 | 151 | R248 | S | 10.0 | 14 | N/A | .000812 |
| 48 | 11 | 2.38 | (150) | R246 | E | 13.0 | 12 | .00365 | .001174 |
| 55 | 15 | 3.44 | 170 | I137 | S09W | 24.0 | 19 | .00380 | .002033 |
| 60 | 31 | 5.35 | 172 | N/A | N/A | N/A | N/A | N/A | N/A |
| 62 | 21 | 3.56 | 152 | S266 | W | 19.0 | 21 | N/A | .00133 |
| 65 | 30 | 4.65 | 164 | S262 | N83W | 24.0 | 33 | .00423 | .00142 |
| 70 | 9 | 1.35 | 152 | G110 | S08W | 8.0 | 9 | .00120 | .000494 |

(1) Average story height of 150 inches assumed

(2) N/A - Not available

intensity zone. Structural damage accounted for approximately 20% of this damage, on the average. Thus, the mean damage ratio for structural damage to post-1947 steel and reinforced concrete buildings (as a fraction of total building replacement cost) at MMI = VII is about 0.1%.

Reference [18] was published in early 1973 before the existing strong motion records became available, and therefore does not include such information. Ground motion intensity is reported only in terms of MMI. It was recognized that existing empirical relationships between MMI and peak ground velocity, and between peak ground velocity and spectral psudo-velocity, could be used to estimate $S_v$ from values of MMI listed for building location codes in Reference [18]. However, it was also recognized that for a significant number of those buildings, actual strong motion data could be used directly. In an effort to use the best available information in this study, the original damage data base was acquired from M.I.T. Data from Tables 2-3 and 2-4 were correlated with the damage data insofar as possible.

Estimates of interstory drift based on actual strong motion records were obtained for 5 of the 59 steel buildings and 12 of the 59 concrete buildings. For the remaining buildings, interstory drift was calculated for each building on the basis of Equation (2-1) and the methods of Reference [1] summarized as follows:

1. Compute peak ground velocity, $V_s$, from MMI using the equation

$$\log V_s = -1.973 + 0.375 \text{ MMI} \qquad (2\text{-}4)$$

2. Assume 5% damping ($\beta$ = .05) . Compute a dynamic amplification factor, $(AF)_\beta$ using the equation

$$(AF)_\beta = 2.18 - 0.147 \log T - 0.633 \log (\beta \times 100) \qquad (2\text{-}5)$$

3. Compute spectral velocity using the equation

$$S_v(T,\beta) = (AF)_\beta V_s \tag{2-6}$$

4. Compute interstory drift using Equation (2-1)

$$\Delta = \Gamma \left(\frac{T}{2\pi Nh}\right) S_v(T,\beta)$$

5. Compute a ductility, $\mu$, defined as

$$\mu = \frac{\Delta}{\Delta_y} \tag{2-7}$$

where $\Delta_y$ is a measure of interstory drift at yield, assumed to be 0.0044 for reinforced concrete and 0.0077 for steel buildings. These are average values derived from estimates presented in Section 2.3.

If $\mu > 1$ , the following steps are executed until convergence is reached (Reference [1]):

6. Adjust damping for inelastic response using the equation

$$(\beta^* \times 100) = 2.16 + 5.2\mu - 0.74\mu^2 \tag{2-8}$$

7. Adjust the fundamental period for inelastic response using the equation

$$T^* = \frac{T}{2\pi}\left[\frac{3\pi}{2} + \sin^{-1}\left(\frac{1}{\sqrt{2\mu-1}}\right) + \sqrt{2\mu-2}\right](1-\beta^{*1.7}) \tag{2-9}$$

8. Adjust spectral velocity for inelastic response using the equation

$$S_v^*(T^*,\beta^*) = (AF)_\mu S_v(T^*\beta^*) \tag{2-10}$$

where

$$(AF)_\mu = \begin{cases} \sqrt{\mu} & : \quad T^* < T_{a\mu} \\ \sqrt{\mu}\left[1 - \left(\frac{T^*-T_{a\mu}}{T_a}\right)\right] & : \quad T_{a\mu} < T^* < T_a \\ 1 & : \quad T^* > T_a \end{cases} \tag{2-11}$$

and where

$$T_a = 2\pi\left(0.070\ V_s^{0.222}\right) \tag{2-12a}$$

$$T_{a\mu} = T_a/\sqrt{\mu} \tag{2-12b}$$

9. Compute inelastic response, $\Delta^*$, using

$$\Delta^* = \Gamma\left(\frac{T^*}{2\pi Nh}\right) S^*_v(T^*,\beta^*)$$

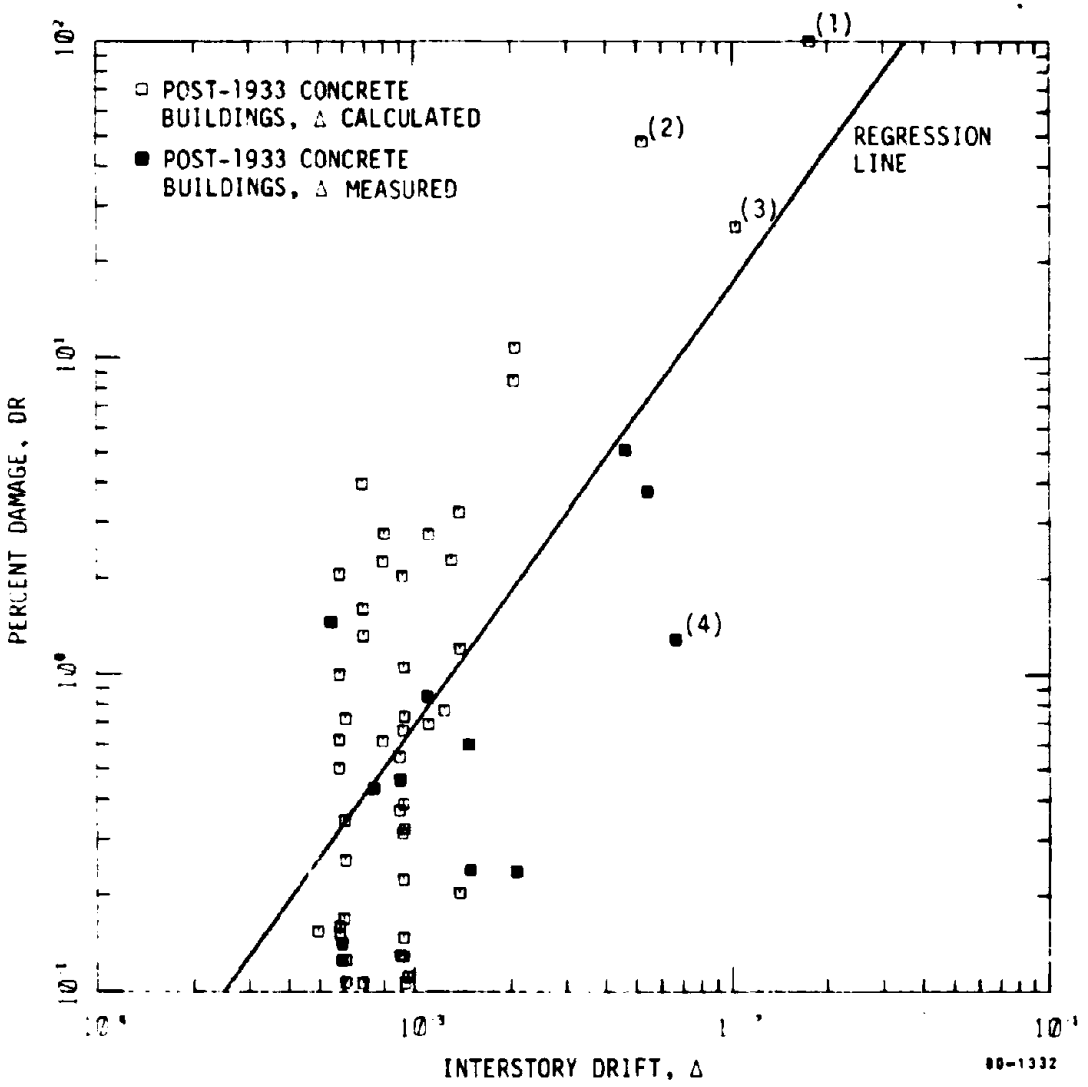
10. Test for convergence of $\Delta^*$, $T^*$, and $\beta^*$ . If any of these quantities differ from their original (preceding) values ($\Delta$, $T$, $\beta$) by more than 1%, then compute

$$\mu^* = \Delta^*/\Delta_y \tag{2-13}$$

and repeat steps (6) through (10).

It should be understood that the above procedure is not used whenever "measured" values of interstory drift, $\Delta_m$, are available. Also, for the three reinforced concrete buildings having damage ratios greater than 20%, estimated levels of peak ground velocity from strong motion instruments in the area were used in place of Equation (2-4).

Figures 2-5 and 2-6 show the damage data points plotted versus interstory drift for reinforced concrete and steel buildings, respectively, along with the results of a linear regression analysis in each case.




The following data points are identified on the basis of prior publication [15]:

(1) Olive View Hospital
(2) Holy Cross Hospital
(3) Indian Hills Medical Center
(4) Bank of California Building, Ventura Blvd.

Figure 2-5. Percent Damage Versus Interstory Drift for Post-1933 Reinforced Concrete Structures, 5-20 Stories

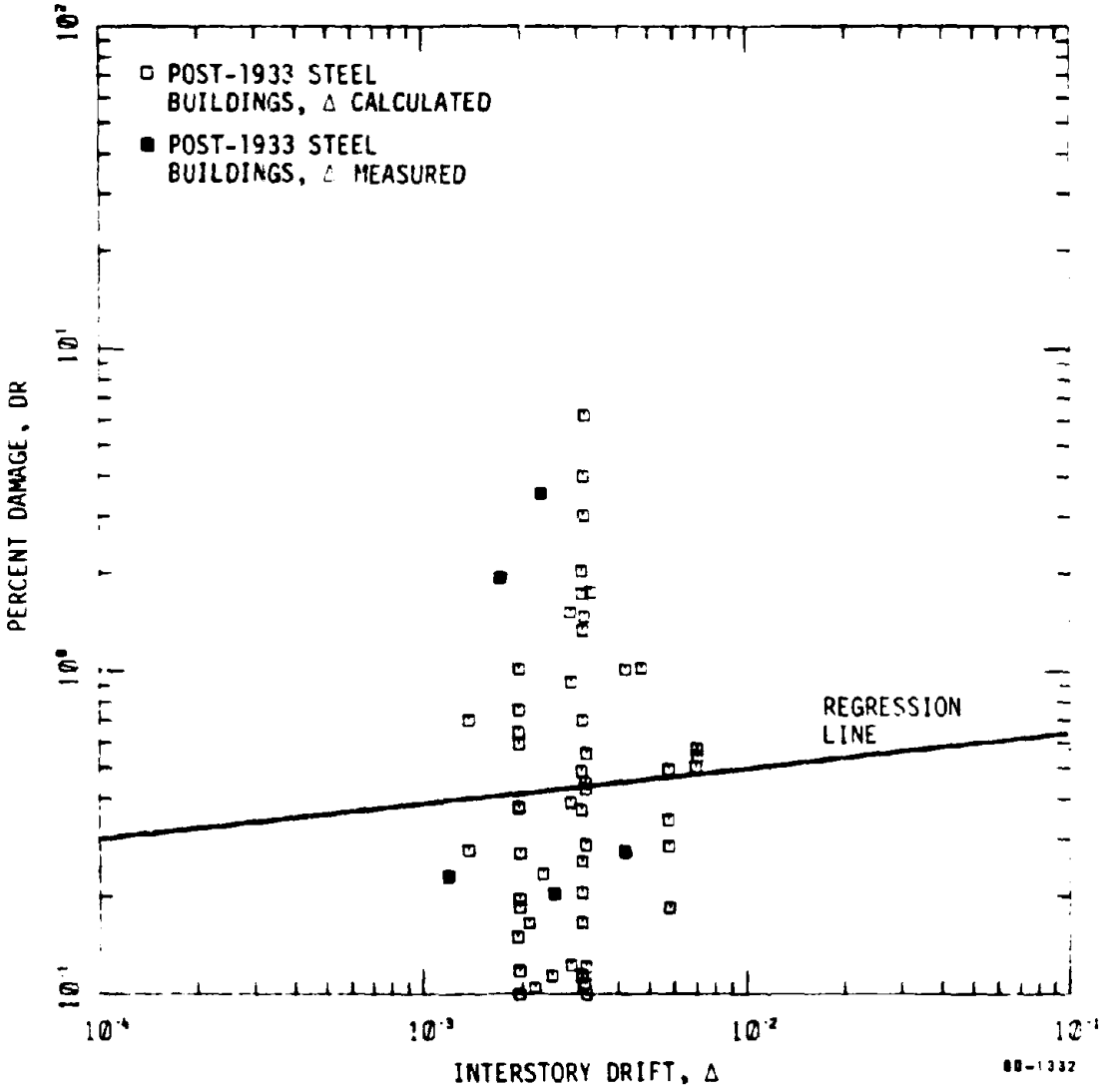



Figure 2-6. Percent Damage Versus Interstory Drift for Post-1933 Steel Frame Structures, 5-20 Stories

The correlation of damage ratio, DR, with interstory drift, $\Delta$ ,is seen to be virtually zero in the case of steel buildings; there was no case of a highly damaged steel building reported.

One might question whether much was gained by relating damage ratio to interstory drift, as opposed to MMI. Figures 2-7 and 2-8 show the same points plotted as a function of MMI, along with the results of a linear regression of DR on MMI. It is apparent that not much has been gained in the way of improving correlation, as others have shown. The biggest problem, of course, is the lack of data at the higher damage levels. Section 2.3 addresses this problem.

### 2.2.2 Wind Damage Data

A convenient wind damage data base such as that discussed for earthquake does not exist. Nevertheless, damage probability matrices similar to those which have been developed for earthquake damage, have also been developed for wind damage. They are based on a survey of expert opinion [14]. From these damage probability matrices, a mean damage ratio can be derived as a function of wind speed. As in the case of earthquake, where MMI was used as a measure of intensity, wind speed is the measure of intensity used here. A relationship is therefore sought between wind speed and interstory drift, so that wind damage may be expressed as a function of interstory drift.

The assumptions used in deriving Equation (2-1) for interstory drift due to earthquake loading can be used again to derive an expression for interstory drift due to wind loading. In particular, if a building is idealized as a simple shear beam, the variation of shear stiffness, k(z), along the building is determined to within a constant by the equation

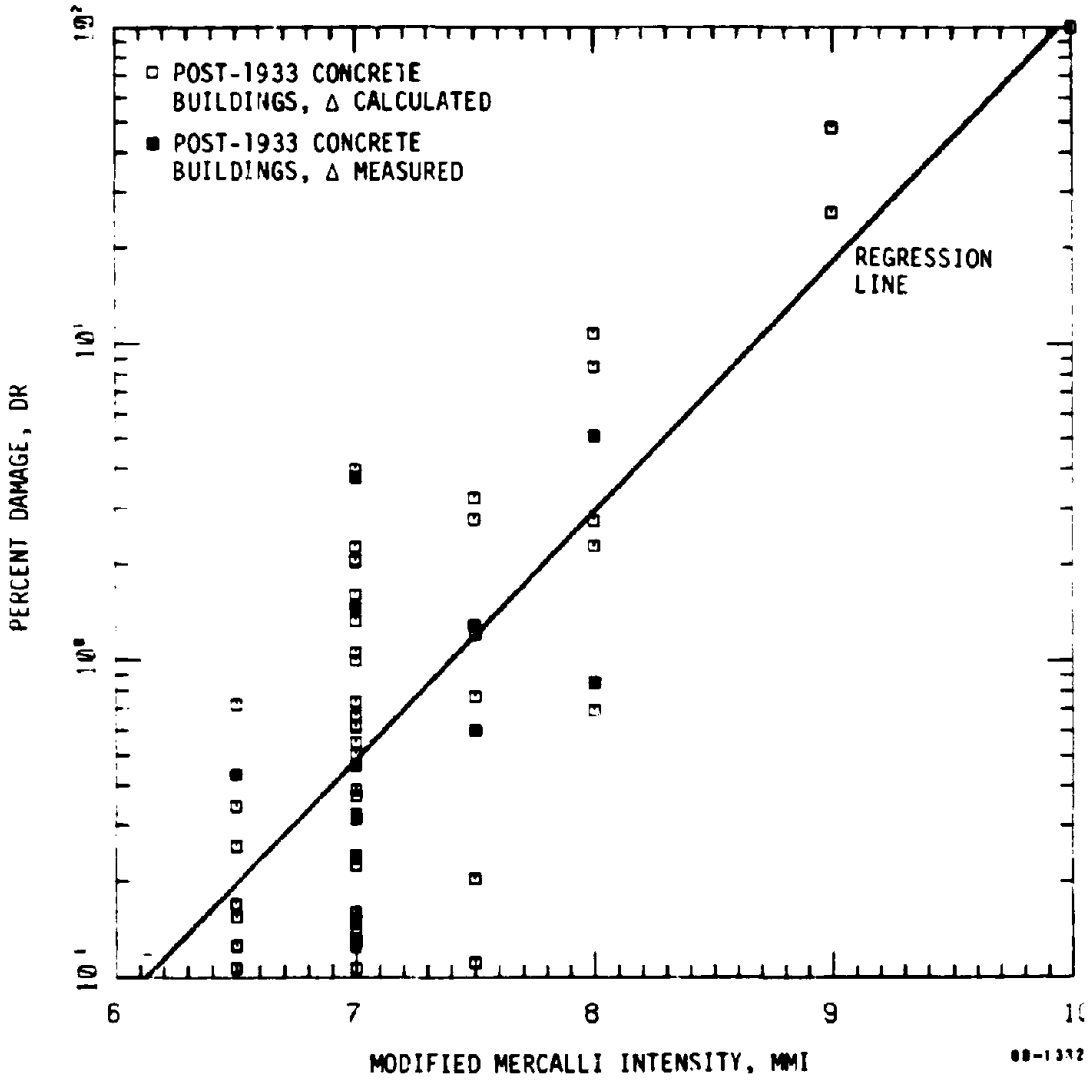



Figure 2-7. Percent Damage Versus Modified Mercalli Intensity for Post-1933 Reinforced Concrete Structures, 5-20 Stories

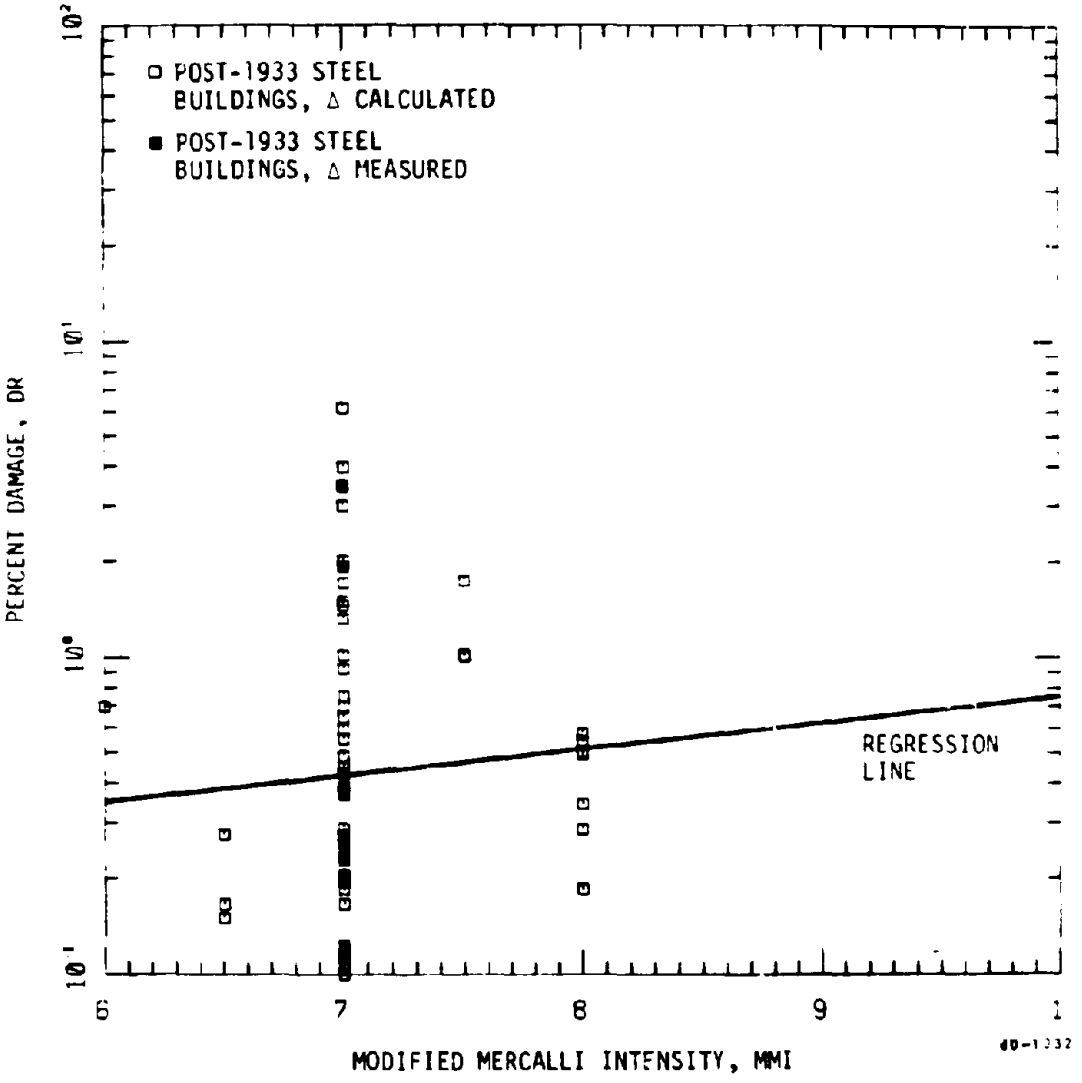



Figure 2-8. Percent Damage Versus Modified Mercalli Intensity for Post-1933 Steel Frame Structures, 5-20 Stories

$$k(z)\,\frac{d^2x}{dz^2} + \frac{dk(z)}{dz}\,\frac{dx}{dz} = p(z) \qquad (2\text{-}14)$$

where k(z) is in units of force and x measures lateral deflection at a point z along the beam. The load distribution is denoted by p(z). Under the assumption that seismic response in the fundamental mode is a linear function of z, the inertial loading p(z), is linear so that $d^2x/dz^2 = 0$ and $dx/dz$ = constant. If p(z) is assumed to be proportional to the modal deflection, $\phi$, for seismic loading then k(z) is found to be

$$k(z) = k_b\left[1 - \frac{z}{H}\right]^2 \qquad (2\text{-}15)$$

where $k_b$ is the constant of integration interpreted as the base shear stiffness. In general, $k_b$ is determined by the base-shear design coefficient of the building. It will, of course, vary according to the size and weight of the building.

In the case of earthquake design, the base shear capacity is governed by building weight, because the lateral forces on the building are inertial loads arising from lateral motion. In the case of wind design, the area of the building against which the wind pressure forces act is also a significant factor. In any case, the lateral resistance capacity of a building (and hence its equivalent base shear stiffness) tends to be proportional to its size in such a way that interstory drift is approximately proportional to wind pressure at some given height above ground.

The wind pressure distribution on a building is usually expressed in the form

$$P(z) = \frac{1}{2} C_p C_g \rho V_0^2 \left(\frac{z}{z_0}\right)^{2\alpha} \tag{2-16}$$

where $C_p$ and $C_g$ are the shape coefficient and gust factor, respectively, $\rho$ is the density of air, and $V_0$ is the wind speed measured at height $z_0$ above ground. The exponent $\alpha$ determines the variation of wind speed with height above ground and tends to decrease with increasing "roughness" of the surrounding area. For major cities $\alpha$ is smallest, and is assumed to be $\alpha = 1/7$ in Reference [1]. The standard height $z_0$ is usually taken to be 30 feet. In this case, the pressure distribution, $P(z)$, is approximately linear for buildings of up to 20 stories, so that a shear stiffness distribution as given by Equation (2-15) results in a linear deflection of the building, i.e., $\Delta = dx/dz$ constant.

Let us suppose that a building is designed to an allowable base shear, $F_a$, equal to some fraction of the base shear $F_y$, required to cause first yield. Then

$$F_a = CF_y \; ; \; C < 1 \tag{2-17}$$

It follows that the interstory drift, $\Delta_a$ corresponding to $F_a$ will be

$$\Delta_a = C \, \Delta_y \tag{2-18}$$

where $\Delta_y = d/h = F_y/k_b$ .

Reference [1] considers the critical ductility, $\mu_c = \Delta_c/\Delta_y$ , to be unity under sustained wind loading. This is a load condition of long duration compared with the fundamental period of the structure, resulting in a large number of cycles of oscillatory motion of amplitude as high as $\Delta_y$ . The structural damage level associated with $\mu_c$ is defined to be 50%. Loads corresponding to $\mu_c$ are considered to be ultimate dynamic

loads in the sense that structures damaged in excess of 50% are considered to be "totally damaged"; i.e., repair of the structures is not economically feasible.* The wind velocity corresponding to a median damage ratio of 50% is herein defined accordingly to be the ultimate wind velocity, $V_{ult}$.

Reference [14] presents a damage probability matrix for high-rise buildings. Steel and reinforced concrete buildings are combined in this category. From the matrix (Table 2-5), an ultimate wind speed of $V_{ult}$ = 250 mph is determined by the median damage ratio of 50%. The corresponding interstory drift is by definition $\Delta_y$ . Interstory drift for other wind speeds is then given by the equation

$$\Delta = \Delta_y \left(\frac{V}{V_{ult}}\right)^2 \tag{2-19}$$

which follows directly from Equations (2-14) and (2-16). If the average interstory drift-to-yield for reinforced concrete and steel buildings is taken to be $\Delta_y$ = .006 , the median wind damage ratio for high-rise buildings can be plotted as a function of interstory drift as shown in Figure 2-9.

There is a hidden assumption implicit in the log-linear relationship illustrated in Figure 2-9. The assumption is that structural damage tends to be a constant fraction of total damage which includes both structural and nonstructural damage. Under this asssumption 50% structural damage implies 50% total damage which permits $\Delta_y$ to be associated with 50% total damage. The assumption appears to be reasonable in the case of earthquake damage where structural damage at MMI=7 was found to be approximately 20% of total damage, while (at the other end of the damage scale) the value of the structure has been estimated to be approximately 25% of the total value of the building [13]. In the

*By law, structures damaged in excess of 50% must be brought entirely up to current code.

Table 2-5. Damage Probability Matrix for Four or More Story Structures (Reference [14])

| DAMAGE STATE | PERCENT DAMAGE* | INTENSITY (WIND SPEED-MPH) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 50 | 75 | 100 | 125 | 150 | 200 | 250 | 300 |
| NONE | 0 - 0.05% | .994 | .950 | .811 | .439 | .239 | .151 | .089 | .006 |
| LIGHT | .05 - 1.25% | .006 | .048 | .169 | .356 | .224 | .084 | .078 | .039 |
| MODERATE | 1.25 - 7.5 % | .0 | .002 | .019 | .188 | .344 | .291 | .156 | .172 |
| HEAVY | 7.5 - 65 % | .0 | .0 | .001 | .018 | .180 | .280 | .261 | .211 |
| VERY SEVERE | 65 - 100 % | .0 | .0 | .0 | .0 | .012 | .132 | .198 | .247 |
| COLLAPSE | 100 % | .0 | .0 | .0 | .0 | .0 | 0.63 | .219 | .326 |

*Ratio of repair cost to total structure replacement cost (X 100)

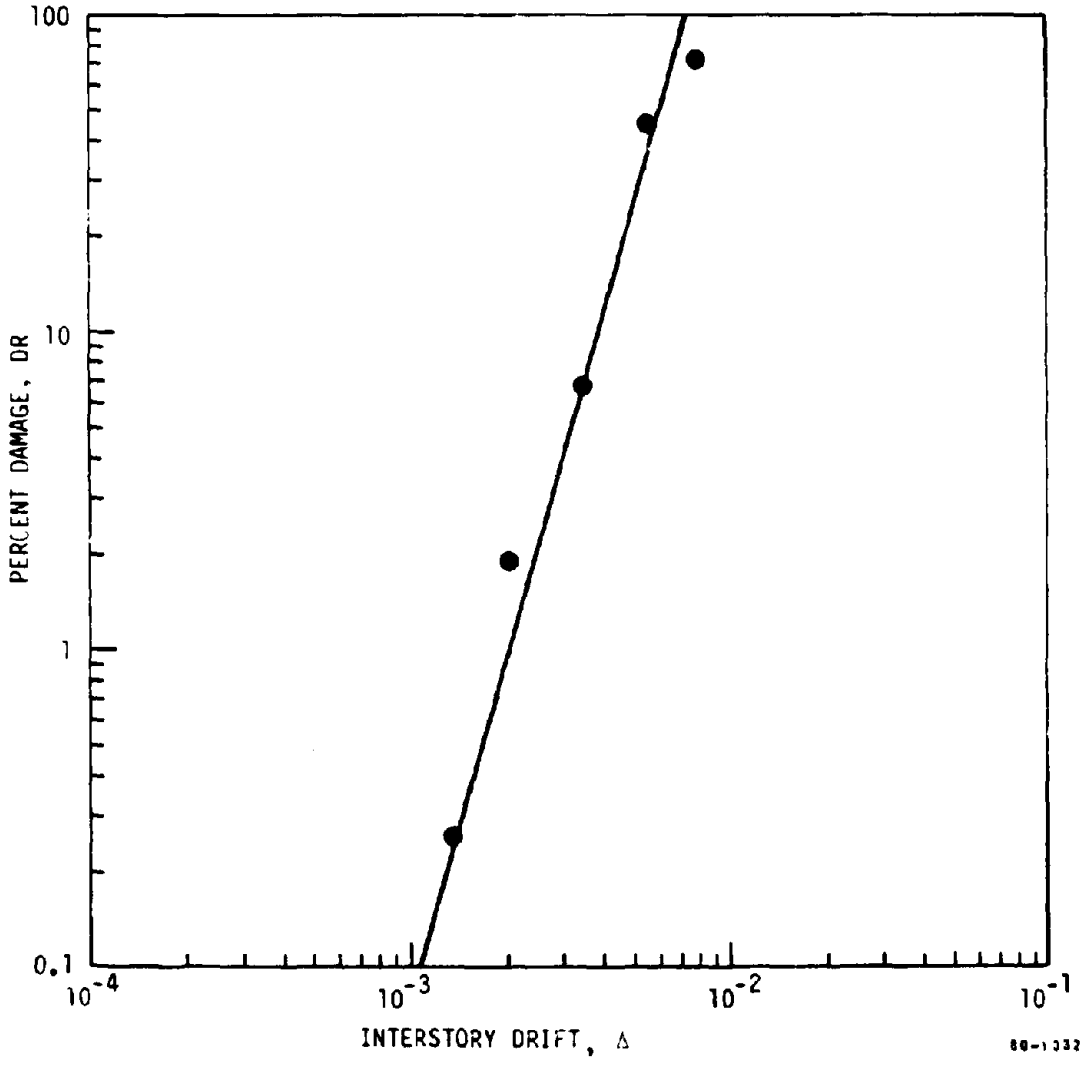



Figure 2-9. Median Wind Damage Versus Interstory Drift for All High-Rise Buildings, 4 or More Stories

absence of any specific information to the contrary, this assumption is made.

It is of interest to note from Figure 2-9 that a damage level of 0.5% corresponds to an interstory drift of approximately $\Delta = \Delta_y/4$ , and a wind speed of 125 mph. The wind speed associated with the damage threshold of 0.1% is approximately 100 mph, not an unreasonable number for design purposes. In retrospect, the average high-rise building should be able to withstand a wind speed of 100 mph without damage. By deductive reasoning, the allowable drift in Equation (2-18), therefore, would be $\Delta_y/6$, or $C = 0.17$ .

## 2.3 Interpretation of Expert Judgment

Structural engineers understand building performance in terms of interstory drift. Therefore, quantities such as drift-to-yield, $\Delta_y$, and ductility-to-failure, $\mu_f$, have established definitions, at least in the sense of structural deformation under static loads as illustrated below. Drift-to-yield is the value of interstory drift at which "first yield" for the structural system being considered is presumed to occur. Ductility-to-failure is that multiple of $\Delta_y$ at which structural damage exceeds 50%, thereby rendering the structure irrepairable. Some experimentally determined values of $\Delta_y$ and $\mu_f$ are listed in Tables 2-6, 2-7 and 2-8 for various steel, concrete, and masonry systems, respectively.

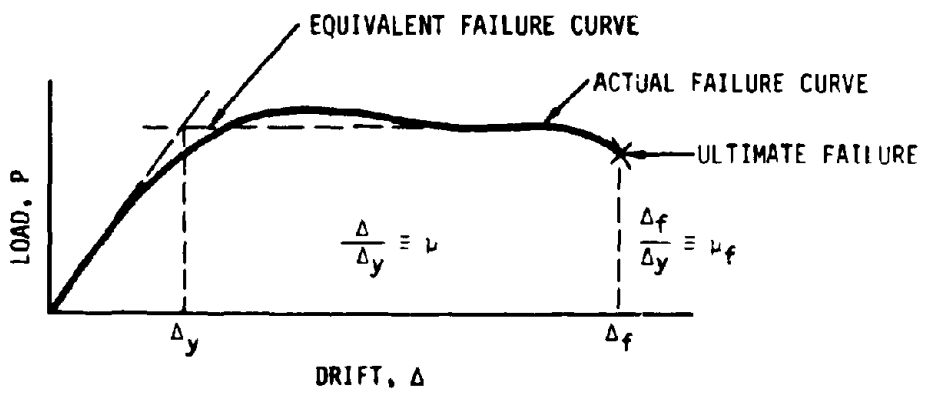


Table 2-6. Capacity of Steel Structures

| ITEM | $\Delta_y$ | $\mu_f$ | REFERENCE |
|---|---|---|---|
| 1 STORY MOMENT FRAME (PINNED AT BASE) | .0070 | 7 | 19 |
| 1 STORY MOMENT FRAME (FIXED AT BASE) | .0100 | 6 | 19 |

Table 2-7. Capacity of Concrete Structures

| ITEM | $\Delta_y$ | $\mu_f$ | REFERENCE |
|---|---|---|---|
| CONCRETE BUILDING FRAME | .0063 | 6.5 | 20 |
| 5 STORY FRAME WITH TIED COLUMNS | .0048 | 4 | 21 |
| 5 STORY FRAME WITH SPIRAL AND TIED COLUMNS | .0071 | 5 | 22 |
| 5 STORY FRAME WITH DROP PANEL SYSTEM | .0067 | 2 | 22 |
| REINFORCED FRAME PLUS REINFORCED WALL PANEL | .0022 | 3 | 23 |
| REINFORCED FRAME PLUS UNREINFORCED WALL PANEL | .0020 | 2 | 23 |
| 1 - STORY DUCTILE FRAME | .0021 | 6 | 23 |
| 24 - STORY DUCTILE FRAME | .0017 | 8 | 23 |
| REINFORCED CONCRETE SHEAR WALL WITH AND WITHOUT SMALL OPENINGS | .0028 | 4 | 24, 25 |
| CONCRETE FRAME WITH REINFORCED CONCRETE INFILL PANEL | .0012 | 5 | 26 |
| CONCRETE FRAME WITH MANSONRY INFILL PANEL | .0017 | 3 | 23 |
| CONCRETE FRAME WITH REINFORCED WALL PANEL | .0017 | 3 | 27 |
| PRESTRESSED CONCRETE BEAM | .0050 | 3 | 28 |
| PRESTRESSED CONCRETE BRIDGE | .0046 | 6 | 29 |

Table 2-8. Capacity of Masonry Structures

| ITEM | $\epsilon_y$ | $\mu_f$ | REFERENCE |
|---|---|---|---|
| REINFORCED BRICK (H/D = 1) | .0015 | 1 | 30 |
| REINFORCED BRICK (H/D = 1) | .0020 | 1 | 30 |
| HOLLOW BRICK | .0016 | 1.5 | 30 |
| UNREINFORCED CMU (F-P) | .0004 | 1 | 23 |
| REINFORCED CMU (F-F) H/D = 0.4 | .0022 | 2.3 | 31 |
| REINFORCED CMU (F-F) H/D = 1 GROUT AT STEEL | .0002 | 20 | 31 |
| REINFORCED CMU (F-F) H/D = 1 SOLID GROUT | .0004 | 4 | 31 |
| REINFORCED CMU (F-F) H/D = 2 SPECIAL TIES, HORIZONTAL STIRRUPS | .0016 | 5 | 31 |
| REINFORCED CMU (F-F) H/D = 2 NO HORIZONTAL STIRRUPS | .0009 | 5 | 31 |
| REINFORCED CMU (F-F) H/D = 2 NO HORIZONTAL STIRRUPS | .0008 | 6 | 31 |
| REINFORCED CMU (F-F) H/D = 2.7 NO HORIZONTAL STIRRUPS | .0008 | 4 | 31 |
| REINFORCED CMU (F-F) H/D = 3.0 SPECIAL TIES | .0007 | 10 | 31 |

H = Height
D = Depth
CMU = Concrete Masonry Units
F = Fixed
P = Pinned

Reference [1] documents values of $\Delta_y$ and $\mu_f$ estimated by three individuals experienced in structural engineering and damage evaluation. These values are shown in Tables 2-9 and 2-10. They reflect separate consideration by type of structural system (frame, shear wall), material (steel, concrete, masonry) and quality (good, average, poor). They are seen to be in substantial agreement with corresponding values in Tables 2-6 through 2-8.

Ideally, the force-deflection properties of the structure are linear up to the point of first yield implying no damage, at least under static loads. In reality, buildings do not behave linearly under deformation approaching first yield, even under static loading. This is particularly true of concrete and masonry structures. Variation in material properties and construction quality alone account for some "premature" yielding. Design variation can be expected to contribute to this effect (at least to the extent that average values of $\Delta_y$ and $\mu_f$ are quoted) and of course, dynamic effects such as duration and higher mode participation contribute further.

Here again, in searching for a realistic relationship between drift-to-yield and damage, one is attracted to the notion of a "damage threshold." The considerations affecting the definition and selection of such a quantity are many; they are not as simple as might be expected. For example, one might ask:

- whether the damage threshold should be associated with $\Delta_y$ or some fraction of $\Delta_y$ ,

- whether it should be the same for all types of buildings, and if not, what parameters govern its value,

- whether it should be the same for both earthquake and wind loading, or for all kinds of earthquakes, and if not, what parameters govern its value, and

Table 2-9. Judged Values of Drift to Yield (in/in) for Various Quality Ratings [1]

| STRUCTURAL SYSTEM | MATERIAL | $\Delta_y$ - DRIFT TO YIELD BY QUALITY | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | GOOD | | | AVERAGE | | | POOR | | |
| | | MO | JO | JA | MO | JO | JA | MO | JO | JA |
| FRAME | STRUCTURAL STEEL | .010 | .015 | .013 | .005 | .010 | .009 | .002 | .006 | .004 |
| | CONCRETE: POURED IN PLACE | .005 | .010 | .012 | .003 | .006 | .008 | .002 | .003 | .004 |
| | PRECAST CONCRETE | .010 | .004 | .003 | .005 | .002 | .002 | .002 | .001 | .001 |
| SHEARWALL | CONCRETE: POURED IN PLACE | .002 | - | .010 | .001 | - | .007 | .001 | - | .003 |
| | PRECAST CONCRETE | .002 | .004 | .003 | .001 | .002 | .002 | .001 | .001 | .001 |
| | MASONRY | .003 | .007 | .010 | .002 | .005 | .007 | .001 | .003 | .003 |

MO - DONALD MORAN
JO - ROY JOHNSTON
JA - JACK JANNEY

Table 2-10. Judged Values of Ductility-to-Failure for Various Quality Ratings [1]

| STRUCTURAL SYSTEM | MATERIAL | $\mu_f$ - DUCTILITY TO FAILURES BY QUALITY | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | GOOD | | | AVERAGE | | | POOR | | |
| | | MO | JO | JA | MO | JO | JA | MO | JO | JA |
| FRAME | STRUCTURAL STEEL | 20 | 16 | 17 | 10 | 8 | 11 | 5 | 2 | 5 |
| | CONCRETE: POURED IN PLACE | 10 | 8 | 13 | 5 | 4 | 9 | 2 | 2 | 4 |
| | PRECAST CONCRETE | NA | 2 | 13 | 3 | 2 | 9 | 2 | 2 | 4 |
| SHEAR WALL | CONCRETE: POURED IN PLACE | 5 | 4 | 11 | 3 | 3 | 7 | 1 | 2 | 4 |
| | PRECAST CONCRETE | NA | 2 | 5 | 2 | 2 | 3 | 1 | 2 | 2 |
| | MASONRY | 5 | 6 | 11 | 3 | 4 | 7 | 1 | 2 | 4 |

MO - DONALD MORAN
JO - ROY JOHNSTON
JA - JACK JANNEY
NA - NOT APPLICABLE

- whether it should depend on the general form of the damage curve, e.g., the log-linear regression curves shown in Figures 2-5 and 2-6.

These questions are interrelated and pose a rather complex decisional problem.

This complexity alone suggests that the damage curves relating damage ratio to interstory drift be kept as simple as possible, if only to minimize the number of parameters (such as damage threshold and its associated value of interstory drift) which must be quantified. The intent here is to choose parameters which have some direct correspondence with, and therefore can be estimated from actual damage data. The nature of the data shown in Figures 2-5 and 2-6 also suggests that the number of parameters be minimized. In fact, there is barely enough information in the data to define a linear relationship with much confidence. In the case of steel buildings, there is clearly not enough.

Several possible types of damage relationships were considered, including the extreme-value type distribution (e.g., out of 100 buildings of the same generic category, what is the probability of the highest damage ratio which might be observed for any of the buildings, given an event of a certain intensity). Other alternatives included the normal probability distribution function, the lognormal probability distribution function, and the simple log-linear (linear on a log-log scale) relationship. The latter was finally selected to represent the median damage ratio of those buildings which are damaged, i.e., which have a median damage ratio equal to or greater than the established damage threshold. The decision was based on the relative simplicity of this alternative compared with the others, and the absence of any compelling reason to choose otherwise.

Having made this choice, the problem of how to interpret expert judgment and/or component test data for establishing values of $\Delta_y$ and $\mu_f$ with respect to the general equation

$$\log DR = C_1 + C_2\left(\log \Delta - \log \Delta_0\right) \tag{2-20a}$$

or

$$\log DR = C_1 + C_2 \log \Delta \tag{2-20b}$$

was addressed. For the time being, $\Delta_0$ is considered to represent an arbitrary reference point for convenience. Clearly, a straight line is determined either by two points, or by a point and a slope. Since $\mu_f$ is by definition associated with 50% damage under static load conditions, one point is thereby determined. Under static loads, the interstory drift-to-failure, $\Delta_f$ is defined to be

$$\Delta_f = \Delta_y \mu_f \tag{2-21}$$

Under dynamic loading, which is oscillatory in nature, a critical ductility, $\mu_c$, is defined in Reference [1] as follows

$$\mu_c = \begin{cases} 1 \quad ; & \text{for wind (long duration)} \\ C(T,M)\mu_f ; & \text{for earthquake} \end{cases} \tag{2-22}$$

where C(T,M) is a multiplicative factor depending on the fundamental period of the building, T, and Richter magnitude, M. This factor ranges over the interval $[1 < C(T,M) < 1/\mu_f]$ ,and is evaluated by the empirically based relationship [1]

$$C(T,M) = \frac{T}{.0046\ e^M} \tag{2-23}$$

Conceptually, it relates to low-cycle fatigue and cumulative damage which is cycle-dependent.

It seems to be more meaningful to determine $C_1$ and $C_2$ in Equation (2-20a) by establishing a second point, rather than by trying to establish the slope, $C_2$, directly. Two pieces of information are required to do so: a value of $\Delta$ and a corresponding value of DR. The need for <u>two</u> additional pieces of information is stressed as opposed to one piece of information, say a value of DR at $\Delta_o$ or some arbitrarily specified fraction of $\Delta_y$. Clearly, DR at $\Delta_y$ is not suitable in general because the slope, $C_2$, is undefined when $\mu_c = 1$. If we do not choose to specify DR at $\Delta_y$, then some other value of $\Delta$ must be chosen.

Figure 2-9 suggests that in the case of wind damage, $\Delta = \Delta_y/4$ may be associated with DR = 0.5% for all buildings on the average. However, such a choice leaves unanswered the questions of whether the damage model should depend on the type of loading (earthquake or wind), and whether it should depend on the type of building, e.g., steel or reinforced concrete.

The fact that DR was found to be 0.5% for both steel and reinforced concrete buildings at an earthquake intensity of MMI = VII suggests a way of establishing a meaningful damage threshold, using Equation (2-1). Since structural damage should be negligible to buildings of proper design and construction at this intensity, the average interstory drift of a particular building under this loading should correspond to the interstory drift of that building at its "<u>significant damage threshold</u>." If the buildings in the M.I.T. damage data base can be characterized by an average story height, and $S_v$ is related to MMI (by Equation (2-4) for example) then having determined values of $\Gamma$ and T/N for both steel and concrete buildings enables one to calculate threshold values of $\Delta$. These values were found to be $\Delta_t = 0.0030$ for steel buildings and $\Delta_t = 0.00085$ for reinforced concrete buildings. The

ratios of $\Delta_t/\Delta_y$ for these two broad classes of buildings, using the averaged values of $\Delta_y = 0.0077$ for steel* and $\Delta_y = 0.0044$ for reinforced concrete** are then

$$\frac{\Delta_t}{\Delta_y} = \frac{0.0030}{0.0077} = 0.39 \quad : \text{ for steel}$$

$$\frac{\Delta_t}{\Delta_y} = \frac{0.00085}{0.0044} = 0.19 \quad : \text{ for reinforced concrete}$$

The average threshold ratio of

$$\frac{\Delta_t}{\Delta_y} = \frac{0.0015}{0.0060} = 0.25 \quad : \text{ for both steel and reinforced concrete}$$

in the case of wind damage is seen to fall between the two, indicating that the "significant damage threshold" may indeed be similar for the two hazards, while varying among different types of buildings.

Damage curves representing "expert judgment" may now be constructed. It is of particular interest to do so for the buildings represented in Figures 2-5 and 2-6 which were damaged during the San Fernando earthquake. In this case, the earthquake had a Richter magnitude of 6.5. Assuming the average building to have 10 stories, values of C(T,M) evaluated from Equation (2-23) are found to be

$$C(T,M) = 0.327 \quad : \text{ for reinforced concrete}$$
$$C(T,M) = 0.523 \quad : \text{ for steel}$$

*This is the geometrical average of the estimated values shown in Table 2-9 for steel frame structures of average quality.

**This is the geometrical average of the estimated values shown in Table 2-9 for (poured-in-place) concrete frame and (poured-in-place) concrete shear wall structures of average quality.

The two sets of points defining damage curves based on expert opinion for concrete and steel are then

| TYPE OF BUILDING | THRESHOLD $\Delta_t$ | THRESHOLD $DR_t$(%) | FAILURE $\Delta_c$ | FAILURE $DR_c$(%) |
|---|---|---|---|---|
| CONCRETE | .00085 | 0.5 | .0072 | 50 |
| STEEL | .0030 | 0.5 | .0350 | 50 |

The corresponding lines (labeled "Prior Estimate") are plotted in Figures 2-10 and 2-11 for comparison with the damage data and regression lines of Figures 2-5 and 2-6.

## 2.4 Combining Judgment with Data

Figures 2-10 and 2-11 illustrate both the promise and the folly of attempts which have been made to utilize the apparent wealth of damage data gathered from the 1971 San Fernando Earthquake. Figure 2-10 shows, for example, a meaningful trend which, although accompanied by a large degree of scatter would appear to be useful in evaluating the damageability of reinforced concrete buildings to future earthquakes. Adding to this hope is the relatively good agreement with professional judgment, not previously compared on this basis. Figure 2-11 shows, on the other hand, how limitations in the data have frustrated attempts to develop meaningful interpretations. Large scatter combined with an inadequate range of ground motion intensity can result in very weak trends which are obviously meaningless. What is needed is a way of combining both types of information so that (1) limited data may be used to validate and perhaps refine a general model, which (2) may then be used with greater confidence in situations where no data exist. This is the thrust of the present section, and the approach developed herein for assessing damageability.

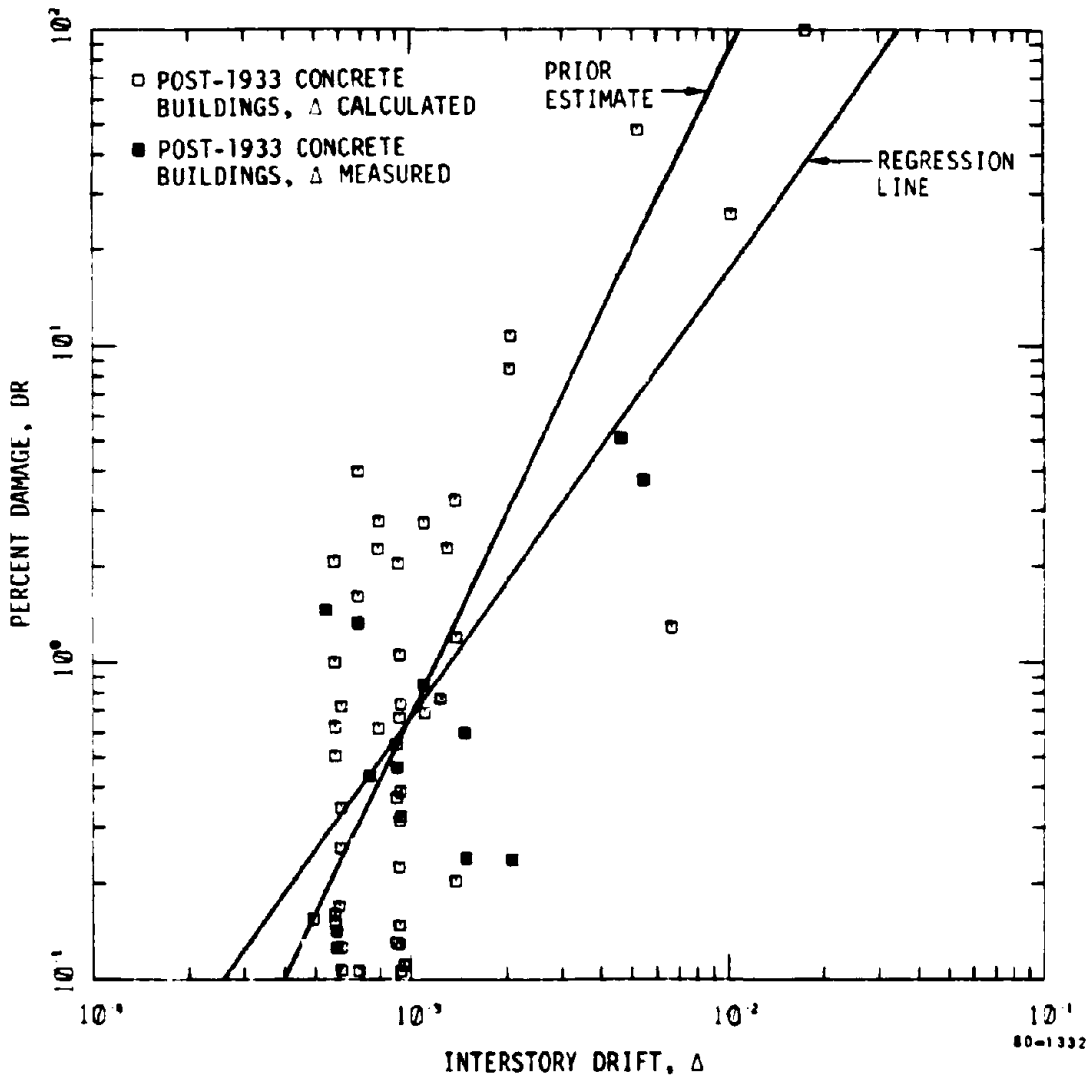



Figure 2-10. Comparison of Damageability Models Based on Professional Judgment and Earthquake Damage Statistics for Post-1933 Reinforced Concrete Buildings, 5-20 Stories

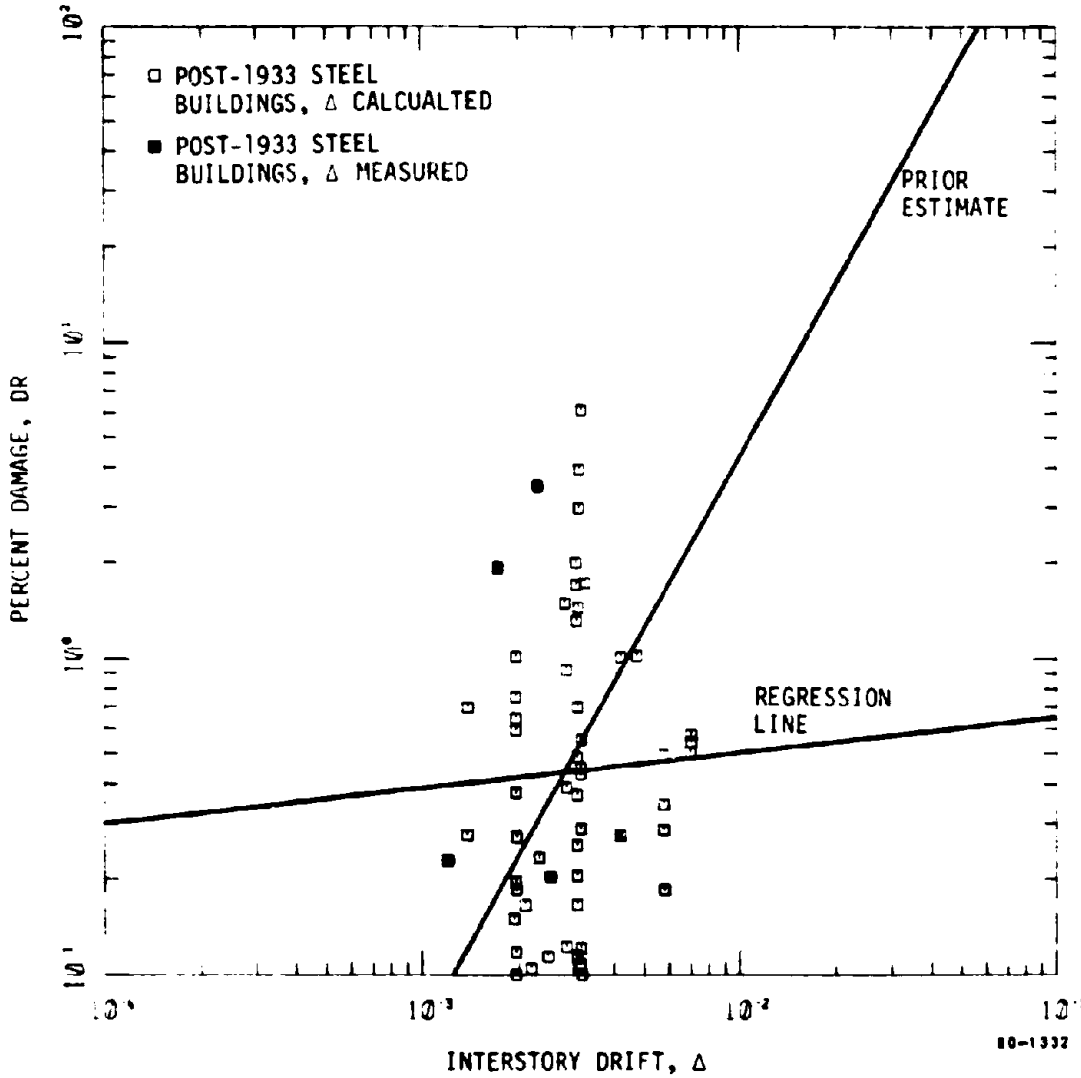



Figure 2-11. Comparison of Damageability Models Based on Professional Judgment and Earthquake Damage Statistics for Post-1933 Steel Frame Buildings, 5-20 Stories

Bayesian estimation theory [32, 33, 34] offers a rigorous procedure for combining engineering judgment with available data. The basic requirements for implementing the procedure are (1) that a suitable mathematical relationship (model) between the judgment and data be developed, and (2) that suitable probability laws be defined as a means of quantifying both judgment and data, either in a probabilistic or statistical sense, and (3) that the parameters of these mathematical relationships be quantifiable in some meaningful way. So far, we have accomplished the first step, namely to formulate a basic mathematical model. The form of the model is given by Equation (2-20).

Having interpreted the key parameters of professional judgment, $\Delta_y$ and $\mu_f$, in light of this model, we were able to determine values of $C_1$ and $C_2$ accordingly. These constants may now be expressed in the form

$$C_1 = \log DR_t \tag{2-23a}$$

$$C_2 = \frac{\log DR_c - \log DR_t}{\log \Delta_c - \log \Delta_t} \tag{2-23b}$$

With $C_1$ defined as above, we must choose $\Delta_0 \equiv \Delta_t$. Now we may write Equation (2-20) as

$$\log DR = \log DR_t + \left[\frac{\log DR_c - \log DR_t}{\log \Delta_c - \log \Delta_t}\right] \left(\log \Delta - \log \Delta_t\right) \tag{2-24}$$

Finally, when both $\Delta_c$ and $\Delta_t$ are expressed as multiples of $\Delta_y$, namely

$$\Delta_c = C(T,M)\mu_f \, \Delta_y = \mu_c \Delta_y \tag{2-25a}$$

$$\Delta_t = \mu_t \Delta_y \tag{2-25b}$$

where $\mu_t$, consistent with the definitions of $\mu_c$ and $\mu_f$, is defined to be a ductility less than unity associated with the threshold of significant damage, we find that

$$C_2 = \frac{\log DR_c - \log DR_t}{\log \mu_c - \log \mu_t}$$

or equivalently

$$C_2 = \frac{\log DR_c - \log DR_t}{\log \mu_f + (\log C - \log \mu_t)}$$

We shall assume that all of the parameters in $C_1$ and $C_2$ are either established by definition ($DR_c$, $\mu_t$) or are otherwise known with relatively high certainty (C), except for log $DR_t$ and log $\mu_f$. Given prior estimates for each of these, we shall try to obtain improved estimates via a Bayesian procedure. To do so, we must first establish probability laws for each. Lognormal distributions are therefore assumed for $DR_t$ and $\mu_f$ (or $\mu_c$), leading to normal distributions for log $DR_t$ and log $\mu_f$ (or log $\mu_c$). The assumption is also made that the data as displayed in Figures 2-5 and 2-6 are lognormally distributed with respect to the damage ratio, DR. Clearly, such an assumption is inappropriate for median damage ratios approaching unity, since by definition the damage ratio may not exceed unity. For damage ratios of 10% or less, the assumption constitutes a reasonable approximation. Inasmuch as a large majority of the data fall below the 10% damage level, the assumption is considered to be acceptable.

It is convenient to present the Bayesian estimator in matrix form, since many parameter values are typically estimated simultaneously from many data points. For notational purposes, the parameters are arrayed in a vector $\{r\}$, while the "data" are arrayed in a vector $\{u\}$. In Bayesian estimation, we are given a prior estimate of the parameters $\{r_0\}$, along with the associated covariance matrix $[S_{rr}]$. We are also given a set of "data points" $\{u_0\}$ along with its associated covariance matrix $[S_{\varepsilon\varepsilon}]$, where $\varepsilon$ denotes the "error of observation." We then seek to minimize the objective function

$$F = \{u_0 - u\}^T [S_{\varepsilon\varepsilon}]^{-1} \{u_0 - u\} + \{r_0 - r\}^T [S_{rr}]^{-1} \{r_0 - r\} \tag{2-26}$$

with respect to the individual parameter values, $r_j$, such that

$$\frac{\partial F}{\partial r_j} = 0 \tag{2-27}$$

This leads to the following recursive relationship

$$\{r\} = \{r_e\} + [S_{rr}]^{-1} + [T]^T [S_{\varepsilon\varepsilon}]^{-1} [T]^{-1} \times [S_{rr}]^{-1} \{r_0 - r_e\} + [T]^T [S_{\varepsilon\varepsilon}]^{-1} \{u_0 - u_e\} \tag{2-28}$$

where the vector $\{r_e\}$ represents the most recent estimate of the parameter vector, and $\{u_e\}$ represents the corresponding response vector computed from the model using parameter values $\{r_e\}$. The rectangular matrix $[T]$, called the sensitivity matrix, has elements defined by

$$T_{ij} = \partial u_i / \partial r_j \tag{2-29}$$

Whenever $\{u\}$ is a nonlinear function of $\{r\}$ (as in the present case), then Equation (2-28) is solved iteratively beginning with $\{r_e\} \equiv \{r_o\}$. (If $\{u\}$ were a linear function of $\{r\}$, the "iteration" would converge in a single step.) The iterative procedure continues until the objective function can be reduced no further. At this point we call the current estimate of $\{r\}$ the "revised estimate," $\{r^*\}$, and compute a revised covariance matrix

$$[S^*_{rr}] = [S_{rr}]^{-1} + [T]^T [S_{\varepsilon\varepsilon}]^{-1} [T]^{-1} \tag{2-30}$$

where $[T]$ is evaluated at $\{r^*\}$.

Whereas the conventional application of this estimator might treat each of the data points, shown in Figure 2-5 or 2-6 for example, as one element of the vector $\{u_o\}$, we have chosen to work the problem in two parts: first by performing the standard linear regression analysis to determine the slope and intercept of the assumed linear damage relationship, and second by using these estimated parameters along with their associated covariance matrix as "data." Thus we define the vector $\{u\}$ to be a two-element vector consisting of

$u_1$ = slope of linear damage relationship, $C_2$
$u_2$ = intercept of linear damage relationship, $C_1' = C_1 - C_2 \log \Delta_o$

where $C_1$, $C_2$ and $\log \Delta_o$ correspond to the terms in Equation (2-20). The "observed" values of $u_1$ and $u_2$ which constitute $\{u_o\}$ are provided by the slope and intercept of the linear regression. Also provided by the linear regression analysis are the elements of $[S_{\varepsilon\varepsilon}]$, the covariance matrix quantifying the "errors of observation." Specifically, these are

$$[S_{\varepsilon\varepsilon}] = \begin{bmatrix} \dfrac{\sigma^2}{\sum (x_i - \bar{x})^2} & -\dfrac{\bar{x}\sigma^2}{\sum (x_i - \bar{x})^2} \\ -\dfrac{\bar{x}\sigma^2}{\sum (x_i - \bar{x})^2} & \dfrac{\sigma^2 \sum x_i^2}{n \sum (x_i - \bar{x})^2} \end{bmatrix}$$

where

$x_i \equiv \log \Delta_i$

$\bar{x} \equiv \overline{\log \Delta}$

$\sigma \equiv$ the standard error of the estimate, log DR

The selected model parameters represented by $r_1$ and $r_2$ are

$r_1 = \log \mu_f \quad ; \; r_2 = \log DR_t$

In specifying $[S_{rr}]$, $r_1$ and $r_2$ were assumed to be uncorrelated. The prior estimate of $r_1$ was obtained by taking the geometric average* of appropriate values in Table 2-10. The corresponding variance of $r_1$ was also determined from those estimated values. The prior estimate of $r_2$ was chosen to be log 0.5, with a coefficient of variation of log 2, i.e., one standard deviation was assumed to be 100%.**

The results of the Bayesian estimation are shown graphically in Figures 2-12 and 2-13. A corresponding numerical summary is shown in Table 2-11.

*Geometric averages of parameter values are used when estimating their logarithms.

**The results of the Bayesian estimation are rather insensitive to the prior estimates of $r_2$. $DR_t$ (prior) is taken to be 0.5% based on the Mean Damage Ratio for MMI=VII in [18].

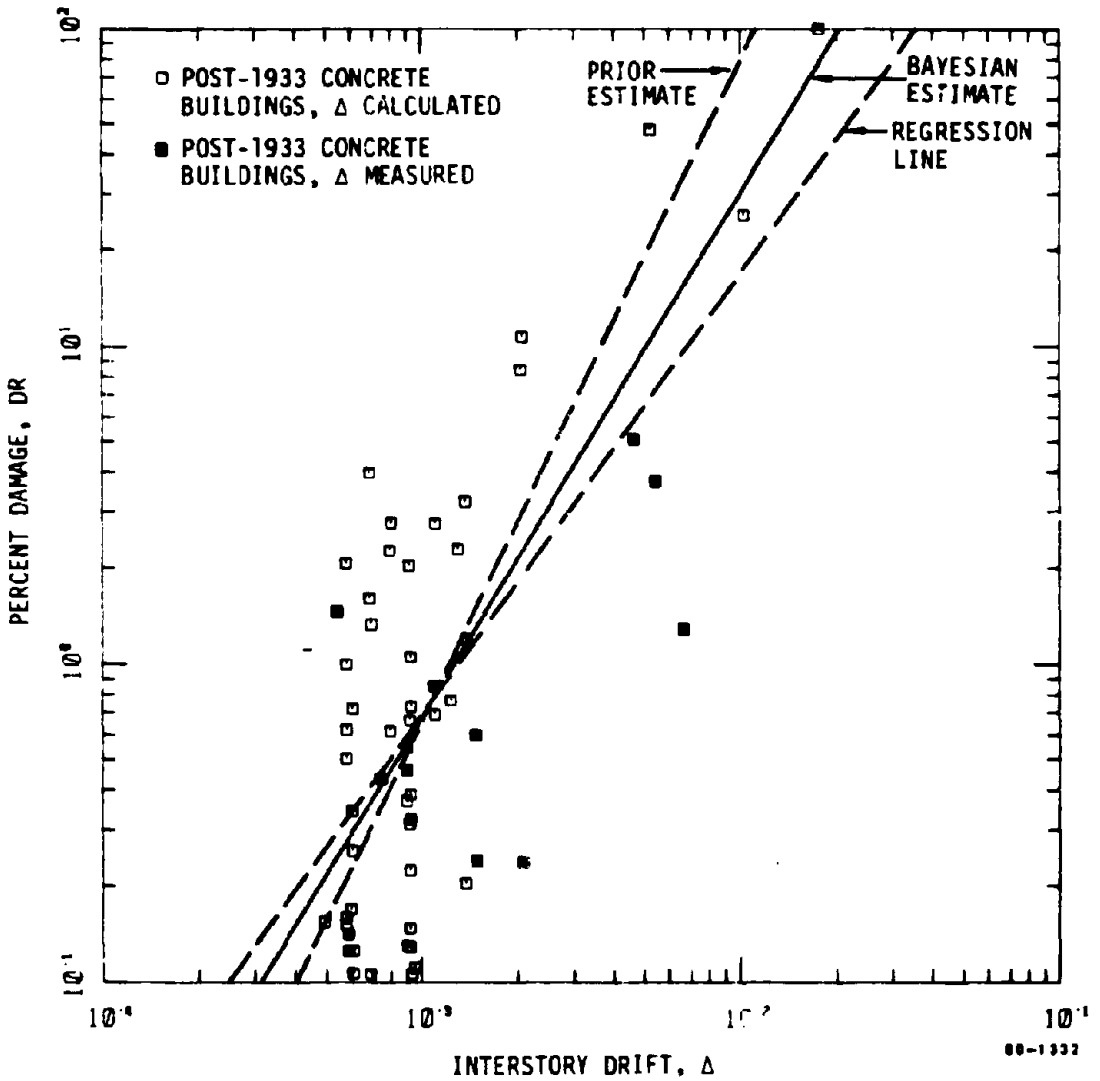



Figure 2-12. Damageability Model for Post-1933 Reinforced Concrete Buildings, 5-20 Stories, Bayesian Estimate

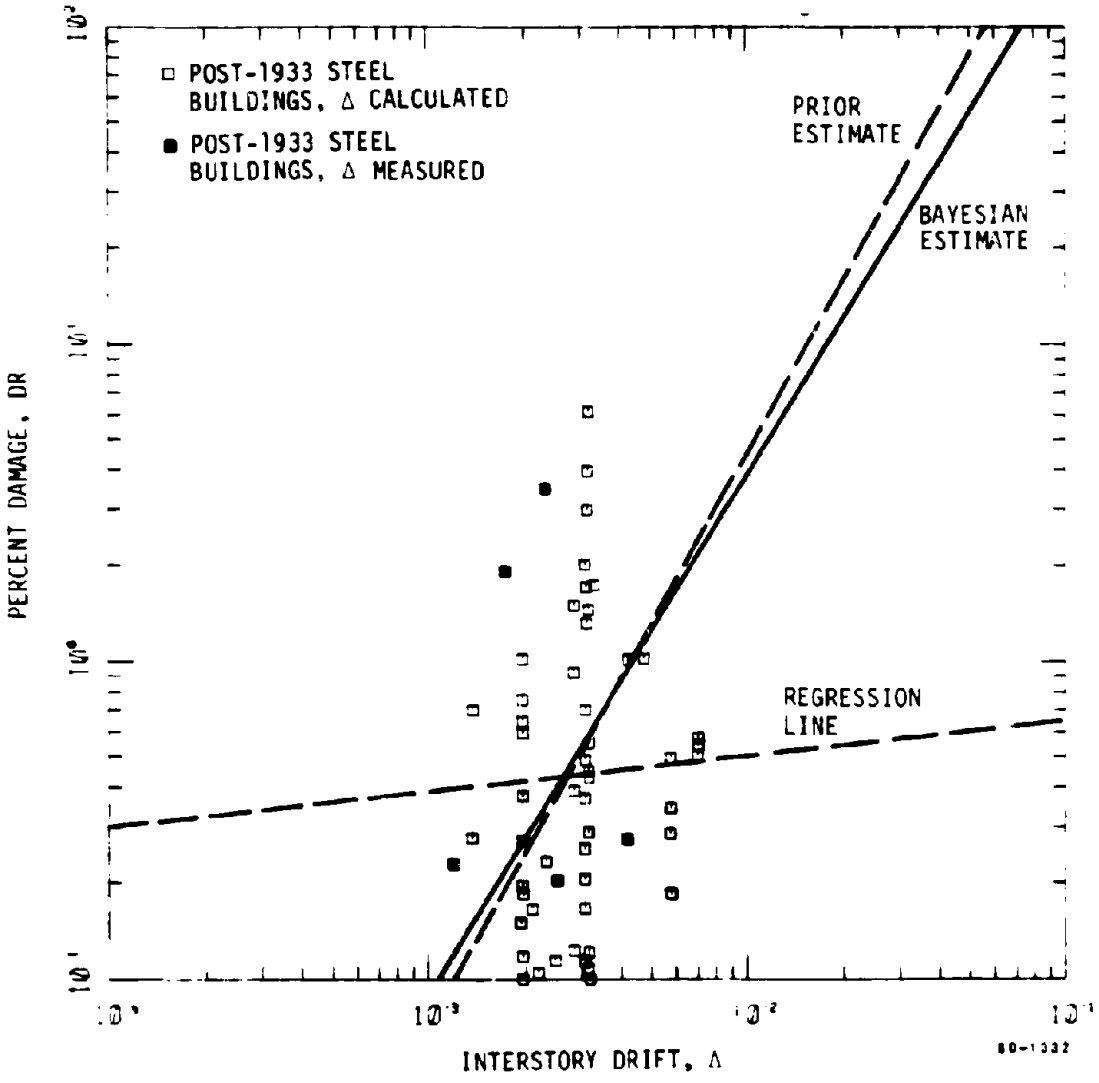



Figure 2-13. Damageability Model for Post-1933 Steel Frame Buildings, 5-20 Stories, Bayesian Estimate

Table 2-11. Summary of Bayesian Estimation Results

| TYPE OF BUILDING | DAMAGE PARAMETER | PRIOR ESTIMATE | BAYESIAN ESTIMATE | PRIOR C.O.V.(%) | BAYESIAN C.O.V.(%) | INCREASE IN CONFIDENCE (%) |
|---|---|---|---|---|---|---|
| CONCRETE | $\mu_f$ | 5.4 | 9.3 | 43 | 25 | 72 |
| CONCRETE | $C_2$ | 1.42 | 1.63 | 58 | 45 | 25 |
| STEEL | $\mu_f$ | 9.6 | 11.5 | 16.5 | 15.8 | 4 |
| STEEL | $C_2$ | .102 | 1.64 | 114 | 26 | 338 |

It is readily seen that a significant revision in $\mu_f$ for concrete was made resulting in a 72% increase in confidence. In the case of steel buildings, the standard error of the regression coefficient, $C_2$, was larger while that for $\mu_f$ was smaller than in the case of concrete buildings. The net result was a comparatively insignificant change in $\mu_f$ for steel with only a 4% increase in confidence. Notably, however, confidence in the estimated slope, $C_2$, relative to that obtained from the regression analysis, increased by 338%.

A number of important conclusions can be drawn at this time on the basis of the foregoing material. It is necessary to recognize them before going on to describe in detail the damageability models which have been developed. These conclusions provide the foundation for the assumptions which have been made in constructing the models.

(1) Available damage data from the 1971 San Fernando earthquake, particularly in the case of reinforced concrete structures, tend to validate the assumption of a linear relationship between log DR and MMI (Figure 2-7).

(2) A combination of theoretical and empirical analysis (Equations (2-1), (2-4) and (2-6)) has shown that a linear relationship between log DR and log $\Delta$ may also be expected (Figure 2-5).

(3) As anticipated, steel buildings were found to be less rigid (more flexible) than concrete buildings, tending to show higher levels of interstory drift for a given intensity of ground motion (Figures 2-5 and 2-6).

(4) Although steel buildings were found to experience more interstory drift than concrete buildings during the 1971 San Fernando Earthquake (incidentally, interstory drift is not a function of building height as shown in Equation (2-1) where

the ratio T/N is constant), they did not experience more damage, indicating that their greater flexibility is apparently compensated for in the design and installation of nonstructural items, to control total building damage.

(5) The concept of a damage threshold ductility, $\mu_t$, corresponding to some fraction of drift-to-yield, $\Delta_y$, above which structural damage is "not insignificant," appears to have practical merit. It provides the "missing link" required to relate the parameters $\Delta_y$ and $\mu_f$ to the linear damage model.

(6) Additional support for the foregoing conclusion is provided by the apparent linear relationship found to exist between log DR and log $\Delta$ in the case of wind damage (Figure 2-9). Here the threshold ductility, $\mu_t = 0.25$ for all high rise buildings on the average (both steel and concrete), was found to be close to the geometric mean of 0.27 computed separately from the values of $\mu_t = 0.19$ and 0.39, respectively, for concrete and steel buildings at MMI = VII.

(7) Finally, the dependence of damage on duration implied by Equation (2-22) tends to be confirmed by independent wind damage information which clearly indicates a slope of

$$C_2 = \frac{\log 50 - \log 0.5}{-\log \mu_t}$$

as defined by Equation (2-23b) and Equations (2-25a, b) where $\mu_c \equiv 1$.

## 2.5 Damageability Models for Total and Structural Damage

On the basis of these conclusions, it is proposed that the "judged" parameters, $\Delta_y$ and $\mu_f$, in conjunction with Equations (2-24) and (2-25) be adopted as the basis of damage models for both structural and nonstructural damage, the sum equaling total building damage. It will be assumed that $DR_c \equiv 50\%$ and $DR_t \equiv 0.5\%$ in either case, where the damage ratio is the percent of repair cost to replacement cost. In the case of structural damage, repair cost refers to the cost of repairing only structural damage, while replacement cost is that portion of the total building replacement cost attributable to the structure alone. In addition, it will be assumed that $\mu_t = 0.19$ in all cases except steel frame buildings where $\mu_t = 0.39$.

With this in mind, Tables 2-9 and 2-10 may be used to develop damage models for both earthquake and wind, for total building damage and stuctural damage, for three types of frame and three types of shear wall construction, for three quality ratings each. Geometric averages were computed for both $\Delta_y$ and $\mu_f$ based on judged values of the three individuals named in the tables. For three of the six types of building construction, namely steel frame, reinforced concrete frame (poured in place), and reinforced concrete shear wall (poured in place), the Bayesian estimates of $\mu_f^*$ were used. For the latter two categories of concrete buildings, ratios of $\mu_f^*/\mu_f$ representative of all reinforced concrete buildings were used in scaling values of $\mu_f$ applicable in each of the two specific categories. The results of this averaging and scaling are reflected in Table 2-12.

Damage "curves" were plotted from these parameter values for three values of $\mu_c$: $\mu_c = 1$, $\mu_c = \mu_f$, and the intermediate value corresponding to $C(T,M) = 0.327$ for concrete and $C(T,M) = 0.523$ for steel, assuming $T = 1.0$ sec. and $T = 1.6$ sec. respectively for 10 story buildings (on the

Table 2-12. Damageability Model Parameter Values by Type of Building and Quality Rating

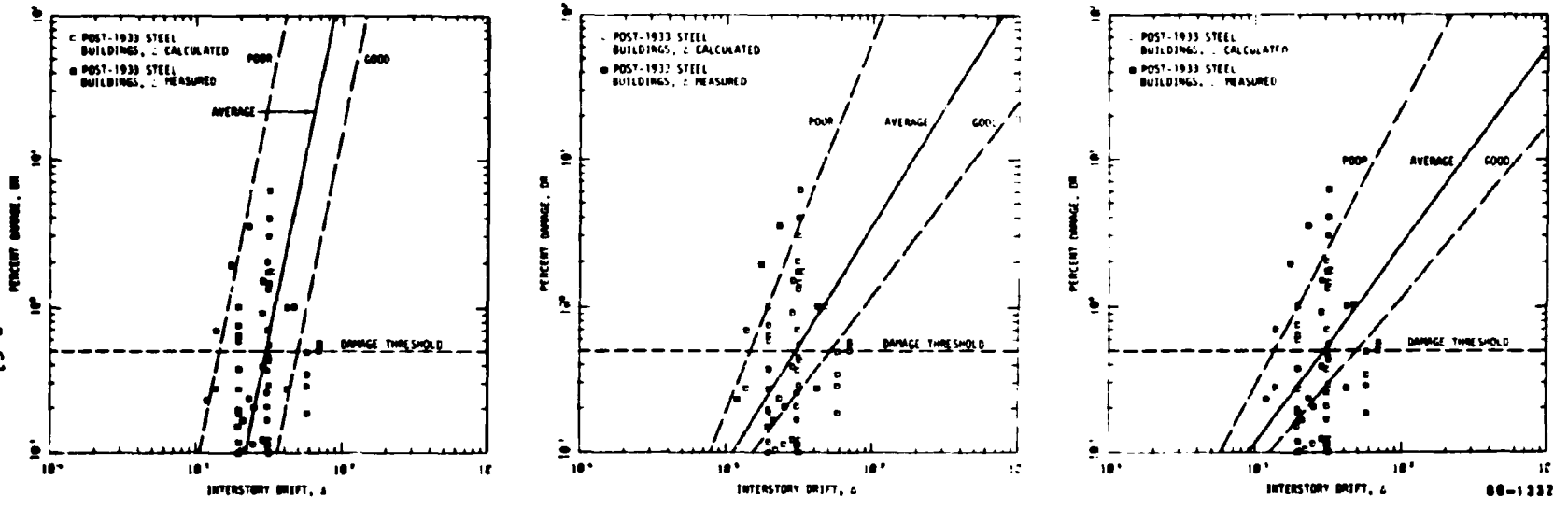
| STRUCTURAL SYSTEM | MATERIAL | GOOD | | AVERAGE | | POOR | |
|---|---|---|---|---|---|---|---|
| | | $\Delta_y$ | $\mu_f$ | $\Delta_y$ | $\mu_f$ | $\Delta_y$ | $\mu_f$ |
| FRAME | STRUCTURAL STEEL | .0125 | 21.0 | .0077 | 11.5 | .0036 | 4.4 |
| | CONCRETE POURED IN PLACE | .0084 | 17.2 | .0052 | 9.6 | .0029 | 4.3 |
| | PRECAST CONCRETE | .0049 | 5.1 | .0027 | 3.8 | .0013 | 2.5 |
| SHEAR WALL | CONCRETE POURED IN PLACE | .0045 | 10.3 | .0026 | 6.7 | .0017 | 3.4 |
| | PRECAST CONCRETE | .0029 | 3.2 | .0016 | 2.3 | .0010 | 1.6 |
| | MASONRY | .0059 | 6.9 | .0041 | 4.4 | .0021 | 2.0 |

average), and M = 6.5 (for the San Fernando earthquake). These curves are shown in Figures 2-14 through 2-19.*

The damage curves shown in Figures 2-14 through 2-19 are plotted over damage data points to help facilitate interpretation. It is pointed out, however, that only in the case of steel buildings are the building categories, by material and type of construction, exactly the same for both the damage curves and the data points. In the case of reinforced concrete buildings, for example, different damage curves are presented for four categories - reinforced concrete poured in place for both frame and shear wall construction, and reinforced concrete precast for both frame and shear wall construction. The damage data are not segregated in this manner; all reinforced concrete buildings are simply classified as "concrete." The great majority of the concrete buildings in this data base, however, are presumed to be reinforced concrete poured in place. Damage curves for the remaining category of reinforced brick masonry shear wall type buildings are also shown against the damage data for concrete buildings. The reason for this is that there were no brick masonry buildings constructed after 1933 for which damage greater than 0.1% was reported. In fact there were only a few brick masonry buildings listed (fifteen in all).

Particular note is made of the dashed lines labeled "Damage Threshold" appearing in each of the figures. They correspond to the value $DR_t$ = 0.5%. Careful examination of each set of three figures (a,b,c) will disclose that the lines corresponding to the same quality rating, but associated with different duration, pass through a common point. The curves were constructed this way for the following two reasons: (1) the wind damage curve of Figure 2-9 intersects the earthquake damage curve for the same class of buildings (all high-rise buildings) at this point,

*The curves are valid for the values of $\mu_c$ indicated, irrespective of number of stories. $\mu_c$ depends on T which generally depends on N, not vice versa.

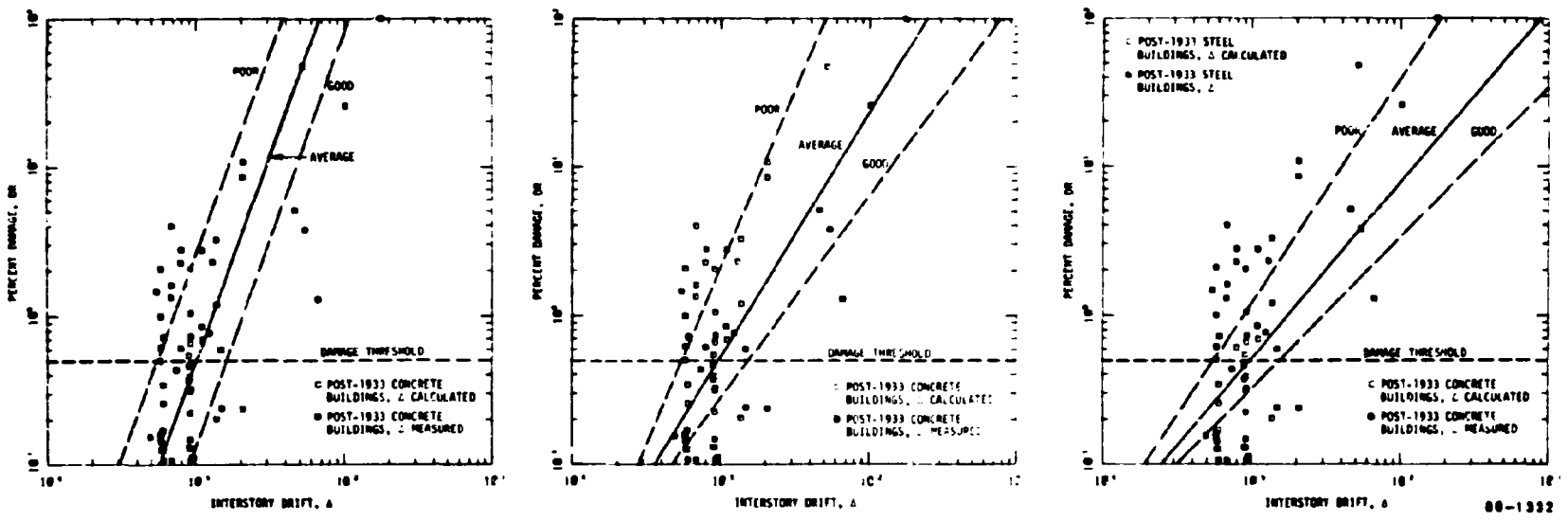



(a) LONG DURATION, $\mu_c=1$

(b) INTERMEDIATE DURATION, $\mu_c=0.537\ \mu_f$

(c) SHORT DURATION, $\mu_c=\mu_f$

Figure 2-14. Damageability Model for Steel Frame Structures Showing Dependence on Relative Duration and Quality Rating

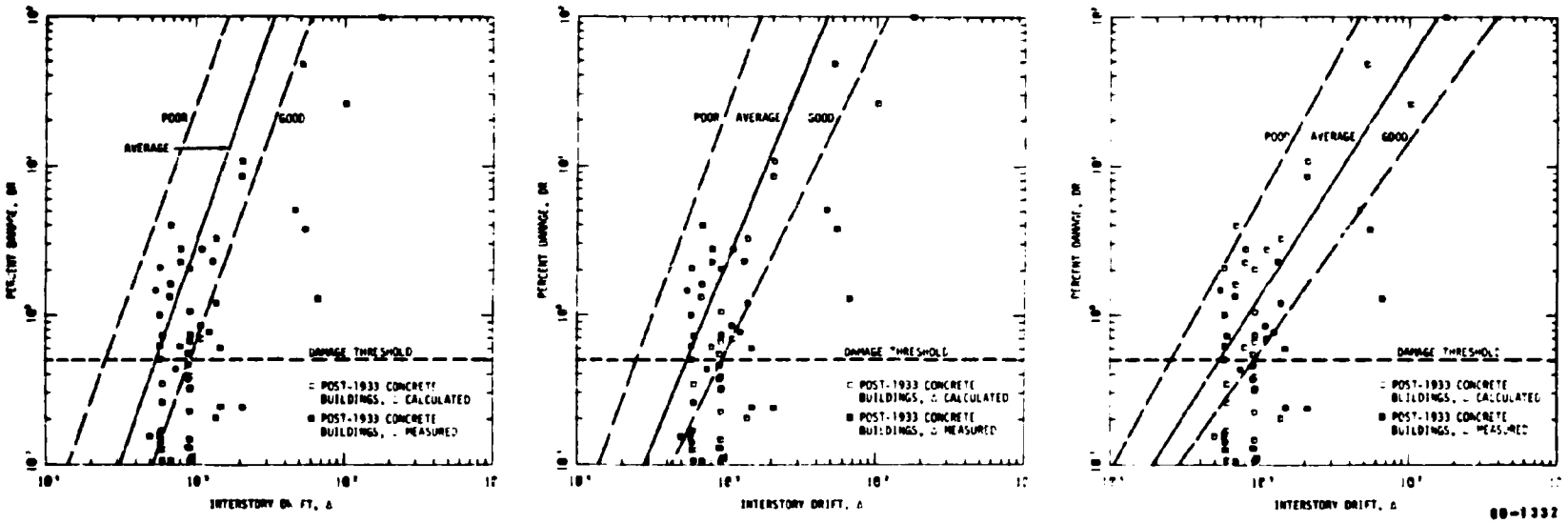



(a) LONG DURATION, $\mu_c=1$

(b) INTERMEDIATE DURATION, $\mu_c=0.327\ \mu_f$

(c) SHORT DURATION, $\mu_c=\mu_f$

Figure 2-15. Damageability Model for Reinforced Concrete Frame Structures Showing Dependence on Relative Duration and Quality Rating

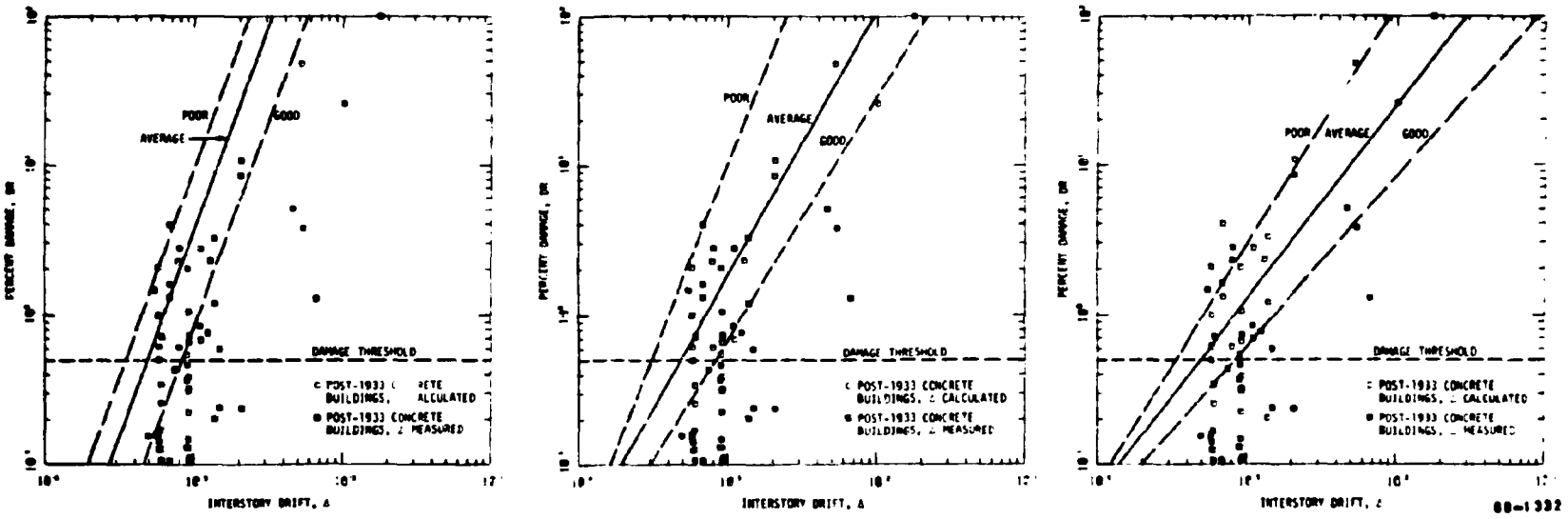



(a) LONG DURATION, $\mu_c=1$

(b) INTERMEDIATE DURATION, $\mu_c=0.327\ \mu_f$

(c) SHORT DURATION, $\mu_c=\mu_f$

Figure 2-16. Damageability Model for Precast Concrete Frame Structures Showing Dependence on Relative Duration and Quality Rating

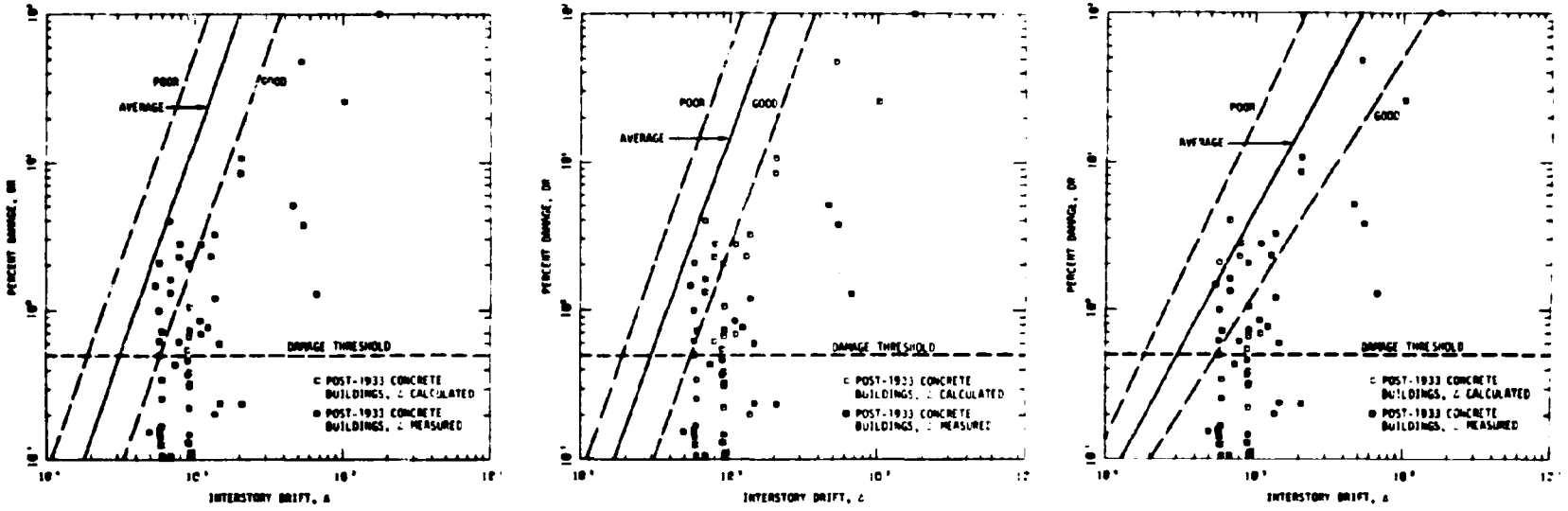



(a) LONG DURATION, $\mu_c = 1$

(b) INTERMEDIATE DURATION, $\mu_c = 0.327\ \mu_f$

(c) SHORT DURATION, $\mu_c = \mu_f$

Figure 2-17. Damageability Model for Reinforced Concrete Shearwall Showing Dependence on Relative Duration and Quality Rating

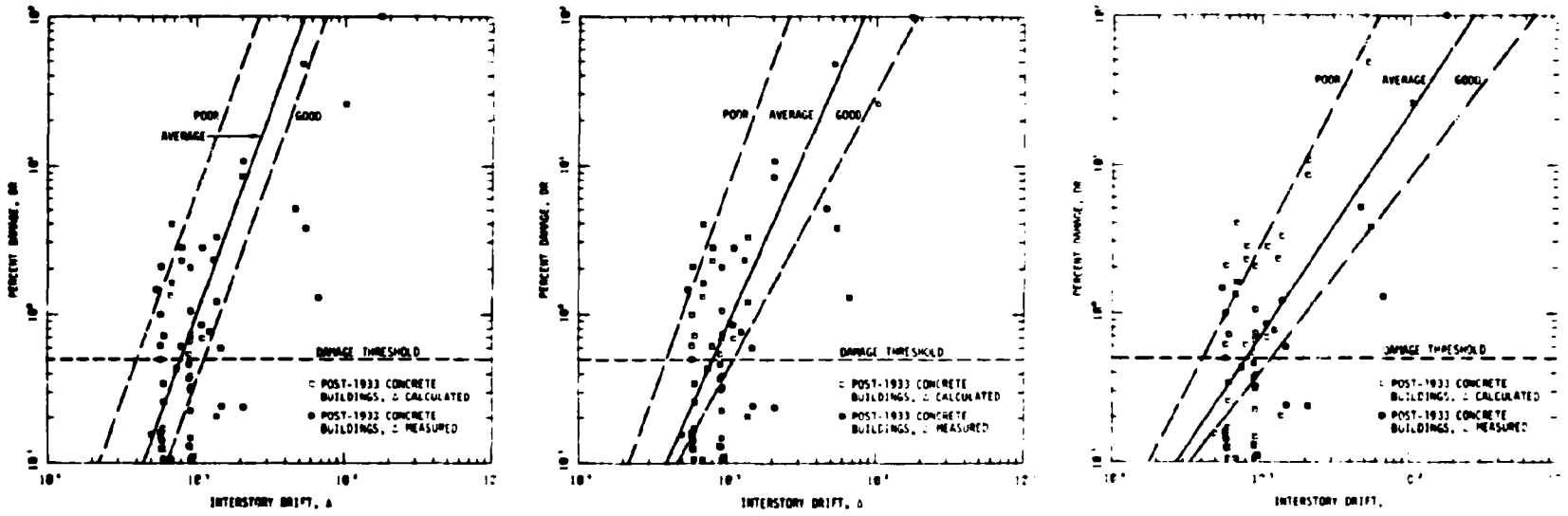



(a) LONG DURATION, $\mu_c=1$

(b) INTERMEDIATE DURATION, $\mu_c=0.327\ \mu_f$

(c) SHORT DURATION, $\mu_c=\mu_f$

Figure 2-18. Damageability Model for Precast Concrete Shearwall Showing Dependence on Relative Duration and Quality Rating




(a) LONG DURATION, $\mu_c=1$

(b) INTERMEDIATE DURATION, $\mu_c=0.327\ \mu_f$

(c) SHORT DURATION, $\mu_c=\mu_f$

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Figure 2-19. Damageability Model for Reinforced Masonry Shearwall Showing Dependence on Relative Duration and Quality Rating

and (2) total damages below this threshold value are not expected to be as sensitive to duration as damages above the threshold value; lower damages are presumed to reflect a more brittle failure mechanism. Since the present study has primarily addressed damages above the specified 0.5% threshold, and lesser damages are not considered to be as important, the proposed damage curves are only considered valid above this damage level. The 'DAMAGE' computer program prints out zero damage for anything less than 0.5%.

As it turns out, four of the pre-1933 brick masonry buildings listed did have damage ratios greater than 0.1%. All were in the MMI = VII intensity zone. Data points for these four buildings are shown by the triangular symbols in Figure 2-20. They appear to be consistent with the upper damage curve labeled "poor."

The data in Table 2-12 and the corresponding Figures 2-14 through 2-19 reflect professional judgment with regard to the effect of "quality" (in design and construction) on the damageability of a building. Quality, as defined in this context, is a subjective rating, although an attempt has been made to be as specific as possible. Reference [1] contains guidelines for the interpretation of the quality ratings. Those guidelines are reproduced in Table 7 of Volume II.

The guidelines of Table 7 (Volume II) may be difficult to apply to existing buildings to the extent that the specific qualities they address are not discernable. (Alternatively, the inspection procedure and point rating system proposed in [35] may be considered.) Since major changes in the building codes were instituted after the 1933 Long Beach earthquake, it is reasonable to rate at least some of the buildings constructed prior to that date as "poor." Clearly, there were some pre-1933 buildings not damaged in the 1971 San Fernando earthquake, which experienced the same intensity of ground motion as others suffering appreciable damage. Recalling that we have excluded buildings

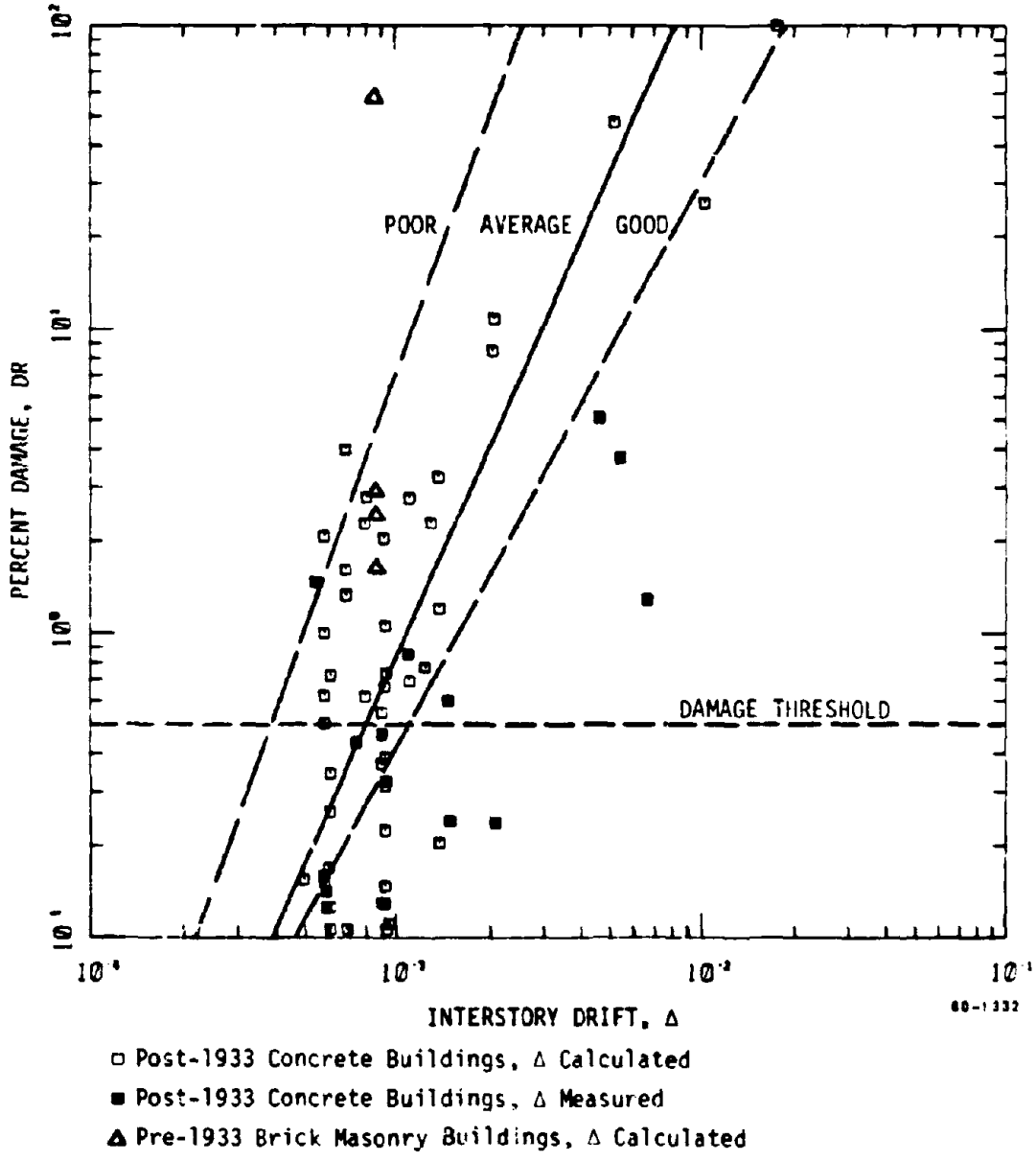



Figure 2-20. Validation of "Poor" Quality Damageability Model for Brick Masonry Buildings

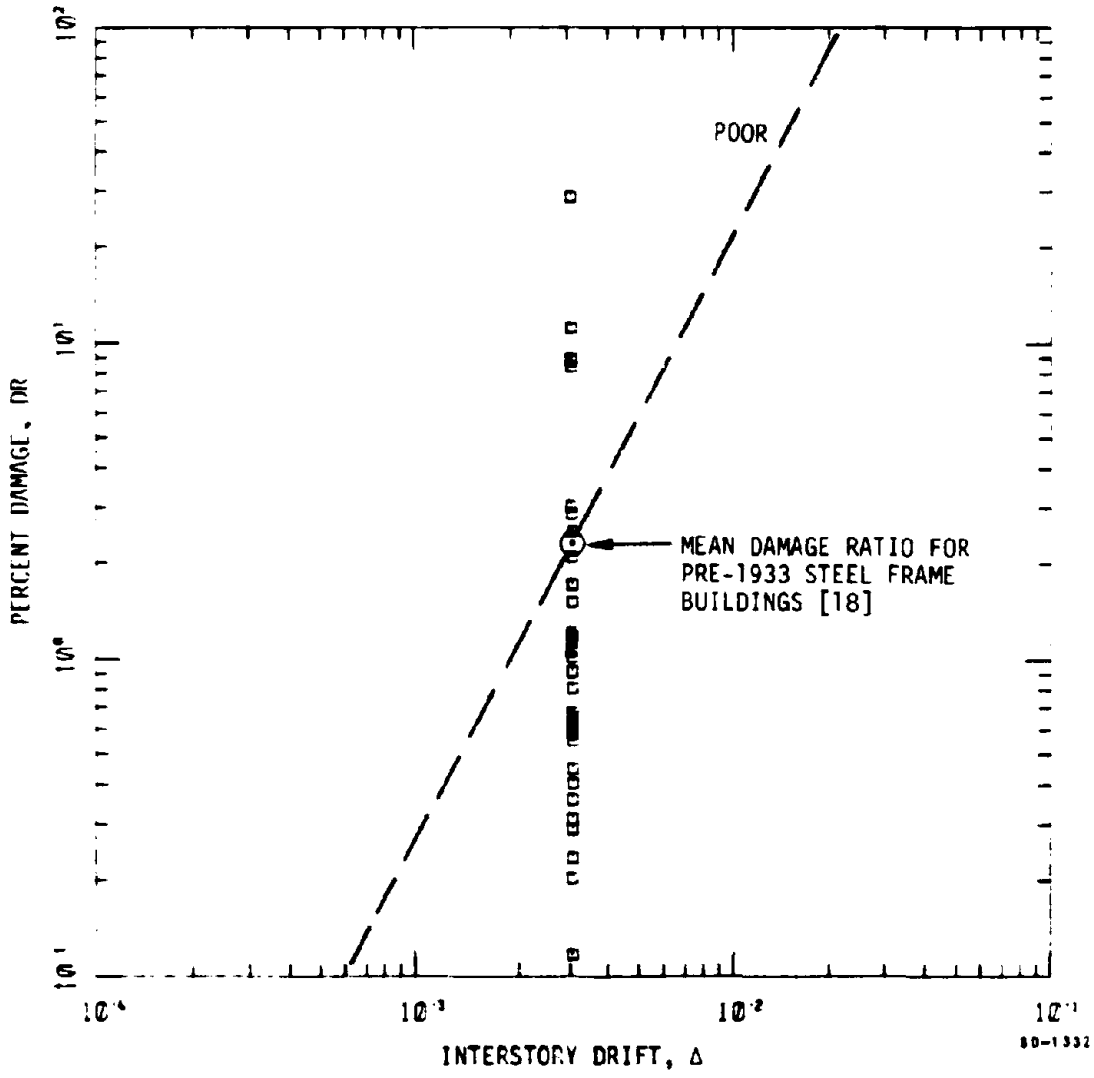
which have experienced less than 0.1% damage from the data base, we may take some comfort in the fact that these buildings are not included in the group labeled "poor." The degree of consistency observed between the simple models based on judged values, and actual damage data as have been interpreted here, suggests that the proposed trends are meaningful.

Two additional comparisons are offered in support of this contention. They are the damage data points associated with pre-1933 steel and reinforced concrete buildings which were in the MMI = VII intensity zone. These data points are shown in Figures 2-21 and 2-22 at the average interstory drift values of $\Delta = 0.0030$ for steel and $\Delta = 0.00085$ for concrete (discussed in Section 2.3). In the case of steel, the "poor" quality curve shown in Figure 2-14b has been replotted in Figure 2-21. In the case of concrete, the dashed curve for "poor" quality was obtained by averaging the judged values of $\Delta_y$ and $\mu_f$ in Tables 2-9 and 2-10, and scaling $\mu_f$ upward by the ratio 9.3/5.4, the ratio of Bayesian to prior estimates of $\mu_f$ for "average" quality, shown in Table 2-11.

On the basis of the discussion in the latter part of Section 2.2 concerning wind damage, wind damageability models are defined to be a special case of the earthquake damageability models, for total building damage and for structural damage. In particular this corresponds to setting $\mu_c \equiv 1$ in which case part (a) of Figures 2-14 through 2-19 represent wind damageability for the six types of buildings considered.*

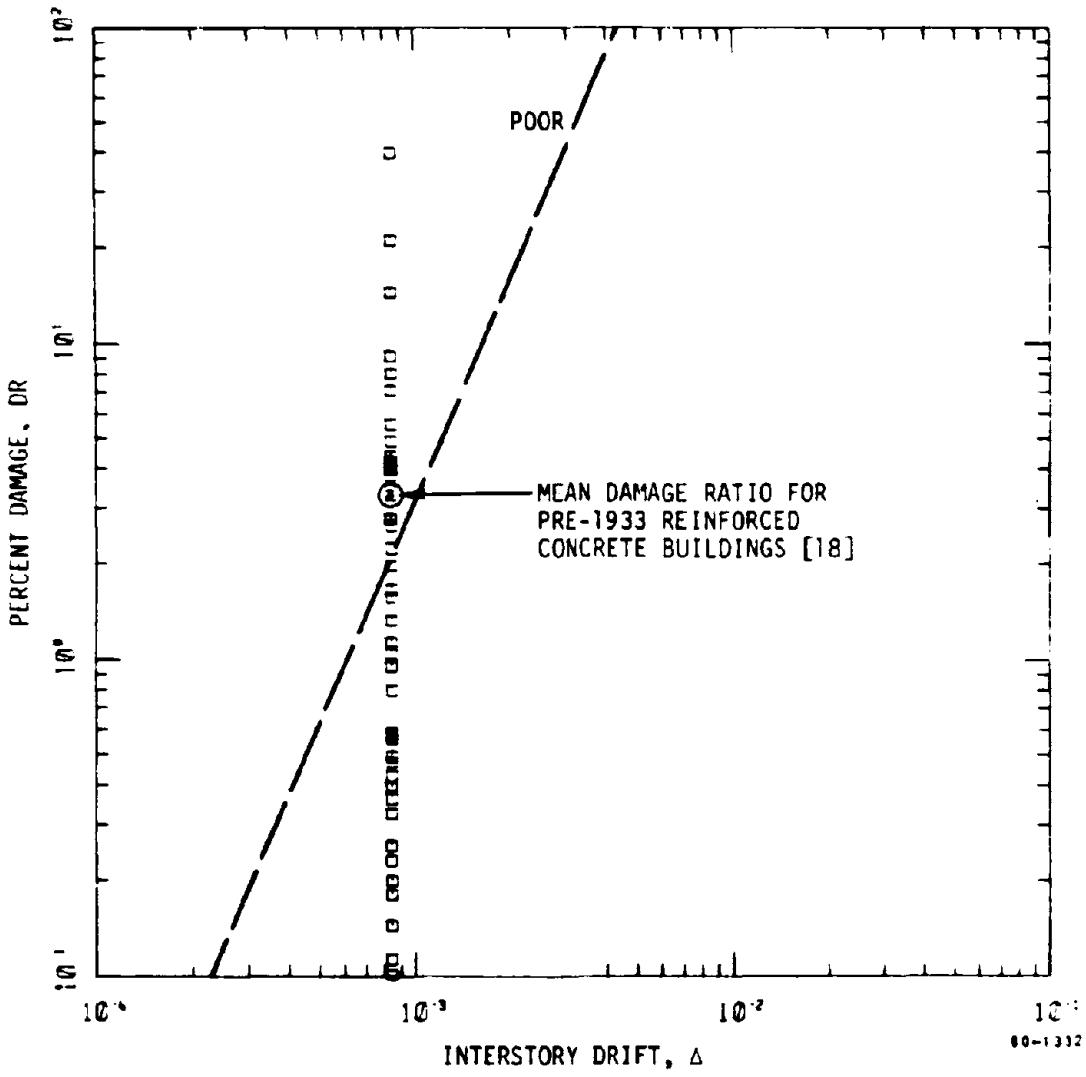
There is one final consideration which concerns the use and interpretation of the foregoing damageability models. That consideration is risk, or the probability of experiencing a particular level of damage (damage ratio). So far, we have confined the discussion of damageability to what is in essence a deterministic interpretation based on the median damage ratio, even though we have discussed the estimation of median

*Note conclusion (7) at the end of Section 2.4.




□ Pre-1933 Steel Frame Buildings, Δ Calculated

Figure 2-21. Validation of "Poor" Quality Damageability Model for Steel Frame Buildings




□ Pre-1933 Reinforced Concrete Buildings, Δ Calculated

Figure 2-22. Validation of "Poor" Quality Damageability Model for Reinforced Concrete Buildings

values within a statistical/probabilistic framework. This has been done for the following reasons: (1) clarity of presentation, (2) recognition of the fact that damageability, if not damage per se, can be meaningfully defined in a deterministic way, and (3) that using the present approach, a probabilistic interpretation can be based on the deterministic one, i.e., it can be treated as an add-on or a supplement to the deterministic interpretaton.

Consistent with the actual damage data used in this study, the regression analysis performed, and the lognormal probability distributions used to characterize the data (the Bayesian estimates of mean regression parameters notwithstanding), one may use the computed "standard error of the estimate" obtained from the regression analysis, along with the estimate of the mean to evaluate the parameters of the lognormal distribution. The following standard errors were computed for steel and concrete buildings:

$\sigma_{\log DR} = 0.458$ : steel

$\sigma_{\log DR} = 0.498$ : concrete

These standard errors may be interpreted as one-sigma bounds.

$$10^{-\sigma}(DR) < DR < 10^{\sigma}(DR)$$

The multiplicative factors $10^{\sigma}$ corresponding to the standard errors previously indicated for steel and concrete buildings are

$10^{\sigma} = 2.87$ : steel

$10^{\sigma} = 3.15$ : concrete

These two values suggest an average of about 3.0 for steel and concrete buildings combined. In the absence of specific data for each of the six building types, it is recommended that a one-sigma factor of three (3.0) be used as a general guide. This means that the upper and lower 95% confidence bounds will be a factor of $(3)^2 = 9$, or nearly an order of magnitude higher and lower. Again, it is cautioned that such an interpretation is only valid at the lower damage levels, certainly not above the median damage ratio of 10%.

Inasmuch as the values of interstory drift used in plotting the damage data points have been determined, for the most part, from the same basic structural response model as that used in assessing damageability, it is appropriate to assume, barring any bias or systematic error in the response predictions, that the recommended one-sigma uncertainty factor of 3.0 includes the random uncertainties due to structural modeling. This means that conditional probability statements can be made about the probability of exceeding or not exceeding a particular damage ratio, given the particular loading applied to the structure by an earthquake, wind or tornado, e.g., given a level of earthquake-induced ground motion, or a wind velocity.

Strictly speaking, the recommended one-sigma uncertainty factor of 3.0 was derived from data considered to represent "average" quality design construction, workmanship, etc., and therefore should be used only with the damage curve representative of "average" quality. Regression analyses have not been performed with damage data corresponding to buildings tentatively rated "poor." In fact, no individual building has been so rated, except to the extent suggested by the pre-1933 building data points shown in Figures 2-20 through 2-22. These figures suggest that if the "poor" quality classification is so defined with a lower damage limit of 0.1%, the corresponding one-sigma uncertainty factors are greater than 3.0. On the other hand, if only those damage data points which are higher than the (median) damage ratios suggested by

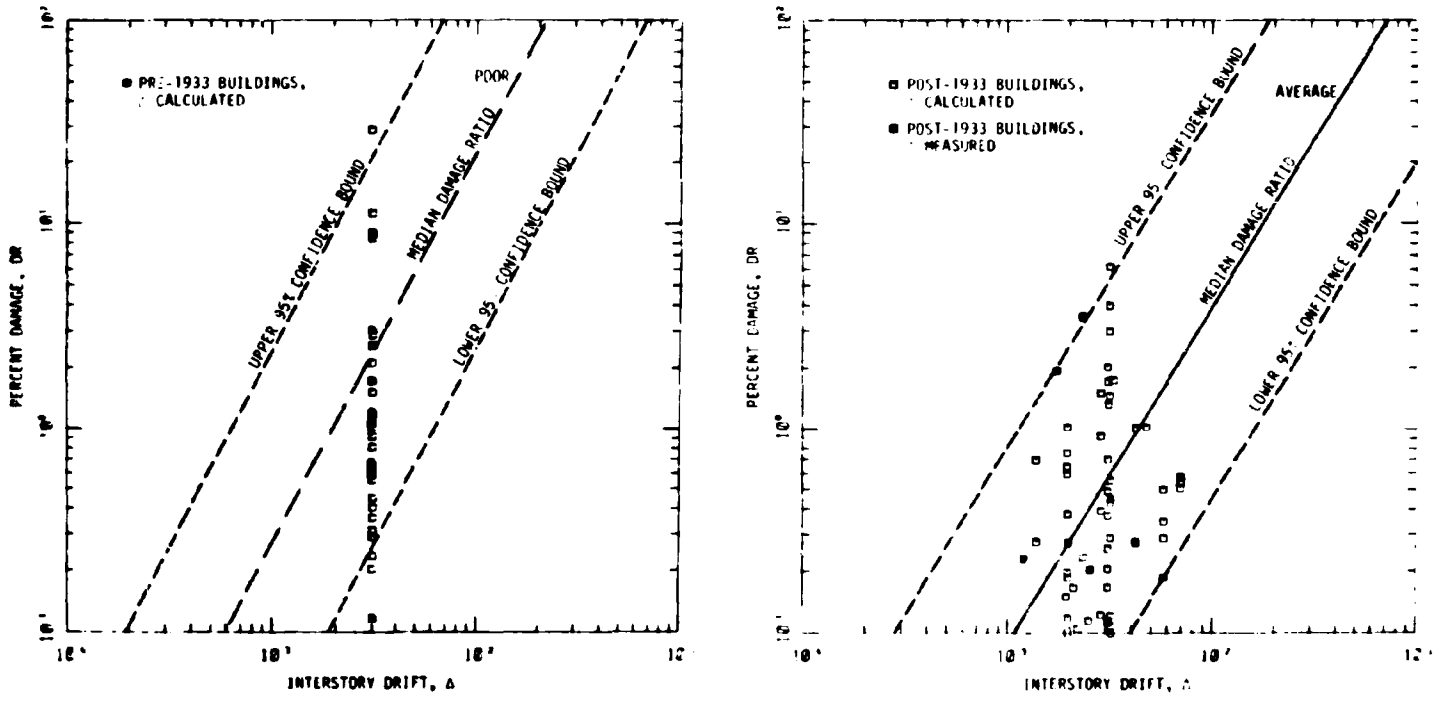
professional judgment are considered, and the "poor" quality curve is assumed to represent the "true" median, then a one-sigma uncertainty factor of approximately 3 would seem to apply once again. The lower limit of 0.1% is very arbitrary in the case of the "poor" quality rating, and may even be inappropriate if that lower limit is deemed appropriate for average buildings. It therefore does not seem unreasonable to adopt this value ($10^{\sigma}=3.0$), at least in a qualitative sense, for a probabilistic interpretation of damageability in the case of buildings rated "poor."

No specific consideration has been given to the probabilistic interpretation of damageability relative to "good" quality. This is a unique situation, compared with the other two, if only because it must include all of the data below the 0.1% damage ratio which have by definition been excluded from the other two. The assumption of a lognormal distribution is not considered to be appropriate in this case.

For purposes of illustration, the median damage ratio curves for steel and concrete buildings, along with the upper and lower 95% confidence bounds based on the one-sigma uncertainty factor of 3.0, for both the "average" and "poor" quality ratings, are shown relative to their respectively corresponding damage data points in Figures 2-23 and 2-24.

## 2.6 Damageability Models for Nonstructural Damage

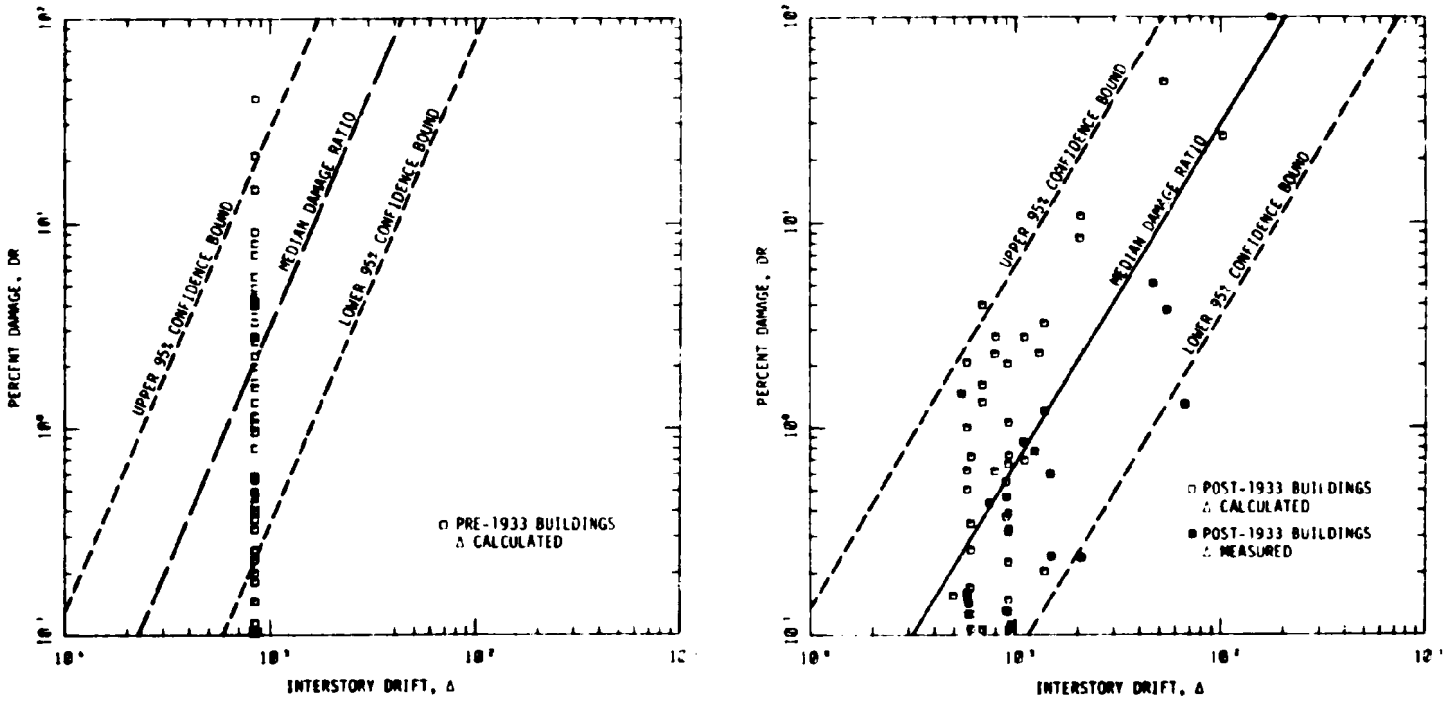
Nonstructural damage is considered in specific categories and is classified by nonstructural component. Two types of nonstructural components are considered for earthquake damage, and two for wind. They are as follows:




(a) "Poor" Quality Buildings

(b) "Average" Quality Buildings

Figure 2-23. Relationship of Proposed 95% Confidence Bounds to Damage Data Points on Steel Frame Buildings, 5-20 Stories




(a) "Poor" Quality Buildings

(b) "Average" Quality Buildings

Figure 2-24. Relationship of Proposed 95% Confidence Bounds to Damage Data Points on Reinforced Conrete Buildings, 5-20 Stories

Earthquake

(1) Nonstructural components sensitive to floor motion, e.g., floor mounted equipment, suspended ceilings, wall fixtures, etc.

(2) Glass

Wind (including tornado)

(1) Partitions and ceilings

(2) Glass

It is noted that one may evaluate total building damage, structural damage and nonstructural damage separately from the damageability models presented in Section 2.5, i.e., the models can be interpreted to represent either total damage, structural damage, or nonstructural damage in terms of repair cost divided by replacement cost. This is a direct result of the assumption (which the limited data seem to support) that structural and nonstructural damage on the average tend to occur in constant proportion to each other, above some damage threshold.

The following subsections discuss damageability models for the specific nonstructural categories named above.

### 2.6.1 Earthquake

The first category of nonstructural components considered under earthquake damageability is that of nonstructural components sensitive to floor motion. In this case, the model proposed in Reference [1] is adopted without modification. It is governed by the following equation:

$$\log DR = -4.62 + 0.552\ I \left(1 - \frac{Q - 3}{6}\right)$$

where

$$I = \text{Max}\ (I_A, I_V)$$

$$I_A = 10.28 + 3.50 \log A$$

$$I_V = 5.16 + 2.73 \log V$$

and where

$I$ = An equivalent Modified Mercalli Intensity (MMI) representing the intensity of floor motion at a given floor, computed by taking the greater of $I_A$ and $I_V$.

$I_A$ = A value of MMI determined by the maximum (peak) acceleration computed for a given floor.

$I_V$ = A value of MMI determined by the maximum (peak) velocity computed for a given floor.

$A$ = Peak floor acceleration

$V$ = Peak floor velocity

$Q$ = The quality factor assigned to nonstructural components within this particular category of nonstructural damage.

$$Q = \begin{cases} 3 : \text{good} \\ 2 : \text{average} \\ 1 : \text{poor} \end{cases}$$

Guidelines for establishing quality ratings are given in Table 8 of Volume II.

Floor acceleration and velocity are influenced by the higher modes of building vibration more so than floor displacement. Inasmuch as earth

quake loading is inherently dynamic (inertial), it is reasonable to expect this category of nonstructural damage to be more important.

Glass constitutes the second special category of nonstructural components damaged by earthquakes. Whereas Reference [1] proposed a model based on mullion clearance and interstory drift, available damage data [18] show no such correlation. For example, mullion clearances used in glass installations for steel buildings are understood to be the same as those used for concrete buildings, even though steel buildings in general experience more interstory drift than concrete buildings during earthquakes of the same intensity. Damage statistics in [18] show no significant difference in glass damage between steel and concrete buildings. In fact, the mean glass damage ratio for concrete buildings was found to be slightly higher than that for steel buildings during the San Fernando earthquake.

Again, the biggest problem with available damage data is that there are not enough buildings in MMI zones other than MMI = VII, so that trends in damage as a function of intensity are not evident. We therefore choose to rely in part on professional judgment as before. Table 2-13 was prepared from damageability estimates made by Donald F. Moran, one of the contributers to Tables 2-9 and 2-10. Listed are percent damages (percent of components damaged) vs interstory drift.

Glass is one of the component categories. Interestingly, Moran does not discriminate between different types of buildings, steel and concrete, for example. It is observed that the damage threshold value (corresponding to MMI - VII) of interstory drift is $\Delta_t$ = .0016, slightly greater than the value of $\Delta$ = .001 for which Moran estimates "zero" damage. Whitman, et al. [18] report that for post-1947 steel structures in the MMI = VII intensity zone, glass damage accounted for 3.5% of the total building damage on the average, while for post-1947 concrete structures in the same intensity zone glass damage accounted for 6.9% of

Table 2-13. Nonstructural Component Damage vs Interstory Drift - Percent of Components Which Suffer Damage*

| COMPONENT | INTERSTORY DRIFT BY QUALITY | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | GOOD | | | | AVERAGE | | | | POOR | | | |
| | .001 | .005 | .01 | .02 | .001 | .005 | .01 | .02 | .001 | .005 | .01 | .02 |
| CEILINGS | 0 | 20 | 30 | 100 | 0 | 20 | 40 | 100 | 10 | 50 | 100 | 100 |
| PARTITIONS | 0 | 10 | 20 | 100 | 0 | 20 | 80 | 100 | 10 | 50 | 100 | 100 |
| TRIM AND VENEER | NA | NA | NA | NA | 0 | 20 | 80 | 100 | 10 | 50 | 100 | 100 |
| GLASS | 0 | 20 | 50 | 100 | 0 | 30 | 60 | 100 | 0 | 40 | 80 | 100 |
| FILLER WALLS BETWEEN FRAMING MEMBERS | 0 | 20 | 50 | 100 | 10 | 30 | 80 | 100 | 10 | 50 | 100 | - |
| CURTAIN WALLS SET OUTSIDE OF FRAMING LINE | 0 | 20 | 40 | 100 | 10 | 20 | 60 | 100 | 10 | 40 | 80 | 100 |

*Estimates by Donald F. Moran

total building damage on the average. The corresponding standard deviations were 14.9% and 20.3% respectively, indicating that the difference between 3.5% and 6.9% is not statistically significant. Inasmuch as zero is undefined on a logrithmic scale, and Moran's estimates are clearly intended to be arithmetic, we would be justified in substituting a data-based value of damage at $\Delta = .0016$ for Moran's estimate of zero damage at $\Delta = .001$. To obtain such an estimate, we first of all average the statistical damage parameters for concrete and steel. Taking geometric averages we find

mean damage = 5%
standard deviation = 17.4%
coefficient of variation = 3.5

If we recognize that the average damage ratio for total building damage of steel and concrete buildings at MMI = VII was 0.5% and make the approximating assumption that glass accounts for 5% of the total building value in terms of replacement cost, we find that the damage ratio for glass damage, as a percent of the replacement cost of glass, is approximately 0.5% for "average" buildings. We may use the coefficient of variation of 3.5 to infer damage ratios for the quality ratings of "good" and "poor." We therefore propose the following damage ratios corresponding to an interstory drift of $\Delta = .0016$:

| | | | | |
|---|---|---|---|---|
| poor | : | 0.5% x 3.5 | = | 1.75% |
| average | : | 0.5% x 1.0 | = | 0.50% |
| good | : | 0.5% x 3.5 | = | 0.14% |

These data points in addition to those from Table 2-11 are plotted in Figure 2-25 and represent all buildings on the average.

It is clear that straight lines cannot be drawn in Figure 2-25 to connect each set of points for the three qualities, even in an approxi-

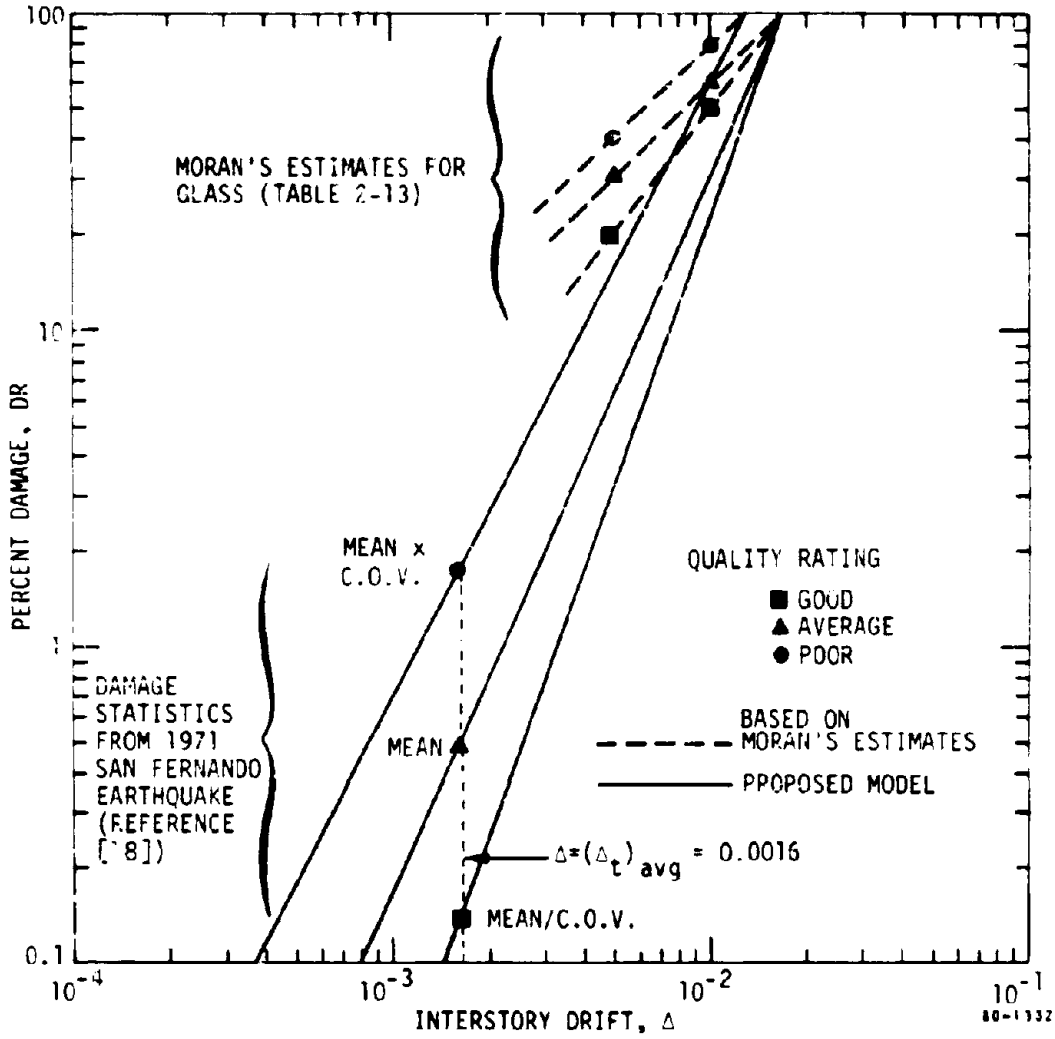
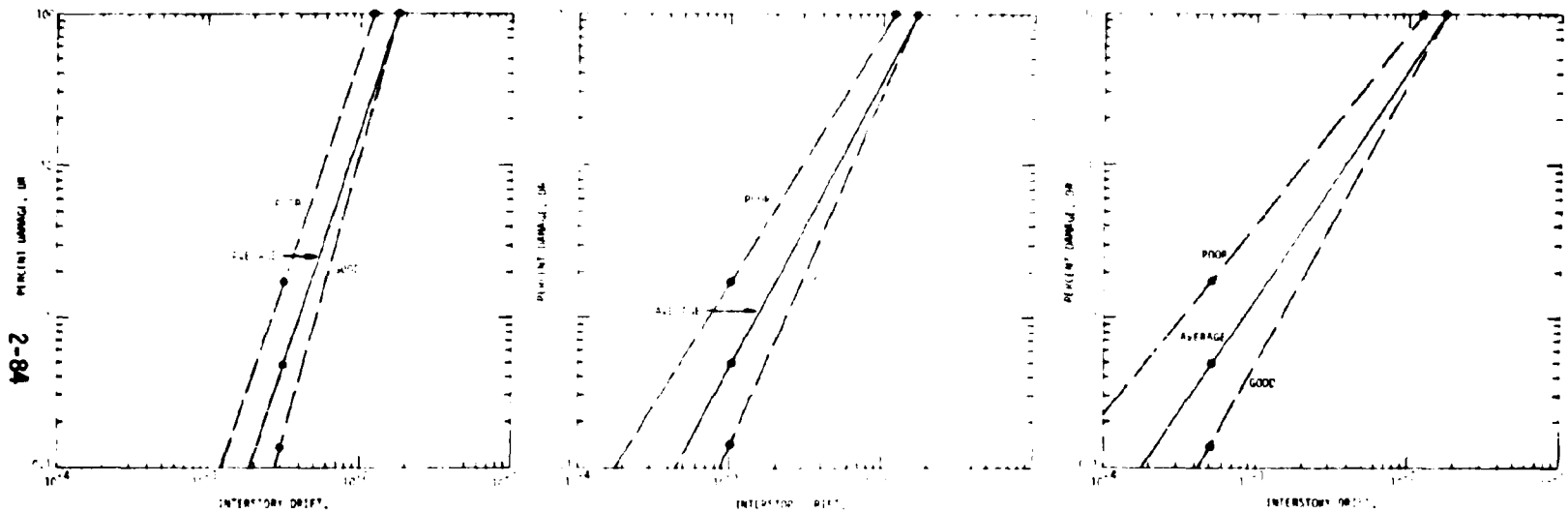



Figure 2-25. Earthquake Damageability Model for Glass - All Buildings

mate sense. On the other hand, when the points are plotted to linear scales, the lower three points for each quality come close to lying in a straight line. The greatest concern here, however, is the relatively high level (20-40%) of damage estimated at a drift of 0.005, in view of SEAOC's recommended allowable drift of 0.005 under earthquake loading [36]. One would expect a much lower glass damage level at allowable drift, since one of SEAOC's principal reasons in recommending this drift limitation was the "protection of health and safety of life" due to glass breakage. Nevertheless, Moran's estimate of 100% damage at an ultimate drift of 0.01 to 0.02 seemed plausible. As a compromise, therefore, between Moran's estimates and available damage data for the 1971 San Fernando earthquake, the straight lines in Figure 2-25 are proposed in principle.

There appears to be no good reason for using the same damage curves for all types of buildings. On the contrary, there is a reason why the same curves should not be used. As previously stated, steel buildings appear to suffer no more glass damage than other buildings, at least in low intensity earthquakes. Yet, if the same damage curves were used, greater damageability for steel buildings would be predicted because the steel buildings deflect more. To compensate for the different stiffnesses between steel and concrete frame buildings, and between moment resisting frame and shear wall buildings, the lower portion of the glass damage curves have been shifted to the right or to the left, depending on the specific ratios of drift-at-damage-threshold, $\Delta_t$, for steel, reinforced concrete frame and reinforced concrete shear wall buildings, to the average value of $\Delta_t = 0.0016$. The resulting curves are plotted in Figure 2-26. The 100% damage points have not been shifted because breakage ultimately depends on interstory drift, and mullion clearances appear to be standard among different types of buildings.

It is intended that all reinforced concrete frame buildings and all reinforced concrete or masonry shear wall buildings each use the same




(a) Steel Frame Buildings

(b) Reinforced Concrete Frame Buildings

(c) Reinforced Concrete and Masonry Shearwall Buildings

Figure 2-26. Earthquake Damageability Models for Glass by Type of Structure and Quality Rating

set of glass damage curves applicable to frame or shear wall, respectively.

### 2.6.2 Wind

The first category of nonstructural damage considered includes nonstructural components sensitive to interstory drift, particularly ceilings and partitions. Since no wind damage data were available for this study, professional judgment was relied upon. It was recognized that nonstructural components in this category tend to have a brittle-type of failure mode; once a critical value of interstory drift is exceeded, the component is damaged, e.g., cracked, separated from its attachments, etc. It was reasoned that it should make little difference whether the interstory drift is caused by earthquake or wind, and therefore, the estimates given by Moran in Table 2-13 should be applicable. In this case, however, rather than attempt to treat each of the nonstructural components separately, it was decided to average their damage ratios at each specified level of interstory drift. Since logrithmic relationships and geometric averages have thus far been used for all other damages sensitive to interstory drift, it seemed appropriate to do so here. Again, however, as in the case of glass damage due to earthquake, zero damage is undefined. Since damage sensitive to interstory drift is not expected to differ appreciably from earthquake and wind loading, the M.I.T. earthquake damage statistics for damage to ceilings and partitions are considered relevant.

Damage statistics reported by Whitman, et al. [18] for post-1947 buildings in the MMI = VII zone are given in percentages of total building damage as follows:

| BUILDING | COMPONENT | MEAN (%) | STANDARD DEVIATION (%) |
|---|---|---|---|
| STEEL | CEILING | 5.8 | 9.4 |
| STEEL | PARTITION | 23.3 | 30.9 |
| CONCRETE | CEILING | 3.7 | 8.2 |
| CONCRETE | PARTITION | 16.3 | 27.5 |

Again the difference in reported damages between steel and concrete buildings does not appear to be statistically significant. Therefore average mean percent damages to ceilings and partitions may be taken as 5% and 20% respectively. To translate this into a damage ratio relative to replacement cost, we must again assume a proportionate value of ceilings and partitions relative to the replacement cost of a building. If this fraction is 5% of building replacement cost, the total of 25% damage results in a damage ratio of 2.5% of the replacement cost of ceilings and partitions. This percentage is considered to apply to nonstructural components in the "average" quality category.

In the case of "good" quality, there appears to be little basis for establishing the low-damage portion of the curve other than to lower the "average" damage point at $\Delta_t$ by one standard deviation. Damage curves for the three quality ratings are shown in Figure 2-27. The two sets of points (Moran's estimates and earthquake damage statistics) are seen to line up reasonably well in this case. Separate curves for steel, concrete frame and concrete shear wall construction, shifted at the lower end by ratios of specific to average drift-at-damage-threshold, are shown in Figure 2-28. The curve labeled "poor" is not shifted since the $\Delta = 0.001$ damage point was specified to be non-zero and "poor" quality installations in more flexible buildings may indeed result in greater damage.

The wind damageability model for glass is different from that of earthquake, because of the overriding effect of wind pressure forces which

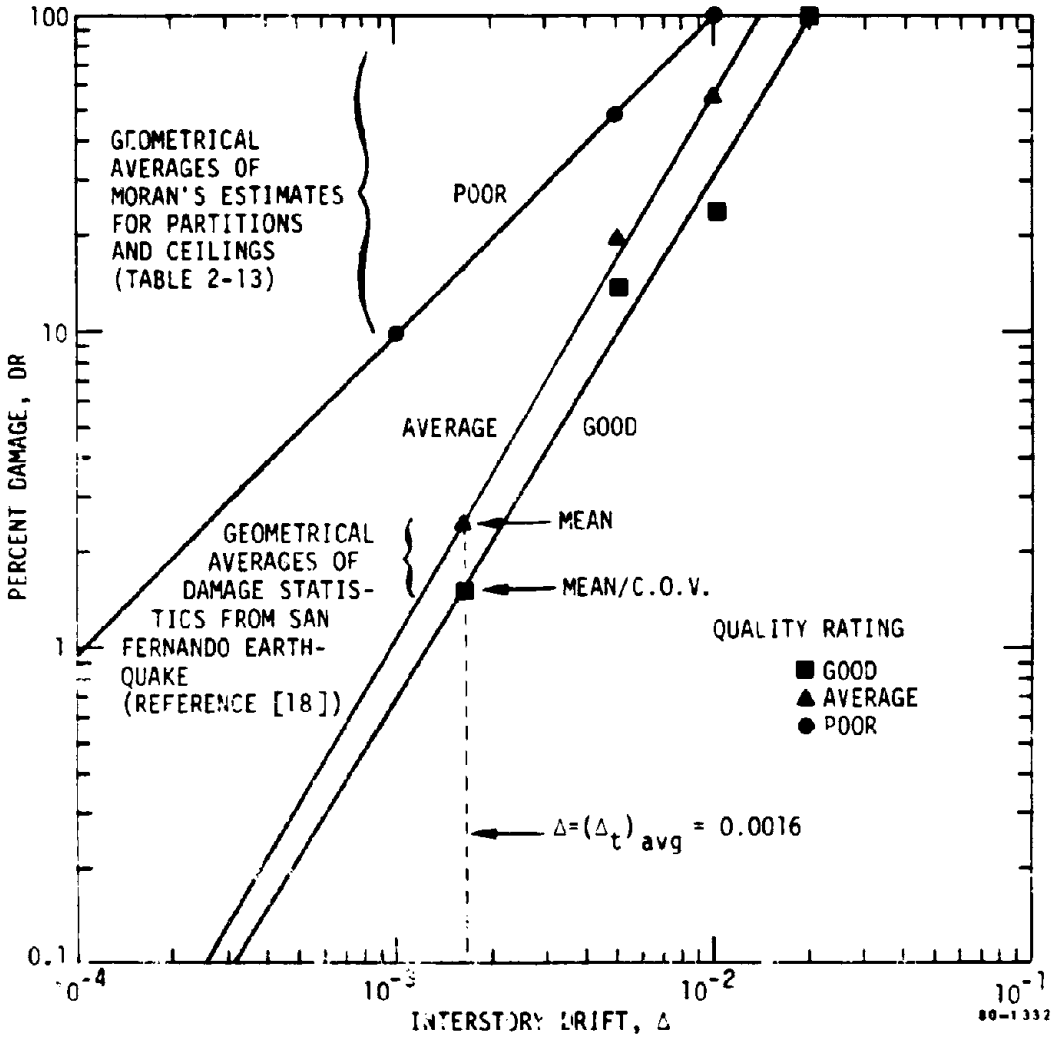
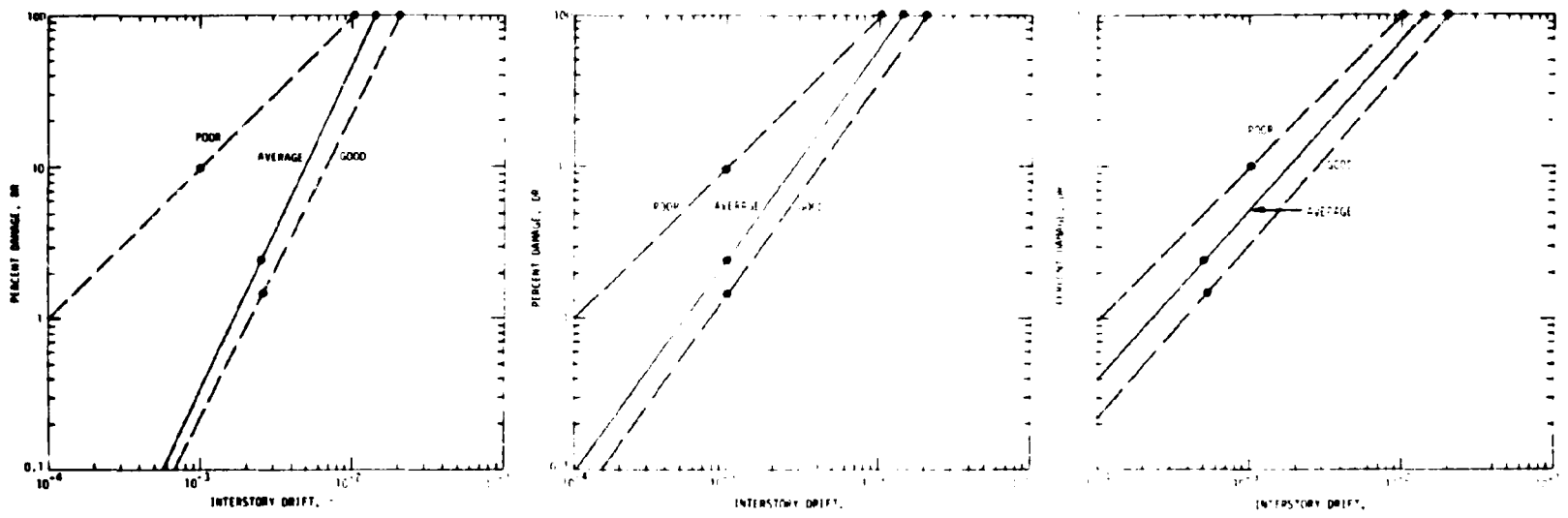



Figure. 2-27. Damageability Model for Partitions and Ceilings - All Buildings




(a) Steel Frame Buildings

(b) Reinforced Concrete Frame Buildings

(c) Reinforced Concrete and Masonry Shearwall Buildings

80-1332

Figure 2-28. Wind Damageability Models for Partitions and Ceilings by Type of Structure and Quality Rating

act directly on the windows. The model proposed in Reference [1] has been adopted here. It is based on statistical glass damage data which exhibit a normal distribution about the critical breaking pressure.

In particular the pressure required to break a window in flexure is given as

$$p_o = \text{breakage pressure} = \frac{\sigma_G t^2 \left[1 + 1.61 \left(a^3/c^3\right)\right]}{0.75a^2}$$

where

$\sigma_G$ = mean breaking stress of glass ($lb/in^2$)

a = minimum glass dimension, either h or b (in)

c = maximum glass dimension, either h or b (in)

b = average window width (in)

h = average window height (in)

t = glass thickness (in)

The fifty percent damage level is at a window pressure equal to $p_o$. The coefficient of variation or dispersion coefficient, $C_{VG}$, is chosen to be 0.25.

The equation for the percent window breakage at any given building floor is:

[Percent of Windows Broken] = DR =

$$\frac{100}{2\pi\,(p_o)\,(C_{vG})} \int_0^p \exp\left\{-\,\frac{1}{2}\left[\frac{Z - (p_o)}{(p_o)\,(C_{vG})}\right]^2\right\} dZ$$

where

$p$ = wind pressure ($lbs/in^2$)

In calculating ($p_o$) the average values of the glass parameters for that floor are used. The percentage of corner windows damaged is estimated using corner pressure values and the percentage of non-corner window damage using the corresponding pressure values.

## 3. SUMMARY OF COMPUTER PROGRAM 'DAMAGE'

This section provides a general overview of the damageability assessment procedure used in the present computer program. Figure 3-1 shows a schematic of the program data deck. The input is organized such that most of the data are input in data blocks consistent with the computational organization of the program. Figure 3-2 shows a macro flowchart of the DAMAGE computer program.

The circled numbers noted in Figure 3-2 denote, for reference, the basic modules of the program. The building's site location, and the desired program options are read in as input to the program in ① . The user has the option of selecting any combination of hazard loads he desires. The hazards which are addressed in the program are earthquake, wind and tornado. The user must also select, at this point, the structural modeling option that will be exercised in the program. Three options are provided: Detailed Frame model; Story Stiffness model; and an Empirical model. The Detailed Frame model generates a stiffness matrix and computes modal deflections from a detailed multi-degree-of-freedom structural model incorporating beam and column framing elements, rigid diaphragms and concrete or masonry shear walls. The Story Stiffness model generates the same information as the Detailed Frame model but from user input story stiffness data. The Empirical model simply computes modal deflections from a linear mode shape model and a user input fundamental period. These three options provide enough program flexibility to allow the user to select the level of detail necessary to his problem.

Depending upon which hazards are considered in the analysis, hazard input loads are generated in ② . For earthquake, a Richter magnitude and effective hypocentral distance must be provided. In addition, the user has the option of specifying peak ground acceleration, velocity and displacement which overrides those computed automatically from Richter

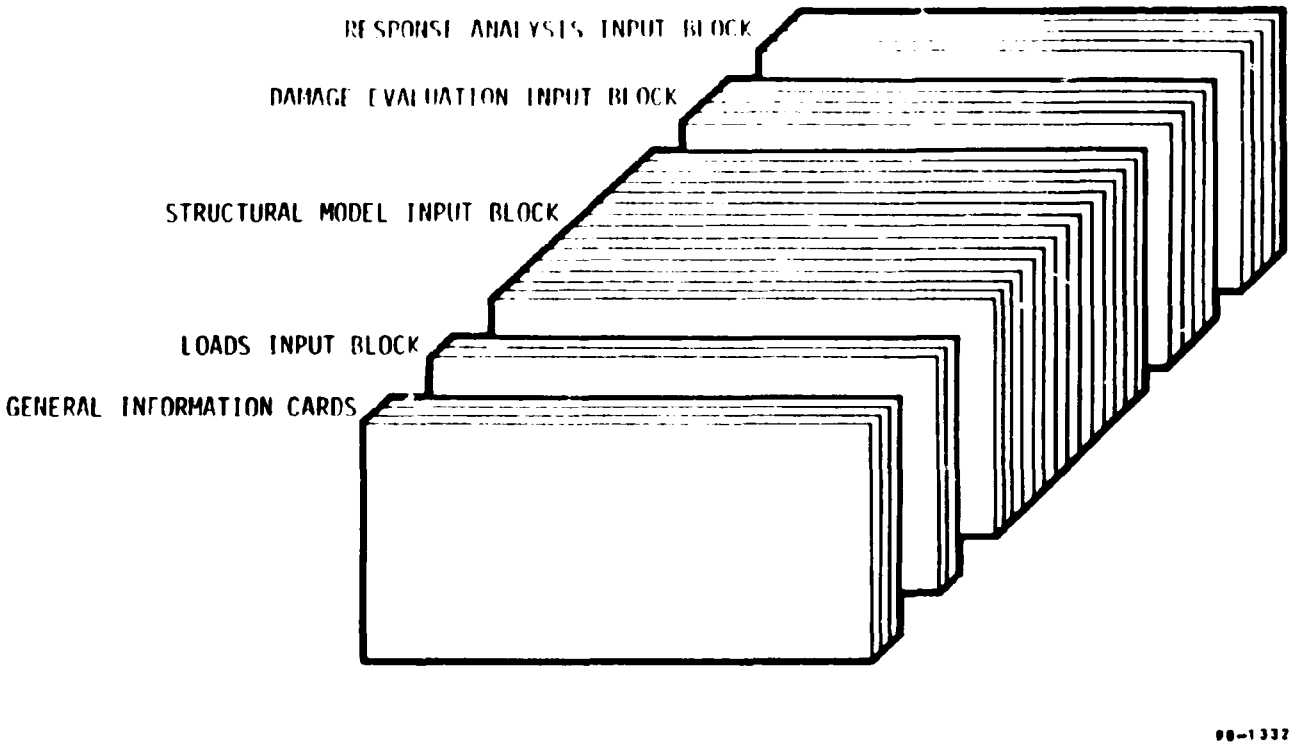



Figure 3-1. DAMAGE Program Data Deck Set-up

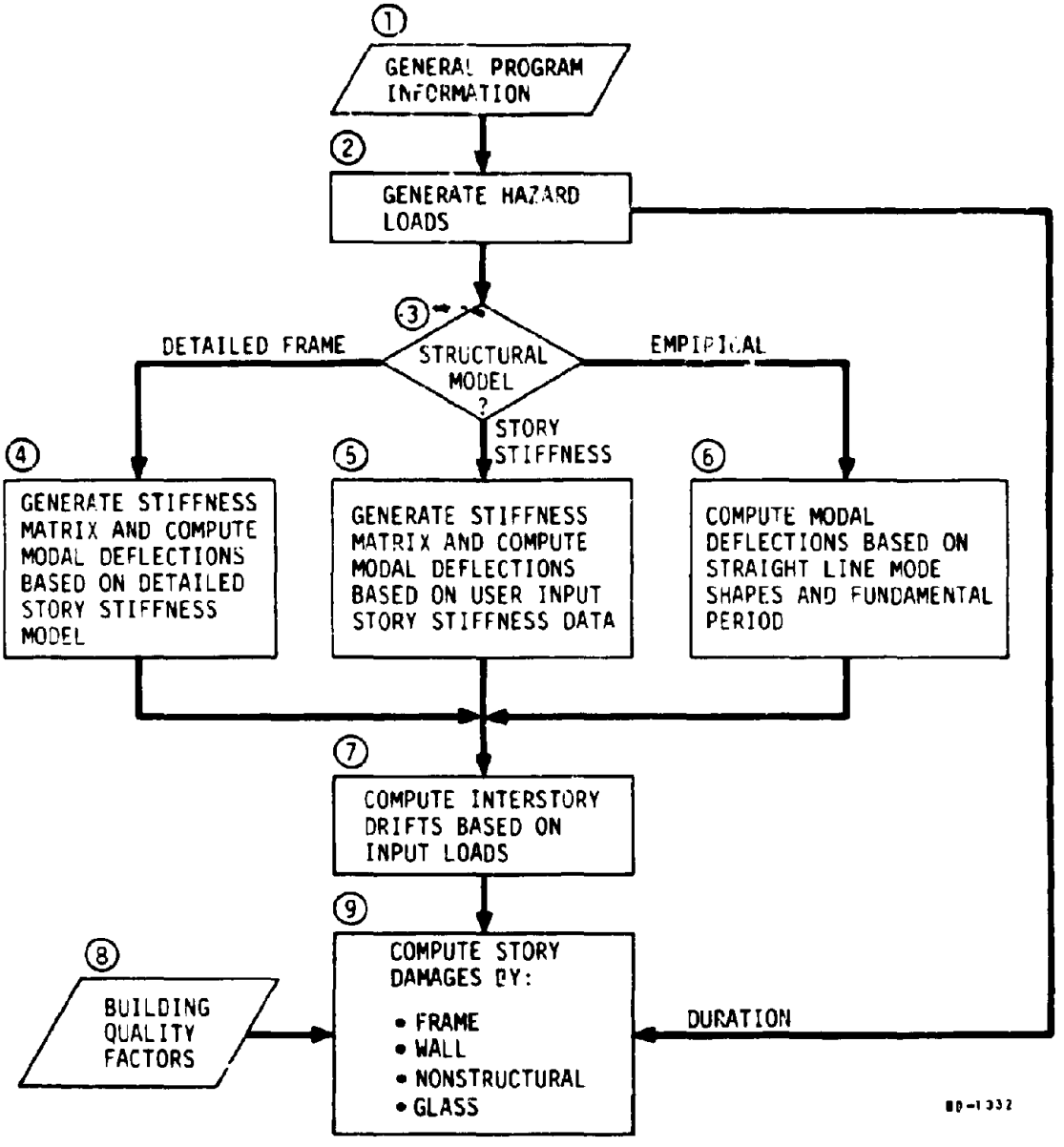



Figure 3-2. Macro Flowchart of DAMAGE Computer Program.

magnitude and distance. For wind, a code describing the terrain surrounding the site, and an estimate of the fastest-mile wind velocity which can be expected to effect the site within some desired exposure period must be input. For tornado, the expected number of tornados per year for the site must be determined.

Once these loads are generated, we then check in ③ to see which structural modeling option is to be exercised. If the Detailed Frame model in ④ is selected, the stiffness matrix and modal deflections are generated using a detailed finite element model. The stiffness matrix is constructed frame by frame, story by story, from top to bottom. Stiffness and mass contributions from each frame are superimposed in formulating a two-dimensional model of the building. The user may choose between two frame modeling options: steel frame and general frame. The general frame model is intended to be used primarily for concrete frame and shear wall type structures, although it could be used for steel frame structures as well. If the story stiffness model in ⑤ is selected, the stiffness matrix and modal deflections are generated based on user input story stiffness data, e.g., story stiffness, floor heights and floor weights. If the Empirical model in ⑥ is selected, modal deflections are simply computed using a linear mode shape model and a user input fundamental period for the building.

Depending upon the type of response analysis desired (i.e., dynamic for earthquake and static for wind), the computer program then computes interstory drifts in ⑦ based on the input hazard loads. For earthquake, if the computed response indicates that the ductility is greater than one, i.e., computed interstory drift exceeds the specified "drift-to-yield," then the response of the structure is altered so that the effects of yielding are reflected. For wind, an estimate of building damping and shape, wind direction and ratio of open surface area to solid area is required to evaluate the static response of the structure to wind loads.

Once the interstory drifts are computed for the subject building, damageability is evaluated. Damageability is determined in three parts:

I. Total damage to the building as a percent of estimated repair cost to replacement cost of the building. This damage category includes structural as well as nonstructural damage.

II. Damage to structural components. Damage to lateral load carrying components including frames and shear walls is estimated as a percent of their repair cost to replacement cost.

III. Damage to selected nonstructural components. Two primary categories of nonstructural damage are considered for both earthquake and wind. Glass damage is one of the categories in each case. For earthquake, the second category is damage to nonstructural components which are sensitive to floor motion. For wind, the second category includes partitions and ceilings which are sensitive to interstory drift.

In ⑧ Building quality factors reflecting the relative strengths, physical condition, integrity, and workmanship of the various systems are determined to help select the appropriate damage model to be used in ⑨ . The relationships allow the estimation of damages on a floor by floor level, and a single estimate of total building damage.

The reader is referred to [1] for details of the methodology used in developing the original program, which has been modified in producing 'DAMAGE.' The modifications fall into one of two general categories:

(1) Modification to the load generation and structural response segments of the program to simplify input requirements. Many infrequently used options have been eliminated by selecting

them internally. The selected options are noted in the print-out so that the user may relate his results to the original methodology. This is not essential to understanding the results, however, so that the user may ignore the notes if he so chooses.

(2) Replacement of the damage modeling subroutine. The new damageability models described in Section 2 replace those contained in the original program, except as specifically noted in Section 2. However, the new models use the same structural response information as the old ones did, so that the interface between the response segment of the program and the damage segment remains unchanged. The new models are completely documented in this report.

In all but the most unusual cases, it is anticipated that the present documentation will be sufficient to use and understand the program. The Users Manual has been completely rewritten in a format intended for use by personnel without computer experience.

# 4. USER TESTING

## 4.1 Test Plan

The objectives of the user test program were threefold: (1) to test the documentation of the DAMAGE computer program by introducing it to new individuals who must use the program, (2) to test the operational and computational aspects of the program by having it applied to practical problems, and (3) to obtain feedback from these individuals on how to improve the program and its documentation.

Mr. Marvin Hopewell of the Department of Planning and Building for the City of Long Beach, California was the program tester. His special assignments with the Department have included the inspection, rating and obtainment of the repair of all Type I, II and III buildings constructed prior to the adoption of seismic lateral force design requirements.

As part of the test plan, two actual buildings were to be selected as test cases for the program. One of the structures was to be representative of a typical steel-frame building. The other structure was to be representative of a typical reinforced-concrete building. As part of the plan, the program tester was presented with a list of five buildings (steel structures and reinforced-concrete structures) for which detailed plans were available and which also experienced the effects of the 1971 San Fernando earthquake. The tester was asked to select from this list one building which he would then model using the present computer program. The selection of one of these specific buildings would allow the comparison of computed earthquake damages provided by the program with those actually recorded during the 1971 earthquake. The second building was selected from the program testers own list. The user was then provided with a draft copy of the present computer manual. He was to initially proceed without any guidance, other than that provided in the manual. Questions, however, were to be answered as they arose, and were

to be noted for future reference. The J.H. Wiggins Company was to be responsible for the actual execution of the computer examples, i.e., all interface with the J.H. Wiggins company computer system was to be done by a staff member. The computer analysis was to be done on the UCS CDC 7600 system in Kansas City. All program input data were to be developed by the tester. The input was to be recorded on standard 80 - column computer code sheets and submitted for execution. An in-house staff member was to be made available at all times to help resolve any errors which may have resulted from misinterpretation of the manual, making appropriate notations for the record. Modeling problems, however, were to be resolved by the user. The user was expected to provide the following:

(1) Computer output results, using the detailed frame model option, for the two subject buildings.

(2) Comments on the adequacy of the model to realistically estimate structural damage.

(3) Comments on the ease of use of the computer manual; and

(4) Recommendations regarding items (2) and (3) which might improve the computer model and/or program manual.

## 4.2 Discussion of Test Cases

The two buildings selected as test cases for the program were: (1) a 12-story reinforced-concrete structure (Bank of California, 15250 Ventura Blvd., Los Angeles); and (2) a ten-story steel frame structure (located in Long Beach, California). The reinforced-concrete structure, built in 1970 under the requirements of the 1968 Los Angeles City Building Code, incurred both structural and non-structural damage during the 1971 San Fernando earthquake. The steel structure, currently under construction has not yet experienced the effect of a major earthquake.

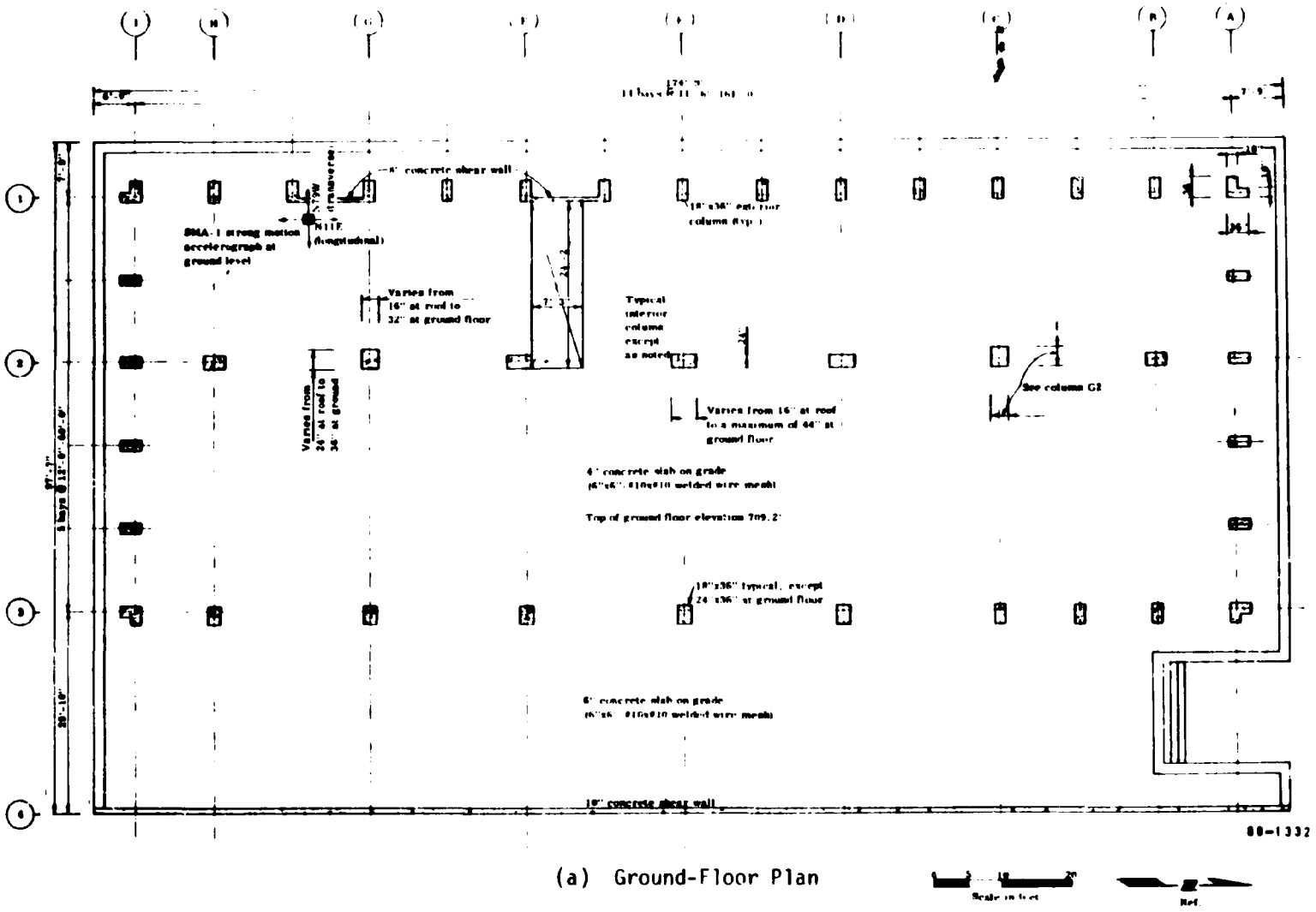
### 4.2.1 Example 1: Reinforced Concrete Frame Building

The Bank of California building, completed in 1970 at a cost of $4 million, currently functions as an office building. The structure is located some 17 miles south of the 1971 epicenter, near the southern end of the San Fernando Valley. Except at the lower levels where some shear walls are located, lateral forces in that direction are resisted by moment-resisting reinforced-concrete space frames consisting of columns and girders. A detailed discussion, as well as presentation of the structural system is provided in Reference [15]. Floor plans and a typical elevation view are reproduced in Figure 4-1.

Strong-motion accelerographs were located on the roof, seventh floor, and ground floor during the earthquake. At each level, accelerations were recorded along the vertical and both horizontal axes of the building. Approximately 28 seconds of motion was recorded. Table 4-1 lists peak recorded accelerations for each floor.

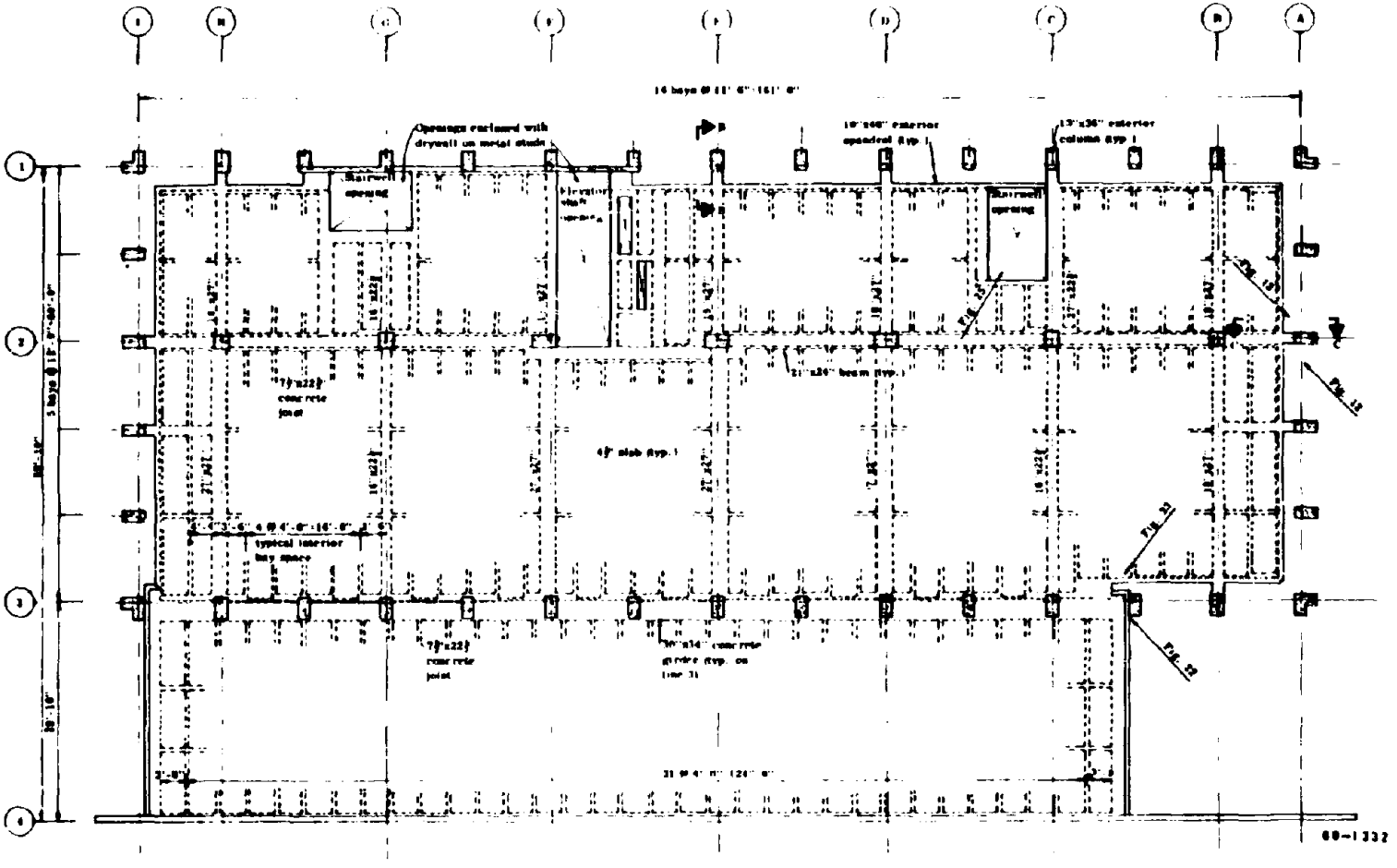
The structure experienced both structural and nonstructural damage. Structural damage consisted of cracking and spalling of concrete columns, spandrels, girder stubs, and a parapet wall at the first story [15]. Nonstructural damage resulted to partitions, ceilings, stairwells, stairs, mechanical equipment, and some furnishings. All of the damage experienced was repairable. Repairs totaled $44,000; $12,000 was spent on epoxy repair of damaged concrete elements [15].

The results of the computer analysis are presented in Table 4-2. Damages are listed for each story by frame, shear wall, glass and nonstructural components sensitive to floor motion. The actual recorded ground motions during the 1971 San Fernando earthquake for the ground level were used as input to the program (peak ground acceleration equal to 0.149g; peak ground velocity equal to 9.25 in/sec; and peak ground



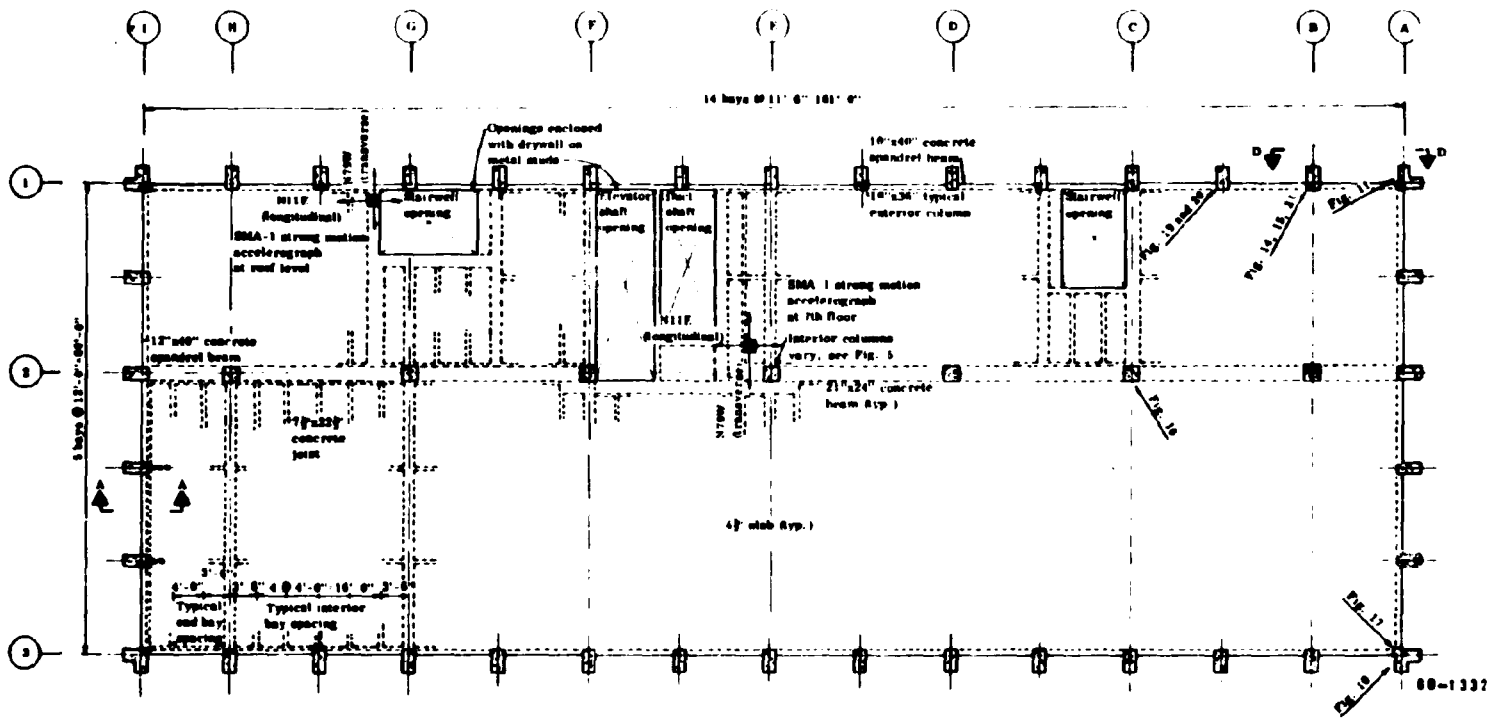

(a) Ground-Floor Plan

Figure 4-1. Example 1: Reinforced Concrete Frame Building - Bank of California [15]



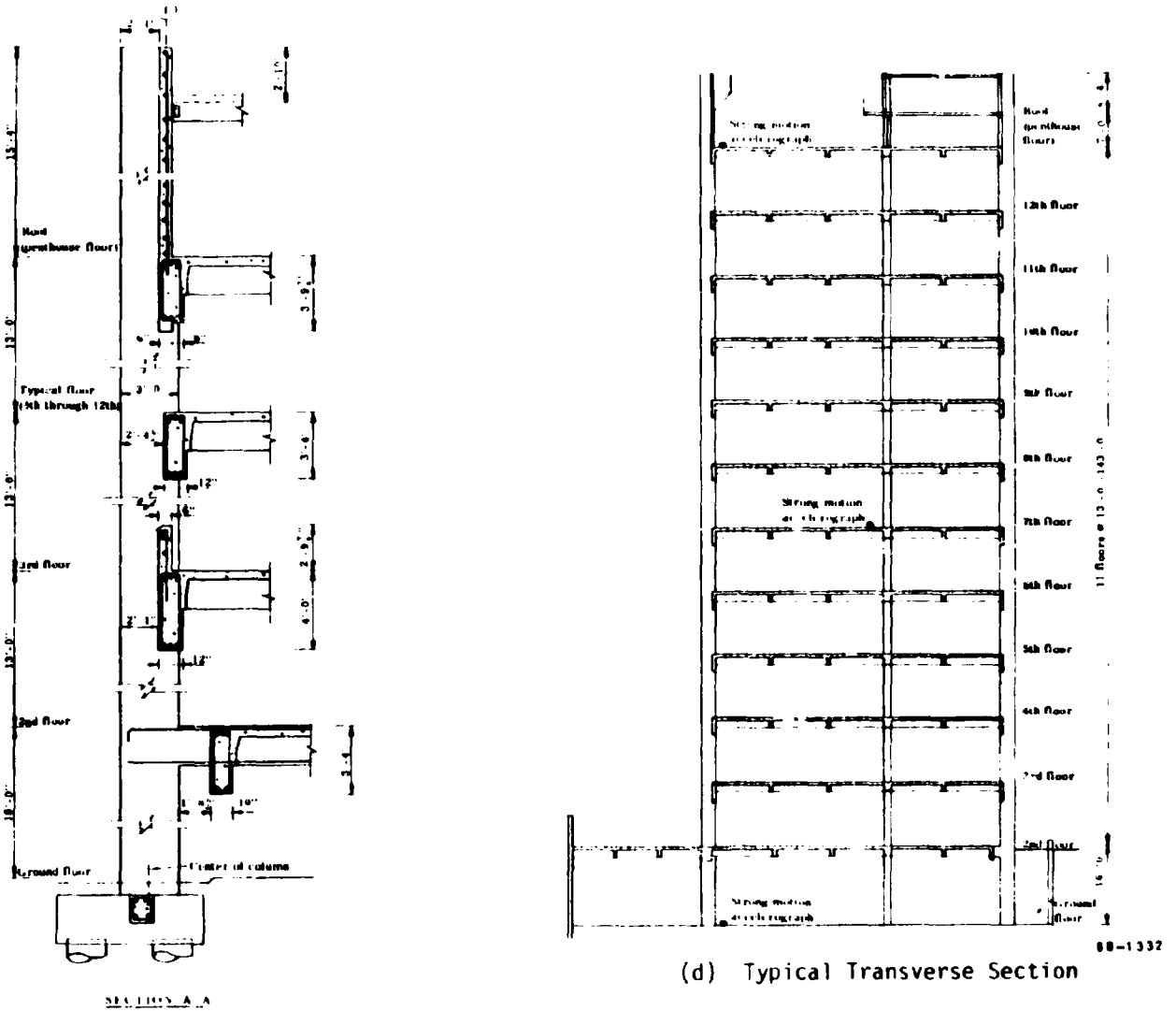
(b) Second-Floor Plan

Figure 4-1. Example 1: Reinforced Concrete Frame Building - Bank of California [15](cont'd)




(c) Typical Floor-Plan - Third through Twelfth Floors

Figure 4-1. Example 1: Reinforced Concrete Frame Building - Bank of California [15](cont'd)




(d) Typical Transverse Section

Figure 4-1. Example 1: Reinforced Concrete Frame Building - Bank of California [15](cont'd)

Table 4-1. Peak Recorded Accelerations During the 1971 San Fernando Earthquake for the Bank of California

| STATION | LONGITUDINAL (N.11°E.) COMPONENT | TRANSVERSE (N.79°W.) COMPONENT | VERTICAL COMPONENT |
|---|---|---|---|
| | g | g | g |
| ROOF | 0.288 | 0.199 | 0.144 |
| 7th FLOOR | 0.260 | 0.242 | 0.155 |
| GROUND FLOOR | 0.225 | 0.149 | 0.096 |

Table 4-2. Assessed Earthquake Damageability for Example 1: Reinforced Concrete Frame Building. Peak Ground Acceleration - 0.149g

TOTAL EARTHQUAKE DAMAGE TO BUILDING (PERCENT OF REPAIR COST/ REPLACEMENT COST)

BUILDING CLASSIFICATION: REINFORCED CONCRETE FRAME

OVERALL QUALITY RATING: AVERAGE

| TOTAL NO. STY. | AVERAGE INTERSTY. DRIFT | BUILDING PERCENT DAMAGE |
|---|---|---|
| 12 | .0043 | 3.14 |

EARTHQUAKE DAMAGE TO STRUCTURAL COMPONENTS (PERCENT OF REPAIR COST/ REPLACEMENT COST)

| | FRAME TYPE: 2 | | | | SHEARWALL TYPE: 0 | | | |
|---|---|---|---|---|---|---|---|---|
| STORY NO. | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR |
| 12 | .0024 | .77 | 1.48 | 5.25 | .0024 | 0.00 | 0.00 | 0.00 |
| 11 | .0032 | 1.07 | 2.19 | 8.66 | .0032 | 0.00 | 0.00 | 0.00 |
| 10 | .0039 | 1.30 | 2.75 | 11.54 | .0039 | 0.00 | 0.00 | 0.00 |
| 9 | .0044 | 1.46 | 3.20 | 14.00 | .0044 | 0.00 | 0.00 | 0.00 |
| 8 | .0047 | 1.63 | 3.56 | 16.04 | .0047 | 0.00 | 0.00 | 0.00 |
| 7 | .0051 | 1.74 | 3.86 | 17.75 | .0051 | 0.00 | 0.00 | 0.00 |
| 6 | .0053 | 1.84 | 4.11 | 19.24 | .0053 | 0.00 | 0.00 | 0.00 |
| 5 | .0055 | 1.92 | 4.32 | 20.53 | .0055 | 0.00 | 0.00 | 0.00 |
| 4 | .0057 | 1.97 | 4.46 | 21.37 | .0057 | 0.00 | 0.00 | 0.00 |
| 3 | .0054 | 1.87 | 4.20 | 19.75 | .0054 | 0.00 | 0.00 | 0.00 |
| 2 | .0035 | 1.15 | 2.39 | 9.67 | .0035 | 0.00 | 0.00 | 0.00 |
| 1 | .0025 | .80 | 1.55 | 5.55 | .0025 | 0.00 | 0.00 | 0.00 |

EARTHQUAKE DAMAGE TO NONSTRUCTURAL COMPONENTS (PERCENT OF COMPONENTS DAMAGED)

| | COMPONENTS SENSITIVE TO FLOOR MOTION | | | | GLASS | | | |
|---|---|---|---|---|---|---|---|---|
| STORY NO. | FLOOR ACCEL.(G) | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR |
| 12 | .22 | 1.69 | 10.89 | 70.04 | .0024 | 1.04 | 2.51 | 7.13 |
| 11 | .17 | 1.54 | 9.76 | 61.79 | .0032 | 2.11 | 4.45 | 11.74 |
| 10 | .16 | 1.38 | 8.55 | 53.11 | .0039 | 3.22 | 6.27 | 15.82 |
| 9 | .15 | 1.22 | 7.42 | 45.13 | .0044 | 4.26 | 7.85 | 19.23 |
| 8 | .16 | 1.07 | 6.38 | 38.01 | .0047 | 5.18 | 9.19 | 22.06 |
| 7 | .16 | .93 | 5.43 | 31.61 | .0051 | 5.99 | 10.34 | 24.44 |
| 6 | .17 | .80 | 4.51 | 25.60 | .0053 | 6.73 | 11.35 | 26.51 |
| 5 | .17 | .65 | 3.58 | 19.61 | .0055 | 7.39 | 12.24 | 28.31 |
| 4 | .16 | .49 | 2.59 | 13.56 | .0057 | 7.83 | 12.82 | 29.49 |
| 3 | .15 | .32 | 1.59 | 7.74 | .0054 | 6.99 | 11.70 | 27.23 |
| 2 | .13 | .21 | .97 | 4.47 | .0035 | 2.48 | 5.07 | 13.16 |
| 1 | .07 | .08 | .29 | 1.10 | .0025 | 1.12 | 2.68 | 7.55 |

FRAME TYPES:

0: NOT APPLICABLE
1: STEEL
2: REINFORCED CONCRETE
3: PRECAST CONCRETE

SHEARWALL TYPES:

0: NOT APPLICABLE
1: REINFORCED CONCRETE
2: PRECAST CONCRETE
3: BRICK MASONRY

displacement equal to 4.06 in). These ground motions correspond to the transverse direction of the building.

The total damage ratio computed for the building is shown in Table 4-2 to be 3.14%, corresponding to an average interstory drift of 0.0043. This compares with the reported damage ratio of 1.1% based on the estimated repair cost of $44,000 and a replacement cost of $4 million. One must keep in mind that the 2D analysis was performed in only the tranverse direction, while actual damage is a result of motion in both the transverse and longitudinal directions. The computed building period was 2.4 sec compared with the 3.0 sec period recorded during the earthquake and the 1.6 sec period measured after the earthquake [15]. The reported damage ratio is within one standard deviation of the computed "damage."

Other damageability information is listed by story, by component, and by the quality rating of that component. Damage estimates are shown for all three quality ratings, considering the possibility that all components of a given type may not be uniform in their quality ratings. In addition, such a display shows the sensitivity of results to the selected quality rating.

Wind damageability was also assessed for this building, based on an assumed wind velocity of 125 mph (chosen arbitrarily to demonstrate the computational procedure). These results are shown in Table 4-3. Glass damage is seen to dominate, particularly in the upper stories of the building at the corners. Wind pressures printed out by story are those pressures (at height H above ground) given by the formula

$$p = \frac{1}{2} C_p C_g \rho V_o^2 \left(\frac{H}{H_o}\right)^{2\alpha}$$

where $V_o$ = 125 mph corresponding to a height, $H_o$ = 30 ft., and in this case the site factor $\alpha$ = 1/7.

Table 4-3. Assessed Wind Damageability for Example 1: Reinforced Concrete Frame Building. Wind Velocity - 125 mph

TOTAL WIND DAMAGE TO BUILDING (PERCENT OF REPAIR COST/ REPLACEMENT COST)

BUILDING CLASSIFICATION: REINFORCED CONCRETE FRAME

OVERALL QUALITY RATING: AVERAGE

| TOTAL NO. STY. | AVERAGE INTERSTY. DRIFT | BUILDING PERCENT DAMAGE |
|---|---|---|
| 12 | .0005 | .07 |

WIND DAMAGE TO STRUCTURAL COMPONENTS (PERCENT OF REPAIR COST/ REPLACEMENT COST)

| | FRAME TYPES: 2 | | | | SHEARWALL TYPE: 0 | | | |
|---|---|---|---|---|---|---|---|---|
| STORY NO. | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR |
| 12 | .0002 | 0.00 | 0.00 | 0.00 | .0002 | 0.00 | 0.00 | 0.00 |
| 11 | .0003 | 0.00 | 0.00 | 0.00 | .0003 | 0.00 | 0.00 | 0.00 |
| 10 | .0003 | 0.00 | 0.00 | 0.00 | .0003 | 0.00 | 0.00 | 0.00 |
| 9 | .0004 | 0.00 | 0.00 | 0.00 | .0004 | 0.00 | 0.00 | 0.00 |
| 8 | .0005 | 0.00 | 0.00 | 0.00 | .0005 | 0.00 | 0.00 | 0.00 |
| 7 | .0006 | 0.00 | 0.00 | .57 | .0006 | 0.00 | 0.00 | 0.00 |
| 6 | .0006 | 0.00 | 0.00 | .76 | .0006 | 0.00 | 0.00 | 0.00 |
| 5 | .0007 | 0.00 | 0.00 | .95 | .0007 | 0.00 | 0.00 | 0.00 |
| 4 | .0007 | 0.00 | 0.00 | 1.09 | .0007 | 0.00 | 0.00 | 0.00 |
| 3 | .0007 | 0.00 | 0.00 | 1.00 | .0007 | 0.00 | 0.00 | 0.00 |
| 2 | .0005 | 0.00 | 0.00 | 0.00 | .0005 | 0.00 | 0.00 | 0.00 |
| 1 | .0003 | 0.00 | 0.00 | 0.00 | .0003 | 0.00 | 0.00 | 0.00 |

WIND DAMAGE TO NONSTRUCTURAL COMPONENTS (PERCENT OF COMPONENTS DAMAGED)

| | CEILINGS AND PARTITIONS | | | | GLASS (PERCENT DAMAGE BY WALL) | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| STORY NO. | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | WIND PRESS (PSF) | WALL 1 | WALL 2 | WALL 3 | WALL 4 | CORNERS |
| 12 | .0002 | .13 | .22 | 1.78 | 27.68 | 16.67 | 1.77 | 8.91 | 8.91 | 99.98 |
| 11 | .0003 | .22 | .36 | 2.56 | 26.54 | 13.72 | 1.45 | 7.27 | 7.27 | 99.95 |
| 10 | .0003 | .34 | .55 | 3.44 | 25.34 | 11.05 | 1.18 | 5.81 | 5.81 | 99.84 |
| 9 | .0004 | .46 | .75 | 4.27 | 24.09 | 8.68 | .94 | 4.55 | 4.55 | 99.53 |
| 8 | .0005 | .58 | .95 | 5.05 | 22.78 | 6.53 | .73 | 3.47 | 3.47 | 98.74 |
| 7 | .0006 | .69 | 1.15 | 5.76 | 21.40 | 4.89 | .56 | 2.56 | 2.56 | 96.85 |
| 6 | .0006 | .80 | 1.33 | 6.39 | 19.92 | 3.45 | .42 | 1.83 | 1.83 | 92.72 |
| 5 | .0007 | .90 | 1.48 | 6.92 | 18.33 | 2.32 | .30 | 1.24 | 1.24 | 84.58 |
| 4 | .0007 | .96 | 1.60 | 7.29 | 16.58 | 1.45 | .21 | .80 | .40 | 70.58 |
| 3 | .0007 | .92 | 1.53 | 7.07 | 14.63 | .83 | .14 | .47 | .47 | 50.25 |
| 2 | .0005 | .50 | .82 | 4.57 | 12.35 | .41 | .08 | .24 | .24 | 26.87 |
| 1 | .0003 | .31 | .51 | 3.26 | 9.49 | .15 | .04 | .10 | .10 | 8.04 |

FRAME TYPES:

0: NOT APPLICABLE
1: STEEL
2: REINFORCED CONCRETE
3: PRECAST CONCRETE

SHEARWALL TYPES:

0: NOT APPLICABLE
1: REINFORCED CONCRETE
2: PRECAST CONCRETE
3: BRICK MASONRY

### 4.2.2 Example 2: Steel Frame Building

The steel frame structure, currently being constructed in Long Beach, California, will function as an office building. The lateral forces are resisted in each direction by moment-resisting steel frames. Attached adjacent to the ten-story structure is a one-story office area. The structure has not as yet experienced the effects of a major earthquake. A plan view and detailed framing elevations are shown in Figure 4-2.

For this building, a Richter magnitude of 6.5, twelve miles away was used. The 6.5 was assumed to occur on the Newport-Inglewood fault at the closest distance to the site.

The total damage ratio computed for the building is shown in Table 4-4 to 1.53%, corresponding to an average interstory drift of 0.0068. Other damageability information is listed by story, by component, and by the quality rating of that component. Damage estimates are shown for all three quality ratings, as was done for the previous reinforced-concrete example.

The next section documents the tester's findings and comments on the two examples, as well as comments on the computer program and manual.

## 4.3 User Feedback

The following are Mr. Hopewell's comments on the testing of the DAMAGE computer program.

> The results concerning the expected damage to the two buildings are believable. The reinforced-concrete Bank of California building has construction characteristics that are not recommended as a general practice unless considerable investigations are

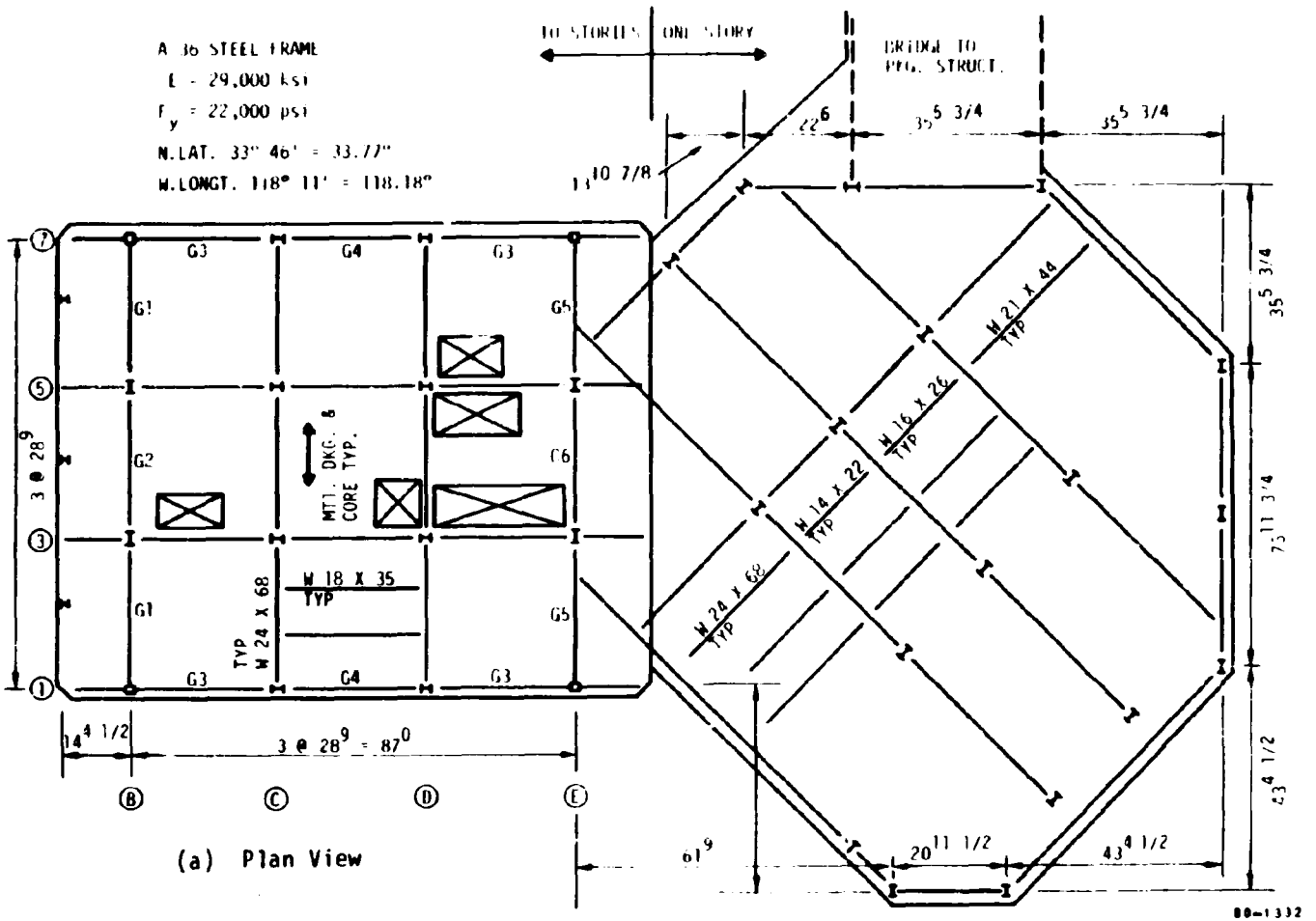
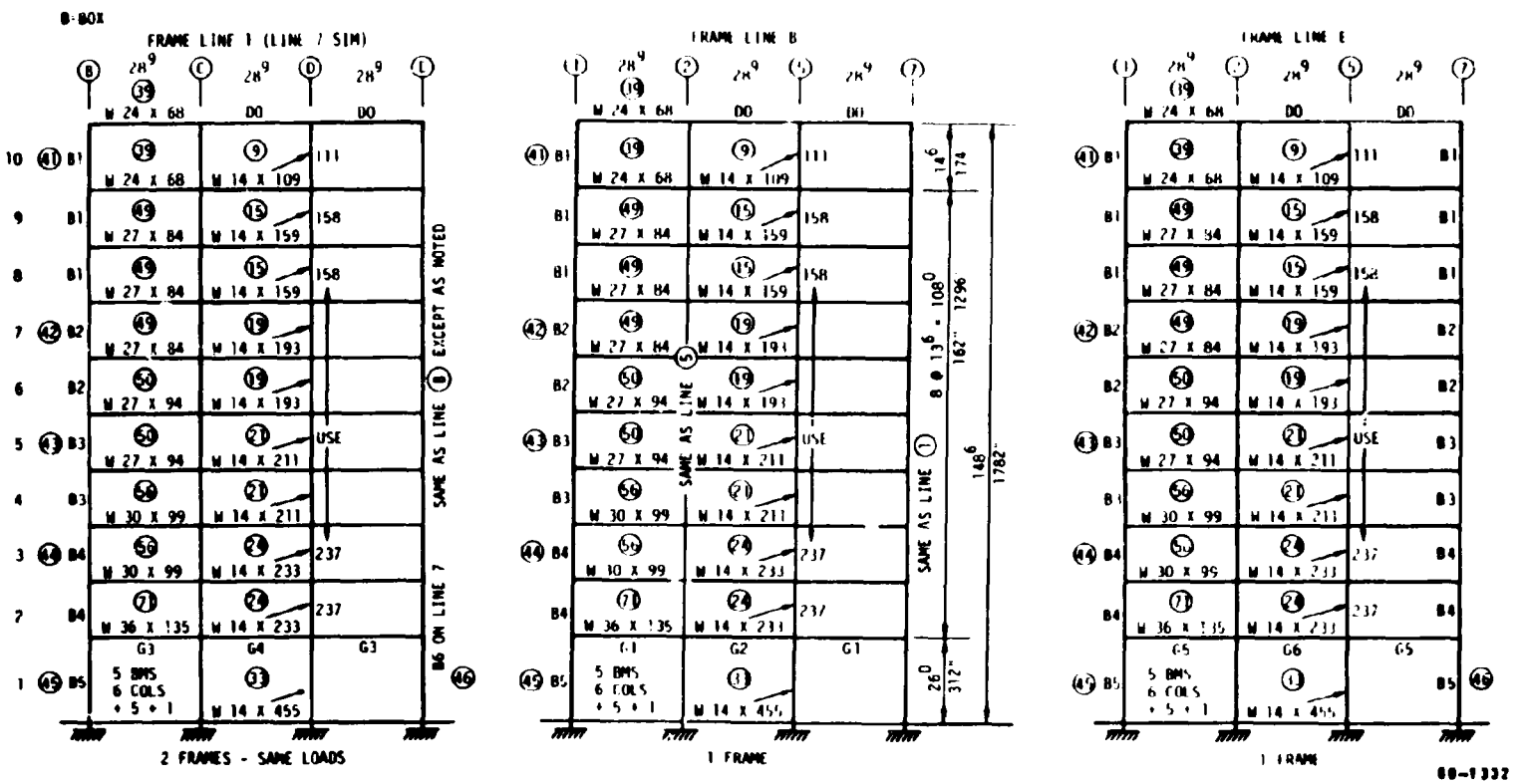



Figure 4-2. Example 2: Steel Frame Building




(b) Framing Details

Figure 4-2. Example 2: Steel Frame Building (cont'd)

Table 4-4. Assessed Earthquake Damageability for Example 2: Steel Frame Building. Peak Ground Acceleration - 0.295g

TOTAL EARTHQUAKE DAMAGE TO BUILDING (PERCENT OF REPAIR COST/ REPLACEMENT COST)

BUILDING CLASSIFICATION: STEEL FRAME

OVERALL QUALITY RATING: AVERAGE

| TOTAL NO. STY. | AVERAGE INTERSTY. DRIFT | BUILDING PERCENT DAMAGE |
|---|---|---|
| 10 | .0068 | 1.53 |

EARTHQUAKE DAMAGE TO STRUCTURAL COMPONENTS (PERCENT OF REPAIR COST/ REPLACEMENT COST)

| | FRAME TYPE: 1 | | | | SHEARWALL TYPE: 0 | | | |
|---|---|---|---|---|---|---|---|---|
| STORY NO. | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR |
| 10 | .0066 | .70 | 1.48 | 9.54 | .0066 | 0.00 | 0.00 | 0.00 |
| 9 | .0072 | .78 | 1.66 | 11.45 | .0072 | 0.00 | 0.00 | 0.00 |
| 8 | .0073 | .79 | 1.67 | 11.58 | .0073 | 0.00 | 0.00 | 0.00 |
| 7 | .0071 | .77 | 1.64 | 11.23 | .0071 | 0.00 | 0.00 | 0.00 |
| 6 | .0070 | .76 | 1.60 | 10.64 | .0070 | 0.00 | 0.00 | 0.00 |
| 5 | .0066 | .71 | 1.47 | 9.71 | .0066 | 0.00 | 0.00 | 0.00 |
| 4 | .0064 | .68 | 1.41 | 9.10 | .0064 | 0.00 | 0.00 | 0.00 |
| 3 | .0060 | .63 | 1.28 | 8.01 | .0060 | 0.00 | 0.00 | 0.00 |
| 2 | .0059 | .62 | 1.25 | 7.74 | .0059 | 0.00 | 0.00 | 0.00 |
| 1 | .0060 | .64 | 1.92 | 14.04 | .0060 | 0.00 | 0.00 | 0.00 |

EARTHQUAKE DAMAGE TO NONSTRUCTURAL COMPONENTS (PERCENT OF COMPONENTS DAMAGED)

| | COMPONENTS SENSITIVE TO FLOOR MOTION | | | | GLASS | | | |
|---|---|---|---|---|---|---|---|---|
| STORY NO. | FLOOR ACCEL.(G) | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR |
| 10 | .27 | 2.10 | 11.98 | 93.15 | .0066 | 2.71 | 5.45 | 17.15 |
| 9 | .13 | 1.60 | 10.22 | 65.10 | .0072 | 3.09 | 7.30 | 22.67 |
| 8 | .15 | 1.32 | 8.11 | 50.00 | .0073 | 3.98 | 7.43 | 23.06 |
| 7 | .16 | 1.17 | 7.65 | 42.58 | .0071 | 3.75 | 7.08 | 22.02 |
| 6 | .17 | 1.09 | 6.53 | 39.01 | .0070 | 3.49 | 6.69 | 20.86 |
| 5 | .18 | 1.05 | 6.21 | 36.67 | .0066 | 2.61 | 5.63 | 17.69 |
| 4 | .18 | .99 | 5.83 | 34.36 | .0064 | 2.47 | 5.06 | 15.98 |
| 3 | .19 | .91 | 5.25 | 30.42 | .0060 | 1.92 | 4.13 | 13.14 |
| 2 | .18 | .77 | 4.33 | 25.63 | .0059 | 1.79 | 3.91 | 12.47 |
| 1 | .17 | .58 | 2.98 | 15.90 | .0060 | 5.04 | 10.13 | 30.94 |

FRAME TYPES:

0: NOT APPLICABLE
1: STEEL
2: REINFORCED CONCRETE
3: PRECAST CONCRETE

SHEARWALL TYPES:

0: NOT APPLICABLE
1: REINFORCED CONCRETE
2: PRECAST CONCRETE
3: BRICK MASONRY

made to justify the design. Columns that do not continue to the foundation are not recommended; using shear walls in the lower level only and using additional frames in the lower two levels creates a change in stiffness that is not recommended; and the setback of spandrels creates unwarranted torsion in the beams and questionable interaction of the spandrels to columns for the anticipated frame action. The problems created by these items are not readily apparent from a normal computer analysis. Since structural damage by yielding was experienced on this building during the '71 earthquake it is likely other parts of the structure were approaching or at yield and additional damage above that experienced in '71 is very possible during a similar earthquake for an inherently brittle material such as concrete.

The typical steel frame Long Beach office building was considered designed in accordance to the 1974 "Recommended Lateral Force Requirements" of the Structural Engineers Association of California, which is the basis for the 1976 "Uniform Building Code" lateral force requirements. As a ductile moment space frame designed to a code that was revised to include knowledge gained in the 1971 earthquake, the building should be considered as superior quality construction. (See comments on Building Damage Information Card.) The building did have a one-story office area on one side attached to the tower frame which creates an unsymmetrical condition on the second floor. The spandrel beams at the four corners of the tower are cantilevered 14 feet in both directions which may be subjected to excessive forces from vertical accelerations. The computed damage to this building is a realistic possibility for an earthquake similar to the one used in the program.

The Computer Manual should be prepared on the assumption that the user has no prior experience in modeling a building for a computer analysis. The fact that a Building Official and his staff may have had some experience working with Design Engineers on computer outputs does not help him on input. Small computer programs that he may be familiar with are not necessarily modeled in a similar manner, therefore, I suggest that the Manual include a reproduction of the first few entries on the input form and a glossary of terms.

The following are specific comments on the sections in the Computer Manual:

1. Sec. 2.2.1 Earthquake Input:
   A.* Return Period card. The average Building Official will require help in selecting this information. I Suggest

*Capital letters refer to card identifiers in the Users Manual.

including references so he knows where to start looking for information so that a reasonable decision can be made.

2. Sec. 2.3.1 Detailed Frame Model:
B thru K. It should be noted that each frame shall be modeled using subsections B thru K for each frame and that subsections G thru J is repeated within each frame model until all stories are entered.

H. Story Description Card. The first field entry comment should read "Number of Consecutive Stories" of identical height and beam and column properties.

3. Sec. 2.5 Damage
A. Building Damage Information Card:
Col. 8 (Field 1) entry items and Table 8 do not equate to my thinking. All buildings in this area have been constructed to some minimum code for years with, I hope, some quality assurance. The larger buildings, especially multi-story, require "Special" or "Registered" Inspectors on job site and they are inturn checked on by the Building Official. For buildings subjected to earthquakes, I equate buildings to the code under which they were constructed, the configuration, symmetry and any known problems during and since construction. In general I would start by considering an entry of 1 for pre-1967 UBC constructed buildings, 2 for pre-1976 UBC and 3 for buildings constructed to the 1976 UBC modified by my above stated concepts. Also most buildings are not readily inspected after completing for the various qualities shown in Table 8. (These comments apply to Col. 40 also.) Col 32 (Field 4) is not clear as to intent. The Bank of California building was clad with glass with some steel stud bulkheads and the Typical L.B. Steel Frame building had precast decorative concrete spandrels and verticals bolted to the frame using slotted holes. Col. 48 (Field 6) does not include concrete diaphrams. Also metal decking should indicate that it includes concrete fill.

C. Window Information Card. This should be clarified. I understand that nominal window dimensions are the height and width of an average pane of glass.

D. Earthquake damage Information Card.
Col. 9-16 (Field 2) entry should relate to the average clearance between the mullions and pane of glass.

In general the Computer Manual provided sufficient information for my structures once I, as a novice, got started.

A look at the manual for structures other than frames gives me the impression I would be hard pressed to model a shear wall or braced frame building.

The use of this program would be useful to access anticipated disaster damage so that a city or other agency can take mitigating measures or make advance plans to mitigate the physical and social problems following a disaster.

MARVIN W. HOPEWELL
Civil Engineer

## 4.4 Closure

The following changes to the User's Manual were made in response to Mr. Hopewell's comments in Section 4.3 - User Feedback.

1. Sec. 2.2.1 - Earthquake Input

   A. Return Period Card

   The earthquake magnitude "return period" card was deleted. It was decided to allow the user to input (a) a Richter magnitude and hypocentral distance and (b) if the user choose to, a set of user - determined earthquake site ground motions. Since a probabilistic earthquake input load is no longer used, the return period card is not necessary.

2. Sec. 2.3.1 - Detailed Frame Model

   B. thru K. Detailed Frame Model Data

   Comments have been incorporated.

   H. Story Description Card

   Comments have been incorporated.

3. Sec. 2.5 - Damage

A. Building Damage Information Card

The quality classifications with respect to frame, shear wall and non-structural systems have been limited to definition of 'poor,' 'average,' and 'good.' It is believed that the guidelines, outlined in Tables 7 and 8 provide the user with enough information to make an accurate assessment of the quality level.

Damages are no longer computed separately for diaphragms. Rather, these damages are included with those of the framing and shear walls.

C. Window Information Card

Statement in manual has been clarified.

D. Earthquake Damage Information Card

Due to a change in the method of computing earthquake damage to windows, that information is no longer necessary. As a result, that input card was eliminated.

# 5. CONCLUSIONS AND RECOMMENDATIONS

## 5.1 Conclusions

The primary objectives of the project as enumerated in Section 1.2 have been achieved. It is hoped that the 'DAMAGE' computer program resulting from this effort will prove to fill the needs of the local building official for whom it was designed. The following conclusions pertain directly to the stated objectives:

(1) The current literature was reviewed for damageability models suitable to the purpose of this project. They were classified into three groups - empirical, theoretical and subjective. None was found to satisfy all of the requirements. Most of the empirical models do not interface with a structural model, in that measures of damage are expressed directly as functions of structural input parameters such as MMI, rather than structural response parameters such as interstory drift, story shear, base shear, etc. Attempts which have been made to correlate damage with structural response have been largely unsuccessful because of insufficient data. Theoretical models, on the other hand tend to be too complex for those who are inexperienced with them, and require input parameters which may be difficult to evaluate because they do not relate well to available damage data. Purely subjective methods fail to meet the requirement of objectivity.

(2) Damageability models, which express damage in terms of a median damage ratio versus interstory drift, were developed by combining limited empirical data with subjective judgment using a Bayesian statistical estimator. These models employ parameters such as drift-to-yield and ductility-to-failure which are related to the slope and intercept of a linear regression of damage ratio on interstory drift.

(3) An existing computer program already having capabilities for modeling hazard loading, structural response and damageability was modified to achieve a simpler more "streamlined" computational procedure. The original damageability models were replace by those documented herein. The amount of required input was reduced substantially by eliminating unnecessary options and coding much of the input data directly into the program.

(4) A new Users Manual for the program was written in a format which should be easier for non-computer oriented personnel to follow. Sample input coding sheets are provided.

(5) Finally, the program and its documentation were independently tested by a local building official who had no prior experience with the program and who had only minimal computer background. Two actual buildings were analyzed, one reinforced concrete frame and one steel frame building. Damageabilities computed for the concrete building are compared with available San Fernando earthquake damage data available for that building. User test procedures, feedback, and response to that feedback are documented herein.

With an acceptable level of damageability assigned by the proper officials, new designs as well as existng structures can be assessed for adequacy and inclusion in a community's current building stock.

In addition to these conclusions which relate formally to the objectives of the project, several others are worth noting. Foremost among them is the rather fortunate (if not surprising) way in which the primary damageability models for total damage and structural damage came together. Some of the significant points are listed below:

- Interstory drift was found to be simply related to MMI, or other measures of ground motion intensity, via two empirically derived constants: T/N which is the ratio of building period to number of stories, and $\Gamma$ which may be interpreted as a modal participation factor. These constants were found to be T/N = 0.1 and 0.16, and $\Gamma$ = 1.05 and 2.34, for concrete and steel buildings respectively. Based on these factors and interstory drifts obtained directly from strong motion records, steel buildings appear to experience (on the average) between 2.5 and 3.5 times more interstory drift than concrete buildings for a given ground motion intensity up to that which causes first yield. This is significantly more than the relative T/N ratios alone indicate, and may be due to greater higher mode participation in steel buildings.

- The duration factor, C(T,M), plays a significant part in the damageability models. There is some evidence, although weak at the present time, that long-period buildings may experience less earthquake damage than short-period buildings (other things being equal) because they undergo fewer cycles of response. Interestingly, the wind damage information available for high-rise buildings tends to confirm this observation, by indicating higher damage levels for wind than earthquake, for the same interstory drift above the 0.5% damage threshold. Presumably, this would result from the relatively long duration of wind-induced structural motion, compared with that of earthquake.

- Based on rather limited data for steel, concrete and brick masonry buildings, the "poor" quality rating expressed in the various damageability models seems to correlate with pre-1933 building damage data, while the "average" quality rating corre-

lates with damage data for buildings built between 1933 and 1971.

## 5.2 Recommendations

While experience to date with the 'DAMAGE' computer program has been encouraging, it must be recognized that the damageability models are based on relatively sparse data, and the judgment of a few individuals. Hopefully, the simplicity and transparency of the models will help to facilitate interpretation, so that any unexpected results may be traced to appropriate causal factors, whether they relate to applied loads, structural response, or damageability. As new damage information becomes available, whether subjective or objective, it can be readily incorporated in the models. As with any new tool, careful surveillance should be maintained while initial experience accumulates.

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# APPENDIX

# COMPUTER PRINT-OUT FOR TEST CASES

# EXAMPLE 1: REINFORCED CONCRETE FRAME BUILDING

```
RUN IDENTIFICATION:
            VENTURA BLVD BANK OF CA   TRANSVERSE
EARTHQUAKE HAZARD IS INCLUDED
WIND HAZARD IS INCLUDED
SITE LOCATION: LATITUDE   34.02  DEGREES, LONGITUDE  118.50  DEGREES
BUILDING MODELING OPTION: DETAILED FRAME
```

E A R T H Q U A K E  L O A D  A N A L Y S I S

SITE GROUND MOTION CHARACTERISTICS

| | | |
|---|---|---|
| RICHTER MAGNITUDE | = | 6.50E+00 |
| HYPOCENTRAL DISTANCE (MILES) | = | 2.25E+01 |
| SITE ACCELERATION (G) | = | 1.49E-01 |
| SITE VELOCITY (IN/SEC) | = | 9.25E+00 |
| SITE DISPLACEMENT (IN) | = | 4.06E+00 |

SITE GROUND MOTION SPECTRUM

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| PERIOD (SEC) | .10 | .20 | .30 | .40 | .50 | .60 | .70 | .80 | .90 | 1.00 |
| VEL (IN/SEC) | .91 | 1.83 | 2.74 | 3.66 | 4.57 | 5.48 | 6.40 | 7.31 | 8.23 | 9.14 |
| PERIOD (SEC) | 1.10 | 1.20 | 1.30 | 1.40 | 1.50 | 1.60 | 1.70 | 1.80 | 1.90 | 2.00 |
| VEL (IN/SEC) | 9.25 | 9.25 | 9.25 | 9.25 | 9.25 | 9.25 | 9.25 | 9.25 | 9.25 | 9.25 |
| PERIOD (SEC) | 2.10 | 2.20 | 2.30 | 2.40 | 2.50 | 2.60 | 2.70 | 2.80 | 2.90 | 3.00 |
| VEL (IN/SEC) | 9.25 | 9.25 | 9.25 | 9.25 | 9.25 | 9.25 | 9.25 | 9.10 | 8.79 | 8.49 |
| PERIOD (SEC) | 3.10 | 3.20 | 3.30 | 3.40 | 3.50 | 3.60 | 3.70 | 3.80 | 3.90 | 4.00 |
| VEL (IN/SEC) | 8.22 | 7.96 | 7.72 | 7.49 | 7.28 | 7.08 | 6.89 | 6.71 | 6.53 | 6.37 |
| PERIOD (SEC) | 4.10 | 4.20 | 4.30 | 4.40 | 4.50 | 4.60 | 4.70 | 4.80 | 4.90 | 5.00 |
| VEL (IN/SEC) | 6.21 | 6.07 | 5.93 | 5.79 | 5.66 | 5.54 | 5.42 | 5.31 | 5.20 | 5.10 |
| PERIOD (SEC) | 5.10 | 5.20 | 5.30 | 5.40 | 5.50 | 5.60 | 5.70 | 5.80 | 5.90 | 6.00 |
| VEL (IN/SEC) | 5.00 | 4.90 | 4.81 | 4.72 | 4.63 | 4.55 | 4.47 | 4.39 | 4.32 | 4.25 |
| PERIOD (SEC) | 6.10 | 6.20 | 6.30 | 6.40 | 6.50 | 6.60 | 6.70 | 6.80 | 6.90 | 7.00 |
| VEL (IN/SEC) | 4.18 | 4.11 | 4.04 | 3.98 | 3.92 | 3.86 | 3.80 | 3.75 | 3.69 | 3.64 |
| PERIOD (SEC) | 7.10 | 7.20 | 7.30 | 7.40 | 7.50 | 7.60 | 7.70 | 7.80 | 7.90 | 8.00 |
| VEL (IN/SEC) | 3.59 | 3.54 | 3.49 | 3.44 | 3.40 | 3.35 | 3.31 | 3.27 | 3.23 | 3.18 |
| PERIOD (SEC) | 8.10 | 8.20 | 8.30 | 8.40 | 8.50 | 8.60 | 8.70 | 8.80 | 8.90 | 9.00 |
| VEL (IN/SEC) | 3.15 | 3.11 | 3.07 | 3.03 | 3.00 | 2.96 | 2.93 | 2.90 | 2.86 | 2.83 |
| PERIOD (SEC) | 9.10 | 9.20 | 9.30 | 9.40 | 9.50 | 9.60 | 9.70 | 9.80 | 9.90 | 10.00 |
| VEL (IN/SEC) | 2.80 | 2.77 | 2.74 | 2.71 | 2.68 | 2.65 | 2.63 | 2.60 | 2.57 | 2.55 |

```
W I N D   L O A D   A N A L Y S I S

ISITE=     1     ITEST=     2     ITY=     1     ISHAPE=     1

 USER3 VALUE OF WIND VELOCITY -FASTEST MILE AT 30 FT ABOVE GROUND - FREE FIELD IN MPH = 125.00
 RETURN PERIOD IN YEARS=   25.0
```

S T R U C T U R A L  A N A L Y S I S

```
NUMBER OF STORIES =     12
TOTAL BUILDING WIDTH (INCHES) =   720.0
TOTAL BUILDING LENGTH (INCHES) =  1932.0
HEIGHT OF PARAPET (INCHES)     =   184.0

NUMBER OF FRAMES  =      7
```

```
-- FRAME NO.  1 --  FRAME MODELING OPTION=    2

NUMBER OF BAYS  =    5
BEAMS:     E=   2100.0  G=    840.0  W(LBS/CU.FT)=   110.00
COLUMNS:   E=   3900.0  G=   1560.0  W(LBS/CU.FT)=   150.00

BEAM TYPE          AB          IB          FB
        1        480.0     64000.0       1.20
        2        400.0     53333.3       1.20

COLUMN TYPE        AC          IC          FC
        1        972.0     96228.0       1.50
        2        648.0     17496.0       1.20

BAY WIDTHS ------
    1 BAYS AT   144.000 INCHES
    3 BAYS AT   144.000 INCHES
    1 BAYS AT   144.000 INCHES

STORY HEIGHTS -------
   11 STORIES AT   156.000 INCHES
    1 STORIES AT   192.000 INCHES
```

```
-- FRAME NO.  2 --  FRAME MODELING OPTION=    2

NUMBER OF BAYS  =    5
BEAMS:     E=   2100.0  G=    840.0  W(LBS/CU.FT)=   110.00
COLUMNS:   E=   3900.0  G=   1560.0  W(LBS/CU.FT)=   150.00

BEAM TYPE          AB          IB          FB
        1        480.0     64000.0       1.20
        2        400.0     53333.3       1.20

COLUMN TYPE        AC          IC          FC
        1        972.0     96228.0       1.50
        2        648.0     17496.0       1.20

BAY WIDTHS ------
    1 BAYS AT   144.000 INCHES
    3 BAYS AT   144.000 INCHES
    1 BAYS AT   144.000 INCHES

STORY HEIGHTS -------
   11 STORIES AT   156.000 INCHES
    1 STORIES AT   192.000 INCHES
```

```
-- FRAME NO.  3 --  FRAME MODELING OPTION=    2

NUMBER OF BAYS  =    2
BEAMS:     E=   2100.0  G=    840.0  W(LBS/CU.FT)=   110.00
COLUMNS:   E=   3900.0  G=   1560.0  W(LBS/CU.FT)=   150.00

BEAM TYPE          AB          IB          FB
        1           .0          .0        1.20
        2        729.0     44286.8        1.20
        3        486.0     29524.5        1.20

COLUMN TYPE        AC          IC          FC
        1           .0          .0        1.20
        2        648.0     69984.0        1.20
        3        960.0     46080.0        1.20
        4        864.0     93312.0        1.20
        5       1056.0     50688.0        1.20

BAY WIDTHS ------

    1 BAYS AT   432.000 INCHES
    1 BAYS AT   288.000 INCHES

STORY HEIGHTS -------

   11 STORIES AT   156.000 INCHES
    1 STORIES AT   192.000 INCHES
```

```
-- FRAME NO.  4 --  FRAME MODELING OPTION=    2

NUMBER OF BAYS  =    2

BEAMS:     E=   2100.0  G=    840.0  W(LBS/CU.FT)=   110.00

COLUMNS:   E=   3900.0  G=   1560.0  W(LBS/CU.FT)=   150.00

BEAM TYPE          AB          IB          FB

        1           .0          .0        1.20
        2         729.0     44286.8       1.20
        3         486.0     24524.5       1.20

COLUMN TYPE        AC          IC          FC

        1           .0          .0        1.20
        2         648.0     69984.0       1.20
        3         960.0     46080.0       1.20
        4         864.0     93312.0       1.20
        5        1656.0     50688.0       1.20

BAY WIDTHS ------

    1 BAYS AT   432.000 INCHES
    1 BAYS AT   288.000 INCHES

STORY HEIGHTS -------

   11 STORIES AT   156.000 INCHES
    1 STORIES AT   192.000 INCHES
```

-- FRAME NO. 5 -- FRAME MODELING OPTION= 2

NUMBER OF BAYS = 2

BEAMS: E= 2100.0 G= 840.0 W(LBS/CU.FT)= 110.00

COLUMNS: E= 3900.0 G= 1560.0 W(LBS/CU.FT)= 150.00

| BEAM TYPE | AB | IB | FB |
|---|---|---|---|
| 1 | .0 | .0 | 1.20 |
| 2 | 729.0 | 44286.6 | 1.20 |
| 3 | 486.0 | 29524.5 | 1.20 |

| COLUMN TYPE | AC | IC | FC |
|---|---|---|---|
| 1 | .0 | .0 | 1.20 |
| 2 | 648.0 | 69984.0 | 1.20 |
| 3 | 960.0 | 46080.0 | 1.20 |
| 4 | 864.0 | 93312.0 | 1.20 |
| 5 | 1056.0 | 50688.0 | 1.20 |

BAY WIDTHS ------

1 BAYS AT 432.000 INCHES
1 BAYS AT 288.000 INCHES

STORY HEIGHTS -------

11 STORIES AT 156.000 INCHES
1 STORIES AT 192.000 INCHES

```
-- FRAME NO.  6 --  FRAME MODELING OPTION=     2

 NUMBER OF BAYS  =     2

 BEAMS:     E=    2100.0  G=     840.0  WEIGHTS/CU.FT)=    110.00

 COLUMNS:   E=    3500.0  G=    1560.0  WEIGHTS/CU.FT)=    150.00

MEAN TYPE          AM          IM          FM

        1           .0          .0        1.20
        2        729.0     442866.8       1.20
        3        486.0      29524.5       1.20

COLUMN TYPE        AC          IC          FC

        1           .0          .0        1.20
        2        648.0     649964.0       1.20
        3        960.0     460860.0       1.20
        4        864.0      93312.0       1.20
        5       1056.0     506888.0       1.20

BAY WIDTHS ------

    1 BAYS AT    432.000 INCHES
    1 BAYS AT    288.000 INCHES

STORY HEIGHTS -------

   11 STORIES AT    156.000 INCHES
    1 STORIES AT    192.000 INCHES
```

```
-- FRAME NO.  6 --  FRAME MODELING OPTION=    2

NUMBER OF BAYS  =    2

BEAMS:     E=   2100.0  G=    840.0  W(LBS/CU.FT)=   110.00

COLUMNS:   E=   3900.0  G=   1560.0  W(LBS/CU.FT)=   150.00

BEAM TYPE          AB          IB          FB

        1           .0          .0        1.20
        2         729.0     44280.8       1.20
        3         486.0     24524.5       1.20

COLUMN TYPE        AC          IC          FC

        1           .0          .0        1.20
        2         648.0     69984.0       1.20
        3         960.0     46080.0       1.20
        4         864.0     93312.0       1.20
        5        1056.0     50688.0       1.20

BAY WIDTHS ------

    1 BAYS AT    432.000 INCHES
    1 BAYS AT    288.000 INCHES

STORY HEIGHTS -------

   11 STORIES AT    156.000 INCHES
    1 STORIES AT    192.000 INCHES
```

-- FRAME NO. 7 -- FRAME MODELING OPTION= 2

NUMBER OF BAYS = 2

BEAMS: E= 2100.0 G= 840.0 W(LBS/CU.FT)= 110.00

COLUMNS: E= 3900.0 G= 1560.0 W(LBS/CU.FT)= 150.00

| BEAM TYPE | AB | IB | FB |
|---|---|---|---|
| 1 | .0 | .0 | 1.20 |
| 2 | 729.0 | 44296.8 | 1.20 |
| 3 | 486.0 | 29524.5 | 1.20 |

| COLUMN TYPE | AC | IC | FC |
|---|---|---|---|
| 1 | .0 | .0 | 1.20 |
| 2 | 648.0 | 69984.0 | 1.20 |
| 3 | 540.0 | 46080.0 | 1.20 |
| 4 | 864.0 | 93312.0 | 1.20 |
| 5 | 1058.0 | 506888.0 | 1.20 |

BAY WIDTHS ------

1 BAYS AT 432.000 INCHES
1 BAYS AT 288.000 INCHES

STORY HEIGHTS -------

11 STORIES AT 156.000 INCHES
1 STORIES AT 192.000 INCHES

STIFFNESS MATRIX (KIPS/IN)

| STORY NO. | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 12 | 2.87E+03 | -3.63E+03 | 7.08E+02 | -3.90E+01 | 2.10E+01 | 1.27E+01 | 1.14E+01 | 9.94E+00 | 8.91E+00 | 8.12E+00 |
| 11 | -3.63E+03 | 7.30E+03 | -4.27E+03 | 6.77E+02 | -6.35E+01 | 1.10E+00 | -4.11E+00 | -3.04E+00 | -2.72E+00 | -2.43E+00 |
| 10 | 7.08E+02 | -4.27E+03 | 7.09E+03 | -4.16E+03 | 6.77E+02 | -5.57E+01 | 6.73E+00 | 6.52E-01 | 1.01E+00 | 8.11E-01 |
| 9 | -3.90E+01 | 6.77E+02 | -4.16E+03 | 7.06E+03 | -4.16E+03 | 6.76E+02 | -5.64E+01 | 6.13E+00 | 1.56E-01 | 5.91E-01 |
| 8 | 2.10E+01 | -6.35E+01 | 6.77E+02 | -4.16E+03 | 7.06E+03 | -4.16E+03 | 6.76E+02 | -5.62E+01 | 6.30E+00 | 3.13E-01 |
| 7 | 1.27E+01 | 1.10E+00 | -5.57E+01 | 6.76E+02 | -4.16E+03 | 7.06E+03 | -4.16E+03 | 6.76E+02 | -5.61E+01 | 6.40E+00 |
| 6 | 1.14E+01 | -4.11E+00 | 6.73E+00 | -5.64E+01 | 6.76E+02 | -4.16E+03 | 7.06E+03 | -4.16E+03 | 6.77E+02 | -5.60E+01 |
| 5 | 9.94E+00 | -3.04E+00 | 6.52E-01 | 6.13E+00 | -5.62E+01 | 6.76E+02 | -4.16E+03 | 7.06E+03 | -4.16E+03 | 6.77E+02 |
| 4 | 8.91E+00 | -2.72E+00 | 1.01E+00 | 1.56E-01 | 6.30E+00 | -5.61E+01 | 6.77E+02 | -4.16E+03 | 7.06E+03 | -4.16E+03 |
| 3 | 8.12E+00 | -2.43E+00 | 8.11E-01 | 5.91E-01 | 3.13E-01 | 6.40E+00 | -5.60E+01 | 6.77E+02 | -4.16E+03 | 7.06E+03 |
| 2 | 7.93E+00 | -2.35E+00 | 7.60E-01 | 4.83E-01 | 7.90E-01 | 4.92E-01 | 6.62E+00 | -5.57E+01 | 6.77E+02 | -4.15E+03 |
| 1 | 9.28E+00 | -2.70E+00 | 8.35E-01 | 5.17E-01 | 7.79E-01 | 1.10E+00 | 9.03E-01 | 7.08E+00 | -5.43E+01 | 6.69E+02 |

| | | |
|---|---|---|
| 12 | 7.93E+00 | 9.28E+00 |
| 11 | -2.35E+00 | -2.70E+00 |
| 10 | 7.60E-01 | 8.35E-01 |
| 9 | 4.83E-01 | 5.17E-01 |
| 8 | 7.90E-01 | 7.79E-01 |
| 7 | 4.92E-01 | 1.10E+00 |
| 6 | 6.62E+00 | 9.03E-01 |
| 5 | -5.57E+01 | 7.08E+00 |
| 4 | 6.77E+02 | -5.43E+01 |
| 3 | -4.15E+03 | 6.69E+02 |
| 2 | 9.82E+03 | -8.23E+03 |
| 1 | -8.23E+03 | 1.53E+04 |

TOTAL WEIGHT VECTOR & DRIFT TO YIELD

| STORY NO. | STORY WT. (KIPS) | DRIFT TO YIELD |
|---|---|---|
| 12 | 1954.43 | .0052 |
| 11 | 1518.85 | .0052 |
| 10 | 1522.85 | .0057 |
| 9 | 1526.85 | .0052 |
| 8 | 1531.85 | .0057 |
| 7 | 1536.85 | .0052 |
| 6 | 1541.85 | .0057 |
| 5 | 1547.85 | .0052 |
| 4 | 1553.85 | .0052 |
| 3 | 1557.85 | .0052 |
| 2 | 1892.39 | .0052 |
| 1 | 2343.42 | .0057 |

CHARACTERISTIC VECTORS

| STORY NO. | MODE NO. 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| 12 | 1.00E+00 | -9.67E-01 | 8.56E-01 | -7.71E-01 | 6.77E-01 | 5.84E-01 |
| 11 | 9.59E-01 | -7.3[illegible]E-01 | 3.47E-01 | -4.11E-02 | -2.99E-01 | -6.10E-01 |
| 10 | 9.05E-01 | -4.02E-01 | -2.2[illegible]E-01 | 7.49E-01 | -9.[illegible]0E-01 | -8.87E-01 |
| 9 | 8.37E-01 | -1.66E-02 | -7.56E-01 | 9.89E-01 | -5.[illegible]4E-01 | 2.99E-01 |
| 8 | 7.56E-01 | 3.66E-01 | -9.7[illegible]E-01 | 5.07E-01 | 5.27E-01 | 1.00E+00 |
| 7 | 6.65E-01 | 6.92E-01 | -[illegible].08E-01 | -3.46E-01 | 1.00E+00 | 6.33E-02 |
| 6 | 5.65E-01 | 9.14E-01 | -3.12E-01 | -9.51E-01 | 3.32E-01 | -9.73E-01 |
| 5 | 4.57E-01 | 1.00E+00 | 3.12E-01 | -6.64E-01 | -7.12E-01 | -4.01E-01 |
| 4 | 3.44E-01 | 9.3[illegible]E-01 | 8.16E-01 | -1.48E-01 | -9.33E-01 | 8.3[illegible]E-01 |
| 3 | 2.30E-01 | 7.41E-01 | 1.00E+00 | 6.75E-01 | -7.53E-02 | 6.91E-01 |
| 2 | 1.23E-01 | 4.5[illegible]E-01 | 8.10E-01 | 1.00E+00 | 8.39E-01 | -5.44E-01 |
| 1 | 5.67E-02 | 2.22E-01 | 4.29E-01 | 6.06E-01 | 8.23E-01 | -5.70E-01 |

E A R T H Q U A K E  R E S P O N S E  A N A L Y S I S

DAMPING CURVE OPTION FOR STEEL AND RC FRAME STRUCTURES

MAXIMUM NUMBER ITERATIONS= 6
MODIFY RESPONSE FOR DUCTILITY
MODAL COMBINATION OPTION = RSS

EFFECTIVE INTERSTORY DRIFT TO YIELD BY STORY (IN/IN)

5.20E-03 5.20E-03 5.20E-03 5.20E-03 5.20E-03 5.20E-03 5.20E-03 5.20E-03 5.20E-03 5.20E-03
5.20E-03 5.20E-03

CRITICAL DAMPING (PERCENT) = 6.62

MODAL RESPONSE

| MODE NO. | FREQUENCY (CPS) | PERIOD (SEC.) | MASS RATIO | PSUDO-SPECTRAL VELOCITY | MAXIMUM BASE SHEAR | AMPLITUDE RATIO | PS.SPECT.VEL. MODIFIED FOR DUCTILITY (ACCEL) | (DISPL) |
|---|---|---|---|---|---|---|---|---|
| 1 | .4741 | 2.358 | .735 | 14.855 | 1508.652 | .753 | 14.855 | 14.855 |
| 2 | 1.3057 | .766 | .129 | 11.744 | 645.107 | 1.907 | 11.744 | 11.744 |
| 3 | 2.2607 | .442 | .060 | 6.925 | 304.588 | 2.841 | 6.925 | 6.925 |
| 4 | 3.1874 | .314 | .035 | 4.974 | 179.626 | 3.814 | 4.974 | 4.974 |
| 5 | 4.1853 | .239 | .016 | 3.826 | 85.569 | 5.375 | 3.826 | 3.826 |
| 6 | 5.2830 | .189 | .007 | 2.883 | 36.991 | -7.747 | 2.883 | 2.883 |

TOTAL RESPONSE

| STORY NO. | I.S. DEFL (IN) | VELOCITY (IN/SEC) | ACCELERATION (G) | I.S. DRIFT (IN/IN) |
|---|---|---|---|---|
| 12 | 3.6087E+01 | 2.0755E+01 | 2.1102E-01 | 2.3133E-03 |
| 11 | 4.9034E+01 | 1.9498E+01 | 1.6785E-01 | 3.1435E-03 |
| 10 | 5.8874E+01 | 1.8083E+01 | 1.5497E-01 | 3.7749E-03 |
| 9 | 6.6377E+01 | 1.6675E+01 | .5104E-01 | 4.2544E-03 |
| 8 | 7.2218E+01 | 1.5311E+01 | 1.5306E-01 | 4.6294E-03 |
| 7 | 7.6905E+01 | 1.3968E+01 | 1.5660E-01 | 4.9298E-03 |
| 6 | 8.0635E+01 | 1.2577E+01 | 1.6225E-01 | 5.1818E-03 |
| 5 | 8.4168E+01 | 1.1018E+01 | 1.6286E-01 | 5.3953E-03 |
| 4 | 8.6294E+01 | 9.1708E+00 | 1.5956E-01 | 5.5320E-03 |
| 3 | 8.2190E+01 | 6.9401E+00 | 1.4480E-01 | 5.2679E-03 |
| 2 | 5.2568E+01 | 4.4462E+00 | 1.2500E-01 | 3.3697E-03 |
| 1 | 4.5487E+01 | 2.2560E+00 | 7.2836E-02 | 2.3451E-03 |

DUCTILITY BY STORY

4.45E-01 6.05E-01 7.26E-01 8.18E-01 8.90E-01 9.48E-01 9.96E-01 1.04E+00 1.06E+00 1.01E+00
6.48E-01 4.61E-01

EFFECTIVE DUCTILITY FOR BUILDING = .77

CRITICAL DAMPING (PERCENT) = 5.73

NODAL RESPONSE

| MODE NO. | FREQUENCY (CPS) | PERIOD (SEC.) | MASS RATIO | PSUDO-SPECTRAL VELOCITY | MAXIMUM BASE SHEAR | AMPLITUDE RATIO | PS.SPECT.VEL. MODIFIED FOR DUCTILITY (ACCEL) | (DISPL) |
|---|---|---|---|---|---|---|---|---|
| 1 | .4241 | 2.356 | .735 | 15.271 | 1545.452 | .753 | 15.221 | 15.221 |
| 2 | 1.3957 | .766 | .129 | 12.021 | 660.378 | 1.907 | 12.021 | 12.021 |
| 3 | 2.2607 | .442 | .060 | 7.085 | 311.628 | 2.841 | 7.085 | 7.085 |
| 4 | 3.1874 | .314 | .035 | 5.088 | 183.725 | 3.814 | 5.088 | 5.088 |
| 5 | 4.1854 | .239 | .016 | 3.913 | 87.502 | 5.375 | 3.913 | 3.913 |
| 6 | 5.2831 | .189 | .007 | 2.948 | 37.822 | -7.747 | 2.948 | 2.948 |

TOTAL RESPONSE

| STORY NO. | I.S. DEFL (IN) | VELOCITY (IN/SEC) | ACCELERATION (G) | I.S. DRIFT (IN/IN) |
|---|---|---|---|---|
| 12 | 3.6963E-01 | 2.1264E+01 | 2.1606E-01 | 2.3694E-03 |
| 11 | 5.0224E-01 | 1.9977E+01 | 1.7191E-01 | 3.2198E-03 |
| 10 | 6.0306E-01 | 1.8528E+01 | 1.5870E-01 | 3.6658E-03 |
| 9 | 6.7999E-01 | 1.7085E+01 | 1.5466E-01 | 4.3589E-03 |
| 8 | 7.3988E-01 | 1.5687E+01 | 1.5671E-01 | 4.7428E-03 |
| 7 | 7.8794E-01 | 1.4310E+01 | 1.6032E-01 | 5.0509E-03 |
| 6 | 8.2824E-01 | 1.2884E+01 | 1.6609E-01 | 5.3093E-03 |
| 5 | 8.6239E-01 | 1.1285E+01 | 1.6670E-01 | 5.5281E-03 |
| 4 | 8.8422E-01 | 9.3922E+00 | 1.6331E-01 | 5.6800E-03 |
| 3 | 8.4198E-01 | 7.1065E+00 | 1.6118E-01 | 5.3973E-03 |
| 2 | 5.3856E-01 | 4.5517E+00 | 1.2788E-01 | 3.4523E-03 |
| 1 | 4.7111E-01 | 2.3093E+00 | 7.4512E-02 | 2.4537E-03 |

DUCTILITY BY STORY

4.56E-01 6.19E-01 7.43E-01 8.38E-01 9.12E-01 9.71E-01 1.02E+00 1.06E+00 1.09E+00 1.04E+00
6.64E-01 4.77E-01

EFFECTIVE DUCTILITY FOR BUILDING = .79

CRITICAL DAMPING (PERCENT) = 5.81

COMBINED FORCE FOR ALL MODES (METHOD RSS )

| STORY | FORCE(KIPS) | SHEAR(KIPS) |
|---|---|---|
| 12 | 422.281 | 422.281 |
| 11 | 245.264 | 667.545 |
| 10 | 176.289 | 843.834 |
| 9 | 139.289 | 983.123 |
| 8 | 114.332 | 1099.455 |
| 7 | 100.624 | 1200.079 |
| 6 | 95.893 | 1295.972 |
| 5 | 96.207 | 1392.179 |
| 4 | 95.110 | 1487.290 |
| 3 | 85.674 | 1572.964 |
| 2 | 81.486 | 1654.450 |
| 1 | 67.656 | 1722.105 |

W I N D  R E S P O N S E  A N A L Y S I S

BUILDING SHAPE CODE =  25  WIND DIRECTION CODE =  1

```
EXTERNAL PRESSURE COEFFICIENT FOR WALL 1 . . .=    .800
EXTERNAL PRESSURE COEFFICIENT FOR WALL 2 . . .=   -.500
SHAPE FACTOR FOR WIND DIRECTION . . .=   1.300

WIND DIRECTION WITH RESPECT TO
NORMAL OF WALL 1 . . . . . . . . . . . . =   0.0 DEGREES


  OPEN AREA RATIO =     .500     WALL CODE =     1
```

WIND ANALYSIS

| STORY NO. | DEFLECTION (INCHES) | DRIFT (IN/IN) | STORY SHEAR (KIPS) |
|---|---|---|---|
| 1 | .922 | .0002 | 13.89 |
| 2 | .895 | .0003 | 40.74 |
| 3 | .855 | .0003 | 66.48 |
| 4 | .801 | .0004 | 90.91 |
| 5 | .734 | .0005 | 114.00 |
| 6 | .656 | .0006 | 135.68 |
| 7 | .566 | .0006 | 155.87 |
| 8 | .466 | .0007 | 174.43 |
| 9 | .358 | .0007 | 191.22 |
| 10 | .244 | .0007 | 206.01 |
| 11 | .134 | .0005 | 218.46 |
| 12 | .063 | .0003 | 228.07 |

D A M A G E  A N A L Y S I S

DAMAGEABILITY INPUTS FOR EARTHQUAKE WIND, OR TORNADO

| STORY NO. | DRIFT TO YIELD(IN/IN) FRAME | DRIFT TO YIELD(IN/IN) WALL | DRIFT TO FAILURE(IN/IN) FRAME | DRIFT TO FAILURE(IN/IN) WALL | QUALITY FACTOR FOR NONSTRUCT. DAMAGE | WINDOW HT.(IN) | WINDOW WIDTH(IN) | THICKNESS OF WINDOW GLASS(IN) |
|---|---|---|---|---|---|---|---|---|
| 12 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 11 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 10 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 9 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 8 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 7 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 6 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 5 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 4 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 3 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 2 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |
| 1 | .0052 | 0.0000 | .0500 | 0.0000 | 2.00 | 78.00 | 40.00 | .20 |

TOTAL EARTHQUAKE DAMAGE TO BUILDING (PERCENT OF REPAIR COST/ REPLACEMENT COST)

BUILDING CLASSIFICATION: REINFORCED CONCRETE FRAME

OVERALL QUALITY RATING: AVERAGE

| TOTAL NO. STY. | AVERAGE INTENSTY. (FRIF) | BUILDING PERCENT DAMAGE |
|---|---|---|
| 12 | .0043 | 3.14 |

EARTHQUAKE DAMAGE TO STRUCTURAL COMPONENTS (PERCENT OF REPAIR COST/ REPLACEMENT COST)

| | FRAME TYPE: 2 | | | | SHEARWALL TYPE: 0 | | | |
|---|---|---|---|---|---|---|---|---|
| STORY NO. | INTENSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | INTENSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR |
| 12 | .0024 | .77 | 1.48 | 5.25 | .0024 | 0.00 | 0.00 | 0.00 |
| 11 | .0032 | 1.07 | 2.19 | 8.60 | .0032 | 0.00 | 0.00 | 0.00 |
| 10 | .0039 | 1.30 | 2.75 | 11.54 | .0039 | 0.00 | 0.00 | 0.00 |
| 9 | .0044 | 1.46 | 3.20 | 14.00 | .0044 | 0.00 | 0.00 | 0.00 |
| 8 | .0047 | 1.63 | 3.56 | 16.04 | .0047 | 0.00 | 0.00 | 0.00 |
| 7 | .0051 | 1.74 | 3.86 | 17.75 | .0051 | 0.00 | 0.00 | 0.00 |
| 6 | .0053 | 1.84 | 4.11 | 19.24 | .0053 | 0.00 | 0.00 | 0.00 |
| 5 | .0055 | 1.92 | 4.32 | 20.53 | .0055 | 0.00 | 0.00 | 0.00 |
| 4 | .0057 | 1.97 | 4.46 | 21.37 | .0057 | 0.00 | 0.00 | 0.00 |
| 3 | .0054 | 1.87 | 4.20 | 19.75 | .0054 | 0.00 | 0.00 | 0.00 |
| 2 | .0035 | 1.15 | 2.39 | 9.62 | .0035 | 0.00 | 0.00 | 0.00 |
| 1 | .0025 | .80 | 1.55 | 5.55 | .0025 | 0.00 | 0.00 | 0.00 |

EARTHQUAKE DAMAGE TO NONSTRUCTURAL COMPONENTS (PERCENT OF COMPONENTS DAMAGED)

| | COMPONENTS SENSITIVE TO FLOOR MOTION | | | | GLASS | | | |
|---|---|---|---|---|---|---|---|---|
| STORY NO. | FLOOR ACCL.(G) | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | INTENSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR |
| 12 | .22 | 1.69 | 10.89 | 70.04 | .0024 | 1.04 | 2.51 | 7.13 |
| 11 | .17 | 1.54 | 9.76 | 61.79 | .0032 | 2.11 | 4.45 | 11.74 |
| 10 | .16 | 1.38 | 8.55 | 53.11 | .0039 | 3.22 | 6.27 | 15.82 |
| 9 | .15 | 1.22 | 7.42 | 45.13 | .0044 | 4.26 | 7.85 | 19.23 |
| 8 | .16 | 1.07 | 6.38 | 38.01 | .0047 | 5.18 | 9.19 | 22.06 |
| 7 | .16 | .93 | 5.43 | 31.61 | .0051 | 5.98 | 10.34 | 24.44 |
| 6 | .17 | .80 | 4.51 | 25.60 | .0053 | 6.73 | 11.35 | 26.51 |
| 5 | .17 | .65 | 3.58 | 19.61 | .0055 | 7.39 | 12.24 | 28.31 |
| 4 | .16 | .49 | 2.59 | 13.56 | .0057 | 7.83 | 12.92 | 29.49 |
| 3 | .15 | .37 | 1.59 | 7.74 | .0054 | 6.99 | 11.70 | 27.23 |
| 2 | .13 | .21 | .97 | 4.42 | .0035 | 2.48 | 5.07 | 13.16 |
| 1 | .07 | .08 | .29 | 1.10 | .0025 | 1.12 | 2.68 | 7.55 |

FRAME TYPES:

0: NOT APPLICABLE
1: STEEL
2: REINFORCED CONCRETE
3: PRECAST CONCRETE

SHEARWALL TYPES:

0: NOT APPLICABLE
1: REINFORCED CONCRETE
2: PRECAST CONCRETE
3: BRICK MASONRY

TOTAL WIND DAMAGE TO BUILDING (PERCENT OF REPAIR COST/ REPLACEMENT COST)

BUILDING CLASSIFICATION: REINFORCED CONCRETE FRAME

OVERALL QUALITY RATING: AVERAGE

| TOTAL NO. STY. | AVERAGE INTENSITY. DRIFT | BUILDING PERCENT DAMAGE |
|---|---|---|
| 12 | .0005 | .07 |

WIND DAMAGE TO STRUCTURAL COMPONENTS (PERCENT OF REPAIR COST/ REPLACEMENT COST)

| | FRAME TYPE: 2 | | | | SHEARWALL TYPE: 0 | | | |
|---|---|---|---|---|---|---|---|---|
| STORY NO. | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR |
| 12 | .0002 | 0.00 | 0.00 | 0.00 | .0002 | 0.00 | 0.00 | 0.00 |
| 11 | .0003 | 0.00 | 0.00 | 0.00 | .0003 | 0.00 | 0.00 | 0.00 |
| 10 | .0003 | 0.00 | 0.00 | 0.00 | .0003 | 0.00 | 0.00 | 0.00 |
| 9 | .0004 | 0.00 | 0.00 | 0.00 | .0004 | 0.00 | 0.00 | 0.00 |
| 8 | .0005 | 0.00 | 0.00 | 0.00 | .0005 | 0.00 | 0.00 | 0.00 |
| 7 | .0006 | 0.00 | 0.00 | .57 | .0006 | 0.00 | 0.00 | 0.00 |
| 6 | .0006 | 0.00 | 0.00 | .76 | .0006 | 0.00 | 0.00 | 0.00 |
| 5 | .0007 | 0.00 | 0.00 | .95 | .0007 | 0.00 | 0.00 | 0.00 |
| 4 | .0007 | 0.00 | 0.00 | 1.09 | .0007 | 0.00 | 0.00 | 0.00 |
| 3 | .0007 | 0.00 | 0.00 | 1.00 | .0007 | 0.00 | 0.00 | 0.00 |
| 2 | .0005 | 0.00 | 0.00 | 0.00 | .0005 | 0.00 | 0.00 | 0.00 |
| 1 | .0003 | 0.00 | 0.00 | 0.00 | .0003 | 0.00 | 0.00 | 0.00 |

WIND DAMAGE TO NONSTRUCTURAL COMPONENTS (PERCENT OF COMPONENTS DAMAGED)

| | CEILINGS AND PARTITIONS | | | | GLASS (PERCENT DAMAGE BY WALL) | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| STORY NO. | INTERSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | WIND PRESS (PSF) | WALL 1 | WALL 2 | WALL 3 | WALL 4 | CORNERS |
| 12 | .0002 | .13 | .22 | 1.78 | 27.68 | 16.67 | 1.77 | 8.91 | 8.91 | 99.98 |
| 11 | .0003 | .27 | .36 | 2.56 | 26.54 | 13.72 | 1.45 | 7.27 | 7.27 | 99.95 |
| 10 | .0003 | .34 | .55 | 3.44 | 25.34 | 11.05 | 1.18 | 5.81 | 5.81 | 99.84 |
| 9 | .0004 | .46 | .75 | 4.27 | 24.09 | 8.68 | .98 | 4.55 | 4.55 | 99.53 |
| 8 | .0005 | .58 | .95 | 5.05 | 22.78 | 6.63 | .73 | 3.47 | 3.47 | 98.74 |
| 7 | .0006 | .69 | 1.15 | 5.76 | 21.40 | 4.89 | .56 | 2.56 | 2.56 | 96.85 |
| 6 | .0006 | .80 | 1.33 | 6.39 | 19.92 | 3.45 | .42 | 1.83 | 1.83 | 92.72 |
| 5 | .0007 | .90 | 1.48 | 6.92 | 18.33 | 2.32 | .30 | 1.24 | 1.24 | 84.58 |
| 4 | .0007 | .96 | 1.60 | 7.29 | 16.58 | 1.65 | .21 | .80 | .80 | 70.58 |
| 3 | .0007 | .92 | 1.53 | 7.07 | 14.63 | .83 | .14 | .47 | .47 | 50.25 |
| 2 | .0005 | .50 | .82 | 4.57 | 12.35 | .41 | .08 | .24 | .24 | 26.87 |
| 1 | .0003 | .31 | .51 | 3.26 | 9.49 | .15 | .04 | .10 | .10 | 8.04 |

FRAME TYPES:

0: NOT APPLICABLE
1: STEEL
2: REINFORCED CONCRETE
3: PRECAST CONCRETE

SHEARWALL TYPES:

0: NOT APPLICABLE
1: REINFORCED CONCRETE
2: PRECAST CONCRETE
3: BRICK MASONRY

END OF JOB.

EXAMPLE 2: STEEL FRAME BUILDING

```
RUN IDENTIFICATION:
          TYPICAL STEEL FRAME BUILDING IN LONG BEACH
EARTHQUAKE HAZARD IS INCLUDED
SITE LOCATION: LATITUDE    34     DEGREES, LONGITUDE  118.10  DEGREES
BUILDING MODELING OPTION: DETAILED FRAME
```

E A R T H Q U A K E  L O A D  A N A L Y S I S

SITE GROUND MOTION CHARACTERISTICS

| | | |
|---|---|---|
| RICHTER MAGNITUDE | = | 6.50E+00 |
| HYPOCENTRAL DISTANCE (MILES) | = | 1.12E+01 |
| SITE ACCELERATION (G) | = | 2.95E-01 |
| SITE VELOCITY (IN/SEC) | = | 1.46E+01 |
| SITE DISPLACEMENT (IN) | = | 4.26E+00 |

SITE GROUND MOTION SPECTRUM

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| PERIOD (SEC) | .10 | .20 | .30 | .40 | .50 | .60 | .70 | .80 | .90 | 1.00 |
| VEL (IN/SEC) | 1.81 | 3.62 | 5.43 | 7.24 | 9.05 | 10.86 | 12.67 | 14.48 | 14.63 | 14.63 |
| PERIOD (SEC) | 1.10 | 1.20 | 1.30 | 1.40 | 1.50 | 1.60 | 1.70 | 1.80 | 1.90 | 2.00 |
| VEL (IN/SEC) | 14.63 | 14.63 | 14.63 | 14.63 | 14.63 | 14.63 | 14.63 | 14.63 | 14.15 | 13.45 |
| PERIOD (SEC) | 2.10 | 2.20 | 2.30 | 2.40 | 2.50 | 2.60 | 2.70 | 2.80 | 2.90 | 3.00 |
| VEL (IN/SEC) | 12.81 | 12.22 | 11.69 | 11.21 | 10.76 | 10.34 | 9.96 | 9.60 | 9.27 | 8.96 |
| PERIOD (SEC) | 3.10 | 3.20 | 3.30 | 3.40 | 3.50 | 3.60 | 3.70 | 3.80 | 3.90 | 4.00 |
| VEL (IN/SEC) | 8.67 | 8.40 | 8.15 | 7.91 | 7.68 | 7.47 | 7.27 | 7.08 | 6.90 | 6.72 |
| PERIOD (SEC) | 4.10 | 4.20 | 4.30 | 4.40 | 4.50 | 4.60 | 4.70 | 4.80 | 4.90 | 5.00 |
| VEL (IN/SEC) | 6.56 | 6.40 | 6.25 | 6.11 | 5.98 | 5.85 | 5.72 | 5.60 | 5.49 | 5.38 |
| PERIOD (SEC) | 5.10 | 5.20 | 5.30 | 5.40 | 5.50 | 5.60 | 5.70 | 5.80 | 5.90 | 6.00 |
| VEL (IN/SEC) | 5.27 | 5.17 | 5.07 | 4.98 | 4.89 | 4.80 | 4.72 | 4.64 | 4.56 | 4.48 |
| PERIOD (SEC) | 6.10 | 6.20 | 6.30 | 6.40 | 6.50 | 6.60 | 6.70 | 6.80 | 6.90 | 7.00 |
| VEL (IN/SEC) | 4.41 | 4.34 | 4.27 | 4.20 | 4.14 | 4.07 | 4.01 | 3.95 | 3.90 | 3.84 |
| PERIOD (SEC) | 7.10 | 7.20 | 7.30 | 7.40 | 7.50 | 7.60 | 7.70 | 7.80 | 7.90 | 8.00 |
| VEL (IN/SEC) | 3.79 | 3.74 | 3.68 | 3.63 | 3.59 | 3.54 | 3.49 | 3.45 | 3.40 | 3.36 |
| PERIOD (SEC) | 8.10 | 8.20 | 8.30 | 8.40 | 8.50 | 8.60 | 8.70 | 8.80 | 8.90 | 9.00 |
| VEL (IN/SEC) | 3.32 | 3.28 | 3.24 | 3.20 | 3.16 | 3.13 | 3.09 | 3.06 | 3.02 | 2.99 |
| PERIOD (SEC) | 9.10 | 9.20 | 9.30 | 9.40 | 9.50 | 9.60 | 9.70 | 9.80 | 9.90 | 10.00 |
| VEL (IN/SEC) | 2.96 | 2.92 | 2.89 | 2.86 | 2.83 | 2.80 | 2.77 | 2.74 | 2.72 | 2.69 |

S T R U C T U R A L  A N A L Y S I S

```
NUMBER OF STORIES =     10
TOTAL BUILDING WIDTH (INCHES) =  1035.0
TOTAL BUILDING LENGTH (INCHES) =  1035.0
HEIGHT OF PARAPET (INCHES)    =   186.0

NUMBER OF FRAMES  =      2
```

```
-- FRAME NO.  1 --  FRAME MODELING OPTION=    1

NUMBER OF BAYS          =    3
NUMBER OF BRACED BAYS   =    0
YOUNGS MODULUS          =  29000.0   (KIPS/SQ IN)

BEAM TYPE          AB          IB          WB
       39          20.0      1820.0      .006
       49          24.8      2810.0      .007
       50          27.7      3270.0      .008
       56          29.1      4000.0      .009
       71          39.6      7820.0      .011

COLUMN TYPE        AC          IC          WC
        9          32.7      1270.0      .009
       15          46.5      1900.0      .013
       19          56.7      2400.0      .016
       21          62.1      2670.0      .018
       24          69.7      3080.0      .020
       33         134.0      7220.0      .038
       41          45.8      1777.6      .013
       42          60.0      2260.0      .017
       43          73.9      2693.4      .021
       44          87.0      3081.2      .025
       45          99.8      3126.8      .028

BAY WIDTHS ------

    3 BAYS AT    345.000 INCHES

STORY HEIGHTS -------

    1 STORIES AT    174.000 INCHES
    8 STORIES AT    162.000 INCHES
    1 STORIES AT    312.000 INCHES

STORY NO.   10 -------STRUCTURE DATA

BEAM     AREA        IZ        W(DL)       A         H     SCJ      SCR

   1     20.0     1820.0    .00567     8.00      7.00   0.        0.
   2     20.0     1820.0    .00567     8.00      7.00   0.        0.
   3     20.0     1820.0    .00567     8.00      7.00   0.        0.

COLUMN

   1     45.8     1777.6    .01297    12.00     12.00   0.        0.
   2     32.7     1270.0    .00925    12.00     12.00   0.        0.
   3     32.7     1270.0    .00925    12.00     12.00   0.        0.
   4     45.8     1777.6    .01297    12.00     12.00   0.        0.

TOTAL WEIGHT OF STORY FRAMING =          9.731 KIPS
```

STORY NO. 9 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 20.0 | 1820.0 | .00567 | 8.00 | 7.00 | 0. | 0. |
| 2 | 20.0 | 1820.0 | .00567 | 8.00 | 7.00 | 0. | 0. |
| 3 | 20.0 | 1820.0 | .00567 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 45.8 | 1777.6 | .01297 | 12.00 | 13.50 | 0. | 0. |
| 2 | 46.5 | 1900.0 | .01317 | 12.00 | 13.50 | 0. | 0. |
| 3 | 46.5 | 1900.0 | .01317 | 12.00 | 13.50 | 0. | 0. |
| 4 | 45.8 | 1777.6 | .01297 | 12.00 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 14.646 KIPS

STORY NO. 8 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 2 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 3 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 45.8 | 1777.6 | .01297 | 13.50 | 13.50 | 0. | 0. |
| 2 | 46.5 | 1900.0 | .01317 | 13.50 | 13.50 | 0. | 0. |
| 3 | 46.5 | 1900.0 | .01317 | 13.50 | 13.50 | 0. | 0. |
| 4 | 45.8 | 1777.6 | .01297 | 13.50 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 15.712 KIPS

STORY NO. 7 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 2 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 3 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 60.0 | 2260.0 | .01700 | 13.50 | 13.50 | 0. | 0. |
| 2 | 56.7 | 2400.0 | .01608 | 13.50 | 13.50 | 0. | 0. |
| 3 | 56.7 | 2400.0 | .01608 | 13.50 | 13.50 | 0. | 0. |
| 4 | 60.0 | 2260.0 | .01700 | 13.50 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 17.964 KIPS

STORY NO. 6 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 2 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 3 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 60.0 | 2260.0 | .01700 | 13.50 | 13.50 | 0. | 0. |
| 2 | 56.7 | 2400.0 | .01608 | 13.50 | 13.50 | 0. | 0. |
| 3 | 56.7 | 2400.0 | .01608 | 13.50 | 13.50 | 0. | 0. |
| 4 | 60.0 | 2260.0 | .01700 | 13.50 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 17.964 KIPS

STORY NO. 5 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |
| 2 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |
| 3 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 73.8 | 2693.4 | .02090 | 13.50 | 13.50 | 0. | 0. |
| 2 | 62.1 | 2670.0 | .01758 | 13.50 | 13.50 | 0. | 0. |
| 3 | 62.1 | 2670.0 | .01758 | 13.50 | 13.50 | 0. | 0. |
| 4 | 73.8 | 2693.4 | .02090 | 13.50 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 20.576 KIPS

STORY NO. 4 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |
| 2 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |
| 3 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 73.8 | 2693.4 | .02090 | 13.50 | 13.50 | 0. | 0. |
| 2 | 62.1 | 2670.0 | .01758 | 13.50 | 13.50 | 0. | 0. |
| 3 | 62.1 | 2670.0 | .01758 | 13.50 | 13.50 | 0. | 0. |
| 4 | 73.8 | 2693.4 | .02090 | 13.50 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 20.576 KIPS

STORY NO. 3 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| 2 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| 3 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 87.0 | 3081.2 | .02465 | 15.00 | 15.00 | 0. | 0. |
| 2 | 69.7 | 3080.0 | .01975 | 15.00 | 15.00 | 0. | 0. |
| 3 | 69.7 | 3080.0 | .01975 | 15.00 | 15.00 | 0. | 0. |
| 4 | 87.0 | 3081.2 | .02465 | 15.00 | 15.00 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 22.924 KIPS

STORY NO. 2 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| 2 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| 3 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 87.0 | 3081.2 | .02465 | 15.00 | 15.00 | 0. | 0. |
| 2 | 69.7 | 3080.0 | .01975 | 15.00 | 15.00 | 0. | 0. |
| 3 | 69.7 | 3080.0 | .01975 | 15.00 | 15.00 | 0. | 0. |
| 4 | 87.0 | 3081.2 | .02465 | 15.00 | 15.00 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 22.924 KIPS

STORY NO. 1 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 39.8 | 7820.0 | .01125 | 8.00 | 7.00 | 0. | 0. |
| 2 | 39.8 | 7820.0 | .01125 | 8.00 | 7.00 | 0. | 0. |
| 3 | 39.8 | 7820.0 | .01125 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 99.8 | 3426.8 | .02827 | 18.00 | .10 | 0. | 0. |
| 2 | 134.0 | 7220.0 | .03792 | 18.00 | .10 | 0. | 0. |
| 3 | 134.0 | 7220.0 | .03792 | 18.00 | .10 | 0. | 0. |
| 4 | 99.8 | 3426.8 | .02827 | 18.00 | .10 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 43.015 KIPS

-- FRAME NO. 2 -- FRAME MODELING OPTION= 1

NUMBER OF BAYS = 3

NUMBER OF BRACED BAYS = 0

YOUNGS MODULUS = 29000.0 (KIPS/SQ IN)

| BEAM TYPE | AB | IB | WB |
|---|---|---|---|
| 39 | 20.0 | 1820.0 | .006 |
| 49 | 24.8 | 2810.0 | .007 |
| 50 | 27.7 | 4270.0 | .008 |
| 56 | 29.1 | 4000.0 | .008 |
| 71 | 39.8 | 7820.0 | .011 |

| COLUMN TYPE | AC | IC | WC |
|---|---|---|---|
| 9 | 32.7 | 1270.0 | .009 |
| 15 | 46.5 | 1900.0 | .013 |
| 19 | 56.7 | 2400.0 | .016 |
| 21 | 62.1 | 2670.0 | .018 |
| 24 | 69.7 | 3080.0 | .020 |
| 33 | 134.0 | 7220.0 | .036 |
| 41 | 45.8 | 1777.6 | .013 |
| 42 | 60.0 | 2260.0 | .017 |
| 43 | 73.8 | 2693.4 | .021 |
| 44 | 87.6 | 3081.2 | .025 |
| 45 | 99.8 | 3426.6 | .028 |
| 46 | 123.8 | 4003.8 | .035 |

BAY WIDTHS ------

3 BAYS AT 345.000 INCHES

STORY HEIGHTS -------

1 STORIES AT 174.000 INCHES
8 STORIES AT 162.000 INCHES
1 STORIES AT 312.000 INCHES

STORY NO. 10 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 20.0 | 1820.0 | .00567 | 8.00 | 7.00 | 0. | 0. |
| 2 | 20.0 | 1820.0 | .00567 | 8.00 | 7.00 | 0. | 0. |
| 3 | 20.0 | 1820.0 | .00567 | 4.00 | 7.00 | 0. | 0. |

COLUMN

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 45.8 | 1777.6 | .01297 | 12.00 | 12.00 | 0. | 0. |
| 2 | 32.7 | 1270.0 | .00925 | 12.00 | 12.00 | 0. | 0. |
| 3 | 32.7 | 1270.0 | .00925 | 12.00 | 12.00 | 0. | 0. |
| 4 | 45.8 | 1777.6 | .01297 | 12.00 | 12.00 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 19.461 KIPS

STORY NO. 4 -------STRUCTURE DATA

| BEAM | AREA | IZ | w(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 20.0 | 1820.0 | .00567 | 8.00 | 7.00 | 0. | 0. |
| 2 | 20.0 | 1820.0 | .00567 | 8.00 | 7.00 | 0. | 0. |
| 3 | 20.0 | 1820.0 | .00567 | 8.00 | 7.00 | 0. | 0. |

COLUMN

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 45.6 | 1777.6 | .01297 | 12.00 | 13.50 | 0. | 0. |
| 2 | 46.5 | 1900.0 | .01317 | 12.00 | 13.50 | 0. | 0. |
| 3 | 46.5 | 1900.0 | .01317 | 12.00 | 13.50 | 0. | 0. |
| 4 | 45.6 | 1777.6 | .01297 | 12.00 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 29.292 KIPS

STORY NO. 8 -------STRUCTURE DATA

| BEAM | AREA | IZ | w(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 2 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 3 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |

COLUMN

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 45.6 | 1777.6 | .01297 | 13.50 | 13.50 | 0. | 0. |
| 2 | 46.5 | 1900.0 | .01317 | 13.50 | 13.50 | 0. | 0. |
| 3 | 46.5 | 1900.0 | .01317 | 13.50 | 13.50 | 0. | 0. |
| 4 | 45.6 | 1777.6 | .01297 | 13.50 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 31.424 KIPS

STORY NO. 7 -------STRUCTURE DATA

| BEAM | AREA | IZ | w(DL) | A | B | SCJ | SCK |
|---|---|---|---|---|---|---|---|
| 1 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 2 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 3 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |

COLUMN

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 60.0 | 2260.0 | .01700 | 13.50 | 13.50 | 0. | 0. |
| 2 | 56.7 | 2400.0 | .01608 | 13.50 | 13.50 | 0. | 0. |
| 3 | 56.7 | 2400.0 | .01608 | 13.50 | 13.50 | 0. | 0. |
| 4 | 60.0 | 2260.0 | .01700 | 13.50 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 35.928 KIPS

STORY NO. 6 -------STRUCTURE DATA

| BEAM | AREA | I2 | W(DL) | A | H | SCJ | SCR |
|---|---|---|---|---|---|---|---|
| 1 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 2 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |
| 3 | 24.8 | 2830.0 | .00700 | 8.00 | 7.00 | 0. | 0. |

COLUMN

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 60.0 | 2260.0 | .01700 | 13.50 | 13.50 | 0. | 0. |
| 2 | 56.7 | 2400.0 | .01608 | 13.50 | 13.50 | 0. | 0. |
| 3 | 56.7 | 2400.0 | .01608 | 13.50 | 13.50 | 0. | 0. |
| 4 | 60.0 | 2260.0 | .01700 | 13.50 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 35.928 KIPS

STORY NO. 5 -------STRUCTURE DATA

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |
| 2 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |
| 3 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |

COLUMN

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 73.8 | 2693.4 | .02090 | 13.50 | 13.50 | 0. | 0. |
| 2 | 62.1 | 2670.0 | .01758 | 13.50 | 13.50 | 0. | 0. |
| 3 | 62.1 | 2670.0 | .01758 | 13.50 | 13.50 | 0. | 0. |
| 4 | 73.8 | 2693.4 | .02090 | 13.50 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 41.152 KIPS

STORY NO. 4 -------STRUCTURE DATA

| BEAM | AREA | I2 | W(DL) | A | H | SCJ | SCR |
|---|---|---|---|---|---|---|---|
| 1 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |
| 2 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |
| 3 | 27.7 | 3270.0 | .00783 | 8.00 | 7.00 | 0. | 0. |

COLUMN

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 73.8 | 2693.4 | .02090 | 13.50 | 13.50 | 0. | 0. |
| 2 | 62.1 | 2670.0 | .01758 | 13.50 | 13.50 | 0. | 0. |
| 3 | 62.1 | 2670.0 | .01758 | 13.50 | 13.50 | 0. | 0. |
| 4 | 73.8 | 2693.4 | .02090 | 13.50 | 13.50 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 41.152 KIPS

STORY NO. 3 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCR |
|---|---|---|---|---|---|---|---|
| 1 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| 2 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| 3 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 87.0 | 3081.2 | .02465 | 15.00 | 15.00 | 0. | 0. |
| 2 | 69.7 | 3080.0 | .01975 | 15.00 | 15.00 | 0. | 0. |
| 3 | 69.7 | 3080.0 | .01975 | 15.00 | 15.00 | 0. | 0. |
| 4 | 87.0 | 3081.2 | .02465 | 15.00 | 15.00 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 45.849 KIPS

STORY NO. 2 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCR |
|---|---|---|---|---|---|---|---|
| 1 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| 2 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| 3 | 29.1 | 4000.0 | .00825 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 87.0 | 3081.2 | .02465 | 15.00 | 15.00 | 0. | 0. |
| 2 | 69.7 | 3080.0 | .01975 | 15.00 | 15.00 | 0. | 0. |
| 3 | 69.7 | 3080.0 | .01975 | 15.00 | 15.00 | 0. | 0. |
| 4 | 87.0 | 3081.2 | .02465 | 15.00 | 15.00 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 45.849 KIPS

STORY NO. 1 -------STRUCTURE DATA

| BEAM | AREA | IZ | W(DL) | A | B | SCJ | SCR |
|---|---|---|---|---|---|---|---|
| 1 | 39.8 | 7820.0 | .01125 | 8.00 | 7.00 | 0. | 0. |
| 2 | 39.8 | 7820.0 | .01125 | 8.00 | 7.00 | 0. | 0. |
| 3 | 39.8 | 7820.0 | .01125 | 8.00 | 7.00 | 0. | 0. |
| COLUMN | | | | | | | |
| 1 | 99.8 | 3426.8 | .02827 | 18.00 | .10 | 0. | 0. |
| 2 | 134.0 | 7220.0 | .03792 | 18.00 | .10 | 0. | 0. |
| 3 | 134.0 | 7220.0 | .03792 | 18.00 | .10 | 0. | 0. |
| 4 | 123.8 | 4003.8 | .03507 | 18.00 | .10 | 0. | 0. |

TOTAL WEIGHT OF STORY FRAMING = 67.641 KIPS

STIFFNESS MATRIX (KIPS/IN)

| STORY NO. | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 10 | 5.64E+02 | -8.33E+02 | 3.23E+02 | -7.16E+01 | 1.83E+01 | -4.27E+00 | 1.64E+00 | -6.40E-02 | 2.20E-01 | 1.14E+00 |
| 9 | -8.33E+02 | 2.11E+03 | -1.69E+03 | 5.03E+02 | -1.21E+02 | 2.95E+01 | -7.34E+00 | 1.92E+00 | -4.78E-01 | 1.74E-01 |
| 8 | 3.23E+02 | -1.69E+03 | 2.88E+03 | -2.02E+03 | 6.25E+02 | -1.48E+02 | 3.61E+01 | -8.93E+00 | 2.35E+00 | -2.10E-01 |
| 7 | -7.16E+01 | 5.03E+02 | -2.02E+03 | 3.34E+03 | -2.34E+03 | 7.24E+02 | -1.73E+02 | 4.22E+01 | -1.06E+01 | 2.41E+00 |
| 6 | 1.83E+01 | -1.21E+02 | 6.25E+02 | -2.34E+03 | 3.72E+03 | -2.53E+03 | 7.75E+02 | -1.85E+02 | 4.55E+01 | -9.73E+00 |
| 5 | -4.27E+00 | 2.95E+01 | -1.48E+02 | 7.24E+02 | -2.53E+03 | 3.98E+03 | -2.75E+03 | 8.58E+02 | -2.07E+02 | 4.47E+01 |
| 4 | 1.64E+00 | -7.34E+00 | 3.61E+01 | -1.73E+02 | 7.75E+02 | -2.75E+03 | 4.33E+03 | -2.95E+03 | 9.16E+02 | -1.92E+02 |
| 3 | -6.40E-02 | 1.92E+00 | -8.93E+00 | 4.22E+01 | -1.85E+02 | 8.58E+02 | -2.95E+03 | 4.72E+03 | -3.31E+03 | 9.09E+02 |
| 2 | 2.20E-01 | -4.78E-01 | 2.35E+00 | -1.06E+01 | 4.55E+01 | -2.07E+02 | 9.16E+02 | -3.31E+03 | 5.32E+03 | -3.01E+03 |
| 1 | 1.14E+00 | 1.74E-01 | -2.10E-01 | 2.41E+00 | -9.73E+00 | 4.47E+01 | -1.92E+02 | 9.09E+02 | -3.01E+03 | 2.97E+03 |

TOTAL WEIGHT VECTOR & DRIFT TO YIELD

| STORY NO. | STORY WT. (KIPS) | DRIFT TO YIELD |
|---|---|---|
| 10 | 1410.06 | .0077 |
| 9 | 1139.24 | .0077 |
| 8 | 1133.12 | .0077 |
| 7 | 1141.73 | .0077 |
| 6 | 1145.93 | .0077 |
| 5 | 1153.25 | .0077 |
| 4 | 1157.35 | .0077 |
| 3 | 1165.15 | .0077 |
| 2 | 1168.35 | .0077 |
| 1 | 2556.24 | .0077 |

CHARACTERISTIC VECTORS

| STORY NO. | MODE NO. 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| 10 | 1.00E+00 | 1.00E+00 | 1.00E+00 | -7.89E-01 | 5.84E-01 | -3.45E-01 |
| 9 | 9.44E-01 | 6.44E-01 | 1.50E-01 | 4.55E-01 | -4.67E-01 | 1.00E+00 |
| 8 | 8.90E-01 | 2.69E-01 | -5.45E-01 | 1.00E+00 | -7.29E-01 | -4.69E-02 |
| 7 | 8.02E-01 | -1.00E-01 | -9.09E-01 | 6.27E-01 | 4.60E-01 | -9.61E-01 |
| 6 | 7.12E-01 | -4.25E-01 | -8.87E-01 | -2.04E-01 | 1.00E+00 | 1.34E-03 |
| 5 | 6.12E-01 | -6.72E-01 | -5.38E-01 | -6.59E-01 | 2.83E-01 | 9.59E-01 |
| 4 | 5.09E-01 | -8.07E-01 | -2.36E-02 | -8.81E-01 | -7.36E-01 | 1.82E-01 |
| 3 | 4.04E-01 | -8.31E-01 | 4.71E-01 | -3.[illegible]1E-01 | -9.13E-01 | -4.86E-01 |
| 2 | 3.05E-01 | -7.54E-01 | 7.6[illegible]E-01 | 3.4[illegible]E-01 | -1.43E-01 | -5.04E-01 |
| 1 | 2.12E-01 | -5.95E-01 | 7.88E-01 | 7.04E-01 | 5.83E-01 | 4.35E-01 |

E A R T H Q U A K E  R E S P O N S E  A N A L Y S I S

DAMPING CURVE OPTION FOR STEEL AND RC FRAME STRUCTURES

MAXIMUM NUMBER ITERATIONS= 6
MODIFY RESPONSE FOR DUCTILITY
MODAL COMBINATION OPTION = RSS

EFFECTIVE INTERSTORY DRIFT TO YIELD BY STORY (IN/IN)

7.70E-03 7.70E-03 7.70E-03 7.70E-03 7.70E-03 7.70E-03 7.70E-03 7.70E-03 7.70E-03 7.70E-03

CRITICAL DAMPING (PERCENT) = 6.62

NODAL RESPONSE

| NODE NO. | FREQUENCY (CPS) | PERIOD (SEC.) | MASS RATIO | PSEUDO-SPECTRAL VELOCITY | MAXIMUM BASE SHEAR | AMPLITUDE RATIO | PS.SPECT.VEL. (ACCEL) | MODIFIED FOR DUCTILITY (DISPL) |
|---|---|---|---|---|---|---|---|---|
| 1 | .3314 | 3.017 | .818 | 14.172 | 872.536 | .732 | 14.172 | 14.172 |
| 2 | .8936 | 1.119 | .136 | 24.190 | 630.037 | -1.804 | 24.190 | 24.190 |
| 3 | 1.4789 | .676 | .035 | 20.628 | 226.334 | 3.770 | 20.628 | 20.628 |
| 4 | 2.1932 | .456 | .008 | 14.117 | 55.399 | 7.407 | 14.117 | 14.117 |
| 5 | 3.1118 | .321 | .002 | 10.080 | 14.734 | 14.380 | 10.080 | 10.080 |
| 6 | 4.2017 | .238 | .001 | 7.548 | 4.708 | 23.488 | 7.548 | 7.548 |

TOTAL RESPONSE

| STORY NO. | I.S. DEFL (IN) | VELOCITY (IN/SEC) | ACCELERATION (G) | I.S. DRIFT (IN/IN) |
|---|---|---|---|---|
| 10 | 1.1290E+00 | 2.4239E+01 | 2.6376E-01 | 6.4885E-03 |
| 9 | 1.1566E+00 | 2.0277E+01 | 1.6809E-01 | 7.1393E-03 |
| 8 | 1.1633E+00 | 1.7781E+01 | 1.4702E-01 | 7.1406E-03 |
| 7 | 1.1447E+00 | 1.5416E+01 | 1.5577E-01 | 7.0659E-03 |
| 6 | 1.1237E+00 | 1.5716E+01 | 1.6567E-01 | 6.9846E-03 |
| 5 | 1.0609E+00 | 1.5282E+01 | 1.7382E-01 | 6.5488E-03 |
| 4 | 1.0252E+00 | 1.4754E+01 | 1.7845E-01 | 6.3286E-03 |
| 3 | 9.5912E-01 | 1.3890E+01 | 1.8362E-01 | 5.9205E-03 |
| 2 | 9.4213E-01 | 1.2453E+01 | 1.8285E-01 | 5.8156E-03 |
| 1 | 2.4788E+00 | 1.0056E+01 | 1.6578E-01 | 7.9448E-03 |

DUCTILITY BY STORY

8.43E-01 9.27E-01 9.33E-01 9.18E-01 9.01E-01 8.50E-01 8.22E-01 7.69E-01 7.55E-01 1.03E+00

EFFECTIVE DUCTILITY FOR BUILDING = .89

CRITICAL DAMPING (PERCENT) = 6.21

MODAL RESPONSE

| MODE NO. | FREQUENCY (CPS) | PERIOD (SEC.) | MASS RATIO | PSEUDO-SPECTRAL VELOCITY | MAXIMUM BASE SHEAR | AMPLITUDE RATIO | PS.SPECT.VEL. MODIFIED FOR DUCTILITY (ACCEL) | (DISPL) |
|---|---|---|---|---|---|---|---|---|
| 1 | .9314 | 3.017 | .818 | 14.331 | 831.746 | .732 | 14.331 | 14.331 |
| 2 | .6936 | 1.119 | .136 | 24.450 | 636.817 | -1.804 | 24.450 | 24.450 |
| 3 | 1.4789 | .676 | .035 | 20.846 | 226.722 | 3.770 | 20.846 | 20.846 |
| 4 | 2.1932 | .456 | .008 | 14.264 | 55.975 | 7.407 | 14.264 | 14.264 |
| 5 | 3.1119 | .321 | .002 | 10.183 | 14.885 | 14.380 | 10.183 | 10.183 |
| 6 | 4.2017 | .238 | .001 | 7.624 | 4.756 | 23.988 | 7.624 | 7.624 |

TOTAL RESPONSE

| STORY NO. | I.S. DEFL (IN) | VELOCITY (IN/SEC) | ACCELERATION (G) | I.S. DRIFT (IN/IN) |
|---|---|---|---|---|
| 10 | 1.1412E+00 | 2.4506E+01 | 2.6660E-01 | 6.5585E-03 |
| 9 | 1.1691E+00 | 2.0503E+01 | 1.6492E-01 | 7.2167E-03 |
| 8 | 1.1760E+00 | 1.7979E+01 | 1.4860E-01 | 7.2590E-03 |
| 7 | 1.1572E+00 | 1.6598E+01 | 1.5744E-01 | 7.1435E-03 |
| 6 | 1.1361E+00 | 1.5890E+01 | 1.6745E-01 | 7.0124E-03 |
| 5 | 1.0727E+00 | 1.5450E+01 | 1.7568E-01 | 6.6214E-03 |
| 4 | 1.0366E+00 | 1.4906E+01 | 1.8037E-01 | 6.3990E-03 |
| 3 | 9.6966E-01 | 1.4041E+01 | 1.8560E-01 | 5.9864E-03 |
| 2 | 9.5259E-01 | 1.2586E+01 | 1.8481E-01 | 5.8802E-03 |
| 1 | 2.5061E+00 | 1.0164E+01 | 1.6755E-01 | 8.0323E-03 |

DUCTILITY BY STORY

8.52E-01 9.37E-01 9.43E-01 9.28E-01 9.11E-01 8.60E-01 8.31E-01 7.77E-01 7.66E-01 1.04E+00

EFFECTIVE DUCTILITY FOR BUILDING = .90

CRITICAL DAMPING (PERCENT) = 6.24

COMBINED FORCE FOR ALL MODES (METHOD RSS )

| STORY | FORCE(KIPS) | SHEAR(KIPS) |
|---|---|---|
| 10 | 375.926 | 375.926 |
| 9 | 165.546 | 541.472 |
| 8 | 80.609 | 622.081 |
| 7 | 37.378 | 659.459 |
| 6 | 20.306 | 679.765 |
| 5 | 19.171 | 698.885 |
| 4 | 30.013 | 728.918 |
| 3 | 48.861 | 777.779 |
| 2 | 71.548 | 849.327 |
| 1 | 224.464 | 1073.791 |

D A M A G E  A N A L Y S I S

DAMAGEABILITY INPUTS FOR EARTHQUAKE WIND, OR TORNADO

| STORY NO. | DRIFT TO YIELD(IN/IN) FRAME | WALL | DRIFT TO FAILURE(IN/IN) FRAME | WALL | QUALITY FACTOR FOR NONSTRUCT. DAMAGE | WINDOW HT.(IN) | WINDOW WIDTH(IN) | THICKNESS OF WINDOW GLASS(IN) |
|---|---|---|---|---|---|---|---|---|
| 10 | .0077 | 0.0000 | .0890 | 0.0000 | 2.00 | 0.00 | 0.00 | 0.00 |
| 9 | .0077 | 0.0000 | .0890 | 0.0000 | 2.00 | 0.00 | 0.00 | 0.00 |
| 8 | .0077 | 0.0000 | .0890 | 0.0000 | 2.00 | 0.00 | 0.00 | 0.00 |
| 7 | .0077 | 0.0000 | .0890 | 0.0000 | 2.00 | 0.00 | 0.00 | 0.00 |
| 6 | .0077 | 0.0000 | .0890 | 0.0000 | 2.00 | 0.00 | 0.00 | 0.00 |
| 5 | .0077 | 0.0000 | .0890 | 0.0000 | 2.00 | 0.00 | 0.00 | 0.00 |
| 4 | .0077 | 0.0000 | .0890 | 0.0000 | 2.00 | 0.00 | 0.00 | 0.00 |
| 3 | .0077 | 0.0000 | .0890 | 0.0000 | 2.00 | 0.00 | 0.00 | 0.00 |
| 2 | .0077 | 0.0000 | .0890 | 0.0000 | 2.00 | 0.00 | 0.00 | 0.00 |
| 1 | .0077 | 0.0000 | .0890 | 0.0000 | 2.00 | 0.00 | 0.00 | 0.00 |

TOTAL EARTHQUAKE DAMAGE TO BUILDING (PERCENT OF REPAIR COST/ REPLACEMENT COST)

BUILDING CLASSIFICATION: STEEL FRAME

OVERALL QUALITY RATING: AVERAGE

| TOTAL NO. STY. | AVERAGE INTENSTY. DRIFT | BUILDING PERCENT DAMAGE |
|---|---|---|
| 10 | .0068 | 1.53 |

EARTHQUAKE DAMAGE TO STRUCTURAL COMPONENTS (PERCENT OF REPAIR COST/ REPLACEMENT COST)

| | FRAME TYPE: 1 | | | | SHEARWALL TYPE: 0 | | | |
|---|---|---|---|---|---|---|---|---|
| STORY NO. | INTENSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | INTENSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR |
| 10 | .0066 | .70 | 1.44 | 9.54 | .0066 | 0.00 | 0.00 | 0.00 |
| 9 | .0072 | .78 | 1.66 | 11.45 | .0072 | 0.00 | 0.00 | 0.00 |
| 8 | .0073 | .79 | 1.67 | 11.58 | .0073 | 0.00 | 0.00 | 0.00 |
| 7 | .0071 | .77 | 1.64 | 11.23 | .0071 | 0.00 | 0.00 | 0.00 |
| 6 | .0070 | .76 | 1.60 | 10.64 | .0070 | 0.00 | 0.00 | 0.00 |
| 5 | .0066 | .71 | 1.47 | 9.71 | .0066 | 0.00 | 0.00 | 0.00 |
| 4 | .0064 | .68 | 1.41 | 9.10 | .0064 | 0.00 | 0.00 | 0.00 |
| 3 | .0060 | .63 | 1.28 | 8.01 | .0060 | 0.00 | 0.00 | 0.00 |
| 2 | .0059 | .62 | 1.25 | 7.74 | .0059 | 0.00 | 0.00 | 0.00 |
| 1 | .0080 | .89 | 1.92 | 14.04 | .0080 | 0.00 | 0.00 | 0.00 |

EARTHQUAKE DAMAGE TO NONSTRUCTURAL COMPONENTS (PERCENT OF COMPONENTS DAMAGED)

| | COMPONENTS SENSITIVE TO FLOOR MOTION | | | | GLASS | | | |
|---|---|---|---|---|---|---|---|---|
| STORY NO. | FLOOR ACCL.(G) | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR | INTENSTY. DRIFT | PERCENT DAMAGE BY QUALITY GOOD | AVERAGE | POOR |
| 10 | .27 | 2.10 | 13.98 | 93.15 | .0066 | 2.71 | 5.45 | 17.15 |
| 9 | .17 | 1.60 | 10.22 | 65.10 | .0072 | 3.89 | 7.30 | 22.67 |
| 8 | .15 | 1.32 | 8.11 | 50.00 | .0073 | 3.98 | 7.43 | 23.06 |
| 7 | .16 | 1.17 | 7.05 | 42.58 | .0071 | 3.75 | 7.08 | 22.01 |
| 6 | .17 | 1.09 | 6.53 | 39.01 | .0070 | 3.49 | 6.69 | 20.86 |
| 5 | .18 | 1.05 | 6.21 | 36.87 | .0066 | 2.81 | 5.61 | 17.64 |
| 4 | .18 | .99 | 5.83 | 34.36 | .0064 | 2.47 | 5.06 | 15.96 |
| 3 | .19 | .91 | 5.25 | 30.42 | .0060 | 1.92 | 4.13 | 13.14 |
| 2 | .18 | .77 | 4.33 | 24.43 | .0059 | 1.79 | 3.91 | 12.47 |
| 1 | .17 | .56 | 2.98 | 15.90 | .0080 | 5.84 | 10.13 | 30.99 |

FRAME TYPES:

0: NOT APPLICABLE
1: STEEL
2: REINFORCED CONCRETE
3: PRECAST CONCRETE

SHEARWALL TYPES:

0: NOT APPLICABLE
1: REINFORCED CONCRETE
2: PRECAST CONCRETE
3: BRICK MASONRY

END OF JOB.