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The John A. Blume Earthquake Engineering Center

Department of Civil Engineering
Stanford University

A STUDY OF SEISMIC RISK FOR NICARAGUA

Part II

COMMENTARY

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The report provides detailed discussions on the development of seismic hazard maps, damage prediction, insurance risk, and design methodology, and is part of a comprehensive study on seismic risk for Nicaragua. Included is the proposed seismic design procedure, design philosophy, acceptable risk, development of the dynamic amplification factor shape statistics, effective structural response spectra, types and behavior of lateral force resisting systems, reliability of design objectives, and discussions of the proposed design and method. A statistical analysis of response spectra and acceleration peaks, basics of elastic dynamic analysis, a planning matrix, special design considerations, and risk data are contained in the appendices as is a discussion of differences which affect any comparison between Nicaragua and SEOAC or UBC seismic load criteria.

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Forecasting	Seismic waves	Damage control

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DEFINITIONS AND NOTATIONS FOR SOME COMMON TERMS

A_D, A_C = are PGA values at the damage and condemnation levels respectively

DAF = Dynamic Amplification Factor

CDS = Condemnation Deformation Spectrum

DDS = Damage Deformation Spectrum

DFS = Design Force Spectrum

DMS = Design Overterning Moment Spectrum

d_T = member design force level factor for a particular type of lateral force resisting system

d_{OT} = design overturning moment factor for a particular type of system

E = Earthquake force on a member due to the DFS response

E'_C = Earthquake force on a member due to the CFS response

L = Structure Economic Life

MCS = Mean Condemnation Spectrum

MDS = Mean Damage Spectrum

MDAF = Mean Dynamic Amplification Factor

P_D, P_C = are the respective probabilities of exceeding A_D, A_C during the structure life L

PDAF = Peak Dynamic Amplification Factor

PGA = Peak Ground Acceleration value of earthquake accelerograph

R = Acceleration Reduction Factor = 0.7

RP_D, RP_C = are the respective return periods for A_D, A_C

R_u = Ultimate Strength Capacity of a member

SRSS = Square Root of the Sum of the Squared modal response to a given spectrum

V_B = Base Shear

V_S = is the coefficient of variation of the individual spectral ordinates as they are scattered about the mean shape value

β_T = Total damping for a given structural system type

$(1+k_T V_S)$ = confidence level factor where k_T depends on the particular type of lateral force resisting system in a structure

Δ = Structure Deformation

θ = Member Deformation

F = Member Load due to V_B

μ_C = measure of average ductulity demand at the condemnation level

$\bar{\mu}_C$ = local member ductility demand at the condemnation level

σ_S = standard deviation of spectral ordinates about mean shape

PREFACE

In January 1975, the first report, "A Study of Seismic Risk for Nicaragua, Part I" was published under the present study. The second and final part of this study is presented herewith in two separate volumes. Report No. 12A is "A Study of Seismic Risk for Nicaragua, Part II, Commentary". Whereas Report No. 12B is "A Study of Seismic Risk for Nicaragua, Part II, Summary".

In order to assist the reader in understanding the development of the proposed methodology, the following order of reading is suggested.

1. Report 12B, Summary Volume.
   This provides an overview of seismic hazard zoning, the design methodology and sample design problems.
2. Report 12A Commentary Volume.
   This volume provides detailed discussions on the development of seismic hazard maps (Chapter II), damage prediction and insurance risk (Chapter III) and the design methodology (Chapters IV through XIII). The summary of the design methodology development is given in Chapter IV. Each chapter begins with a description of the scope for that chapter. This should aid the reader in grasping the intent of the chapter.

The results presented in these reports represent a recommended methodology. For formulation of a building regulation based on this methodology, further study and coordination with Nicaraguan architects, engineers and planners is needed.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

CHAPTER I

INTRODUCTION

SCOPE

This document is the second and final report on the study of seismic risk for Nicaragua. In this chapter, the initial Part I report of this study is summarized and the relevancy of the material of this final Part II report is introduced. Some basic definitions of hazard and risk are given.

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I-1 Summary of the Initial Part I Report

In the report titled "A Study of Seismic Risk for Nicaragua, Part I" published as a technical report No. 11 by the John A. Blume Earthquake Engineering Center at Stanford University in January 1975, the following topics were discussed: (Reference 1)

- Geological and seismological setting of Nicaragua
- Seismic data base
- Development of probabilistic models to obtain seismic hazard information in the form of iso-acceleration maps.
- Seismic risk zoning for the country based on seismic hazard maps. Concepts of return period, acceleration zone graphs and consistent risk design.
- Probabilistic intensity forecasting and damage estimation. Insurance risk or damage potential.
- Relationship of iso-acceleration and acceleration zone graphs to seismic design provisions.

It can be seen from the above summary topics that the first report dealt primarily with the seismic hazard evaluation of the country. Very little attention was paid to the incorporation of seismic hazard maps with a design methodology. Major effort was concentrated on understanding the seismic history of the country. Based on the forecasting models developed, a future "risk" based loading information in the form of peak ground accelerations was developed. This was the first and most important step in the development of a design methodology based on an "acceptable risk" criteria.

I-2 Some Basic Concepts Concerning Hazard and Risk

In order to convey the importance of seismic hazard and risk analysis to the reader, some basic notions are presented in this section. In the earthquake engineering literature, there is in general, ambiguity regarding two words. They are: Hazard and Risk. Seismic hazard is regarded by many to be synonymous with seismic risk. Earthquake engineers and planners use these two words loosely and interchangeably in their work. There is some danger in this ambiguity since these two words within the context of earthquake engineering have different meanings.

Seismic hazard is defined as "expected occurrence of future adverse seismic event".

Seismic risk is defined as "expected consequences of a future seismic event".

Consequences may be life loss, injury, economic loss, function loss and damage. Expected hazard and expected risk have an implication of future uncertainty. Hence, it is not surprising that principles of probabilistic forecasting and decision making are essential in any seismic hazard or seismic risk analysis.

In a recent report (Reference 2) to the United States Congress by the U. S. Executive Office of the President, Office of Emergency Preparedness, the following two recommendations were made.

1) The development of seismic hazard maps is an essential first step in hazard reduction and prepardness planning.
2) The greatest potential for reducing the loss of life and property from earthquakes lies in restructuring the use of land in high risk areas and in imposing appropriate structural engineering and materials standards both upon new and existing buildings.

As can be seen from above, it is essential that a seismic hazard map be prepared for the region under study as a first step. This was accomplished in Part I of this study (Reference 1).

The Vice Ministry for Urban Planning in Managua has developed a land use map based on seismic hazards such as

1) Surface rupture above the fault
2) Earthquake induced landslides
3) Subsidence
4) Liquefaction potential

These land use maps together with ground shaking hazard maps developed in this study can be used to develop a proper building design methodology.

Such a methodology can help to assess the inherent risk of existing structures due to future probable earthquakes. It can also help to formulate a seismic load resistance requirements for new construction so that a certain level of acceptable risk is achieved.

In this report, a final seismic hazard map in the form of an Iso-Contour Map is presented in Chapter II. The seismic ground shaking hazard can also be represented for major cities in the form of acceleration zone graphs (AZG) presented in their final recommendation form in the same chapter.

After some discussions regarding the damage potential estimates and insurance risks in various parts of Nicaragua in Chapter III, the rest of the report is devoted to the development of a design methodology which,when implemented,could help in reducing the future seismic risks to an acceptable level.

A word is needed as to why the total work of ground shaking hazard map development and its use in developing structural standards be lumped under one title of "Seismic Risk Analysis". It is felt by the authors of this report that unless the development of hazard maps in properly incorporated with their use in building standards and codes, there will be discontinuity in proper communication between geologists, seismologists, planners and engineers. This is the first time that a reliability or risk based methodology covering seismology, geology, planning and engineering standards is developed. Development of hazard maps without consistently developing a seismic load resistance requirement does not constitute a total seismic risk analysis. Similarly, developing a seismic load resistance requirement without properly evaluating the seismic load level for some acceptable levels of risks also does not constitute a rational approach. In this study of the seismic risk of Nicaragua, we have attempted to do both the above tasks rationally and consistently.

CHAPTER II

SEISMIC ZONING

SCOPE

A single Iso-Contour Map representing future probable seismic loadings is developed in this chapter. Also, modified acceleration zone graphs for major cities of Nicaragua are presented.

II-1 Iso-Contour Map

In "A Study of Seismic Risk for Nicaragua, Part I" report, iso-acceleration maps for various return periods were presented. In particular the following iso-acceleration maps for a given exposure time, "risk" and return period were made available. See Table 2-1.

Table 2-1

*Chart #	Exposure Time Years	"Risk"	Return Period Years	Risk/Yr.
8	50	10%	475	.2%
9	50	20%	225	.4%
10	50	50%	72	1.4%
11	20	10%	190	.5%
12	20	20%	90	1.1%
13	20	50%	29	3.4%

* Chart numbers referred to are those in Reference 1.

(To understand the relationship between the return period, probability of exceedance or "risk" and exposure time, refer to Reference 1, Chapter V and repeated here as Table 2-2 and Figure 2-1).

One could obtain the peak ground acceleration for a given site for a given return period by using an appropriate chart mentioned in Table 2-1. If the site of interest is in one of the following eleven cities, the acceleration zone graphs presented in Reference 1 could be used. The eleven cities considered are:

1. Managua
2. Masaya
3. Leon
4. Granada
5. Rivas
6. Chinendega
7. Juigalpa
8. Estelli
9. San Carlos
10. Matajalpa
11. Bluefields

It is not practical to develop a separate iso-acceleration map for each of the return periods of interest. In that case, many such maps would be needed to satisfy the needs of different design situations. Also, it is not possible to include so many maps for any future seismic code formulation. It is much more practical to have one seismic hazard map which includes information on peak ground acceleration as a function of acceptable risk exposure time or return period. From this single map it is possible to develop iso-acceleration maps for different return periods and "risks".

Table 2-2

Return Period as a Function of Economic Life and
Probability of Non-exceedence

Economic Life Years / Probability of not exceeding %	10	20	30	40	50	100
90	95	190	285	390	475	950
80	45	90	135	180	225	449
70	29	57	84	113	140	281
60	20	40	59	79	98	196
50	15	29	44	58	72	145
40	11	22	33	44	55	110
30	9	17	25	34	42	84
20	7	13	19	25	31	63
10	5	9	14	18	22	44
5	4	7	11	14	18	34
1	3	5	7	9	11	22
0.5	2	4	6	8	10	19

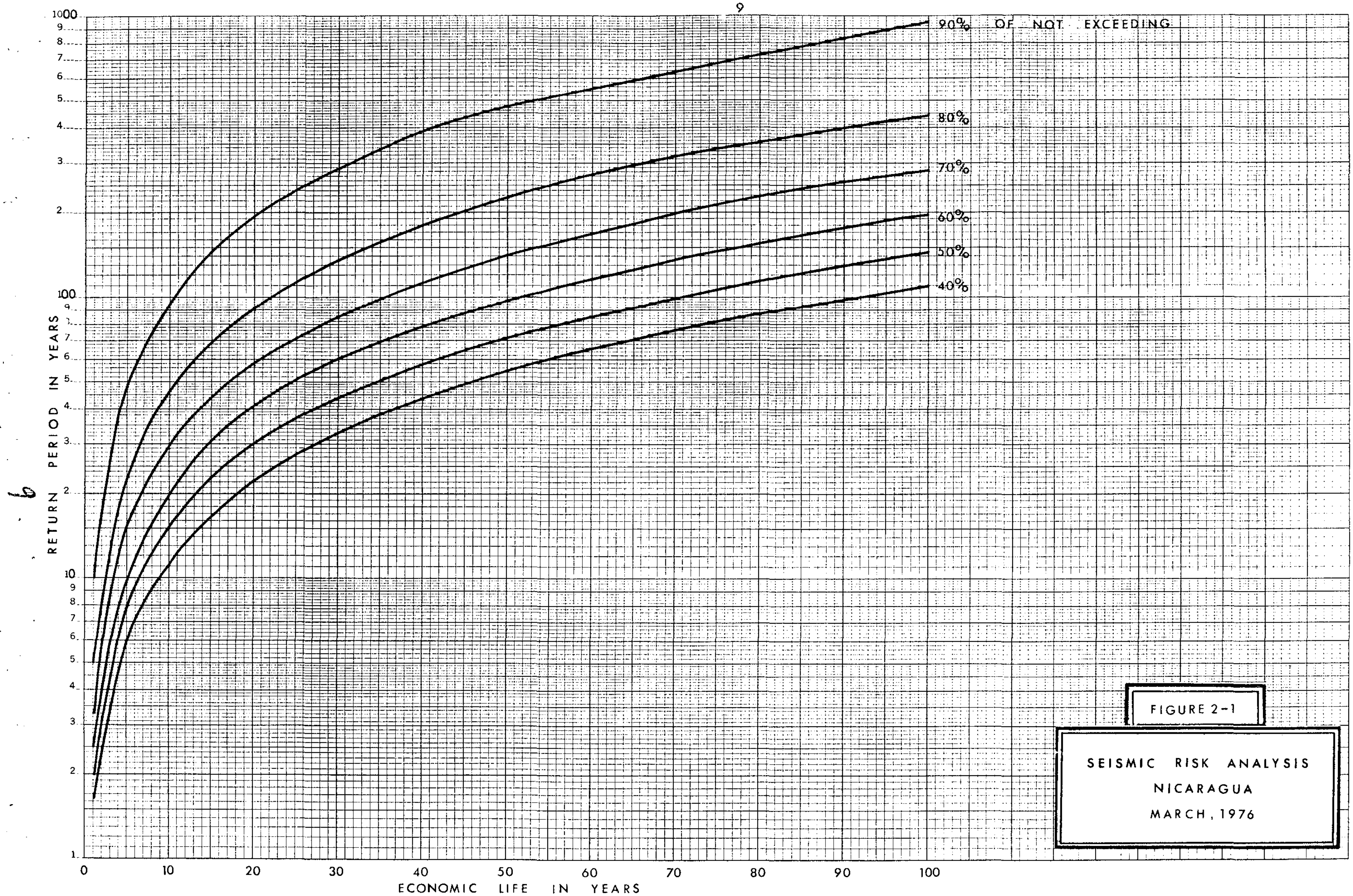


FIGURE 2-1

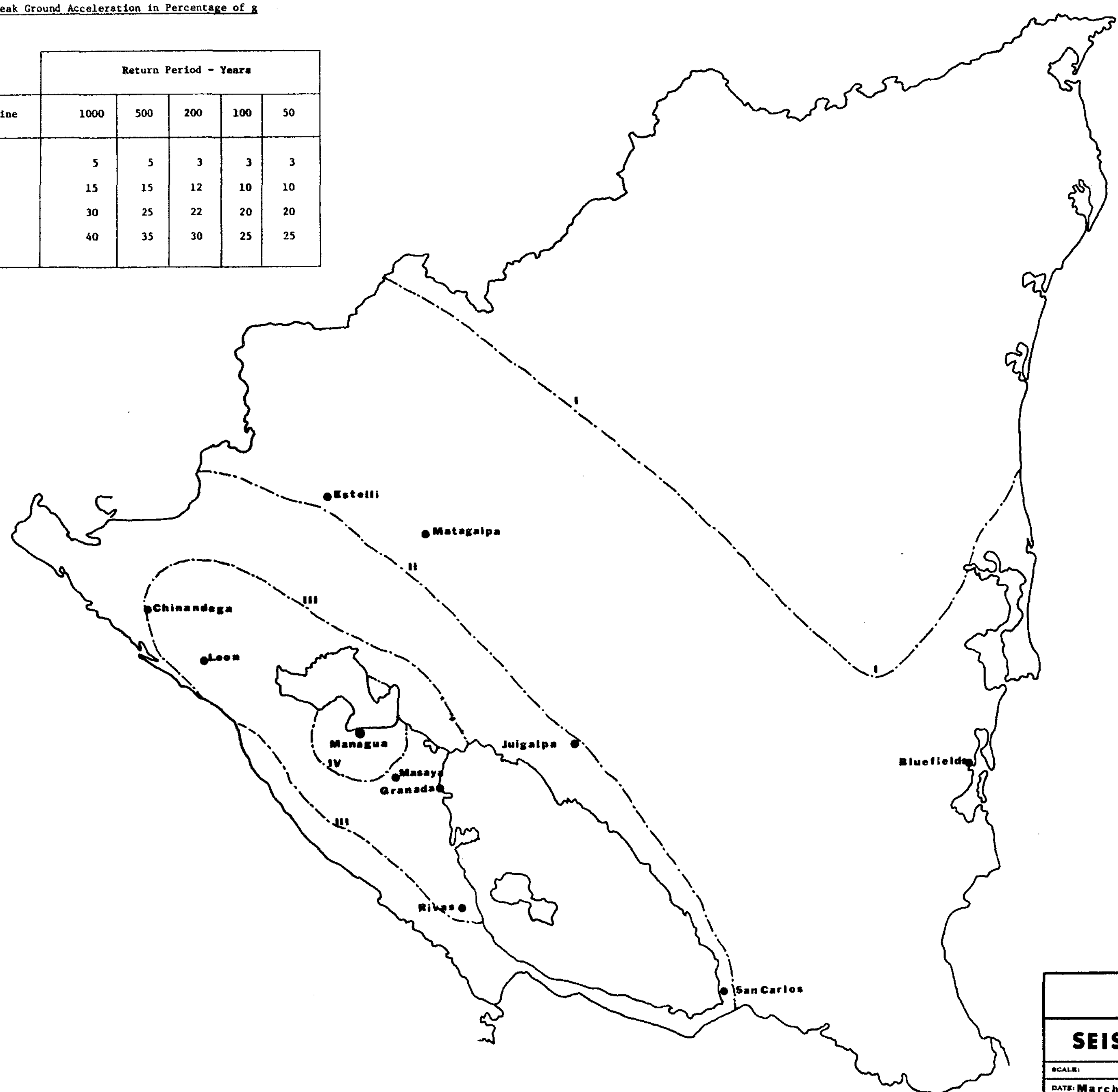
SEISMIC RISK ANALYSIS
NICARAGUA
MARCH, 1976

Figure 2-2 represents such a seismic hazard map for Nicaragua. Each numbered line on the map is a contour. The numerical value of peak ground acceleration, corresponding to each contour is given in Table 2-3. Thus, the contour map (Figure 2-2) together with Table 2-3 gives the information on peak ground acceleration at any location as a function of the return period. In Chapter V, suggested return periods for various use classes of structures will be presented. As an example, the peak ground acceleration corresponding to contour line III for a 500 year return period is 25% of g where g is the acceleration due to gravity. Similar statements can be made for other contour lines and return periods.

It should be stressed that these contour lines do not represent boundaries of seismic zones. They represent a numerical value of the peak ground acceleration for a specific return period. In that respect, the contour lines are similar to elevation contour lines. To obtain the value of peak ground acceleration between any two contour lines corresponding to (say) 500 year return period, a linear interpolation between these two lines must be made. As an example, consider a site east of Matagalpa which is equidistant from contour lines I and II. It is desired to determine the peak ground acceleration for this example site corresponding to a return period of 500 yrs. From Table 2-3, it can be seen that the PGA corresponding to contour II and 500 yr. return period is 15 percent of g. Also, the PGA for contour I and 500 yr. return period is 5 percent of g. Hence, the PGA at the example site for a 500 yr. return period should be approximately 10 percent of g.

Peak Ground Acceleration in Percentage of g

Contour Line	Return Period - Years 1000	500	200	100	50
I	5	5	3	3	3
II	15	15	12	10	10
III	30	25	22	20	20
IV	40	35	30	25	25



NICARAGUA

SEISMIC HAZARD MAP

SCALE:

DATE: March, 1976

FIGURE 2-2

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Table 2-3

Peak Ground Acceleration in Percentage of g

	Return Period - Years				
Contour Line	1000	500	200	100	50
I	5	5	3	3	3
II	15	15	12	10	10
III	30	25	22	20	20
IV	40	35	30	25	25

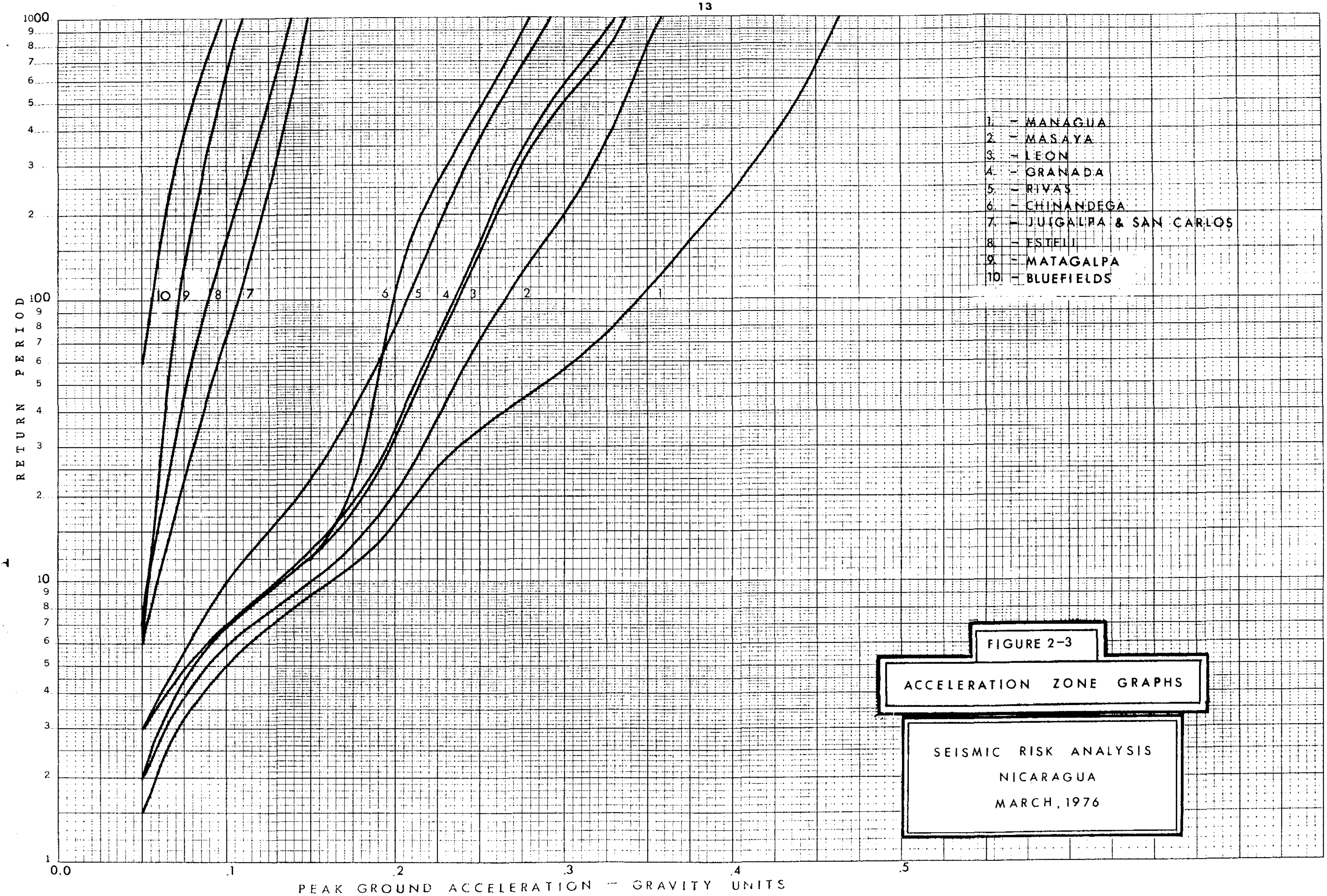

FIGURE 2-3

ACCELERATION ZONE GRAPHS

SEISMIC RISK ANALYSIS

NICARAGUA

MARCH, 1976

The region east and northeast of contour line 1 can be considered as the low plateau. Thus the minimum peak ground acceleration that one can consider corresponds to the values for contour I. The highest PGA values are for the region around Managua.

It should be stressed again that for determining PGA values for any of the eleven cities shown on the map, the acceleration zone graphs, given in Figure 5-1 of Reference 1 and modified in its final form as Figure 2-3 of this chapter, should be used.

As a further classification on the use of Iso-Contour Map of Figure 2-2 and the acceleration zone graphs of Figure 2-3, consider a site within contour line IV. Assume that this site is equidistant from Managua and the boundary of contour IV. What should be the PGA for this example site for a return period of 100 years?

From the acceleration zone graph of Managua; Figure 2-3, the PGA corresponding to a 100 year return period is 0.35g. From Table 2-3, the PGA corresponding to contour IV and a 100 year return period is 0.25g. Thus, using linear interpolation, the peak ground acceleration for the example site is 0.30g corresponding to 100 year return period. Thus, by combined use of Table 2-3 and Figures 2-2 and 2-3, one could obtain the information about the future peak ground acceleration for a specific return period. All the values presented in this seismic hazard map or the acceleration zone graphs assume "firm" site conditions. For soft site conditions, modifications to these values (discussed in later chapters) or specific site study may be needed.

II-2 Concluding Remarks on Seismic Hazard Maps

Unlike the older seismic zone maps (such as the 1973 Uniform Building Code "risk" map) the recommended hazard map takes into account the frequency of seismic events, the level of "risk" one is willing to take in selecting a specific peak ground acceleration value and the future time horizons for which one wishes to consider the economic or structural life of the facility being designed.

Various questions come up regarding the reliability and long range stability of such hazard maps. Some of the questions are:

1. How reliable are the maps that are developed based on only historical data?
2. How stable are such maps? In other words, will these hazard maps change dramatically with each new future seismic event?
3. Is the formulation such that any new information available in the future can be incorporated to update the hazard maps?
4. What is the effect of local site conditions on the values obtained from these maps?

These and many such questions were discussed in Reference 1 of this study. However, in summary, the following responses can be given to the four questions posed above.

With respect to the reliability of results based on historical data, it is felt that for engineering and planning purposes and for seismic code formulation, the results presented are sufficiently reliable. The usual economic life of any engineered facility is usually less than 100 yrs. to 200 yrs. In terms of geological time spans, this is a short period. Hence, we can assume that the geological processes during this short period are at a steady state. Hence, any information available from historical data can be extrapolated into similar time spans in the future. This discussion does not mean to imply that there are no errors introduced. This possibility always exists. However, to wait for a complete geological information before developing a "seismic load" criteria for a country is unrealistic and impractical.

Concerning the stability of the hazard map, it is felt that the results presented here are quite stable. It was shown in reference 1 that 13 seismic sources were considered to develop the suggested hazard map. As long as the future seismic events can be assigned to any one of these sources, the shape of the maps as well as the level of PGA's suggested should not change substantially. The only time the maps should be updated and changed is when a major seismic event occurs in a region where no previously known seismic source or sources existed. In that case, the formulation and the computer programs are such that the suggested maps can be readily updated with the new information incorporated. Thus, in reply to the third question, such maps could be updated very easily. <u>As a general recommendation, it is felt that such maps should be updated every five to seven years</u>. (See Reference 3)

Effect of local site conditions (micro-characteristics) is usually felt in the amplitude of vibrations and in the frequency content of the vibration. The hazard map developed here is based on "average" soil condition. Thus, no site specific information is included in their development. However, in Chapter VI the effect of soft soil is introduced by changing the shape of the response spectrum to include higher period components. However, it should be pointed out that for important facilities such as dams, power plants, hospitals, etc., a site specific study should be conducted. Such information can then be used to modify the values suggested by the hazard map of this chapter and the spectrum shape of Chapter VI.

In conclusion, it can be said that the seismic ground shaking hazard information developed in this study represents "a state-of-the-art" engineering solution. It is not the total information but it is one of the best that can be developed with the available knowledge and resources.

CHAPTER III

DAMAGE PREDICTION

SCOPE

In this chapter three methods of damage prediction are introduced. It is shown that a damage potential for a certain class and type of structure is proportional to the level of seismic hazard. With this argument in mind, some observations regarding the "insurance risk" are made.

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III-1 Damage Prediction Methods

Various methods, of predicting damage due to a given level of seismic event, are available in the literature. Knowledge of the future damage and loss due to a postulated seismic event can be a vital input for disaster mitigation, earthquake insurance and in developing a rational seismic resistive design formulation. In a recent report (see Reference 4) three state-of-the-art methods of predicting damage were studied. These three methods are:

1. The Spectral Matrix Method (SMM). In this procedure, a probabilistic formulation for demand (seismic load) and capacity (resistance) using theoretical models and based on empirical observations is developed to produce damage estimates.

2. The Seismic Element Method. In this method, the demand is median spectral acceleration (for 5% damping) based on statistically developed spectral shapes. No statistical variation in demand is considered.

3. The Decision Analysis Method. This method is based on damage data obtained from past earthquakes and statistically fitted to empirical equations. In the damage potential studies presented in Part I of this study, this particular method was employed.

The following summary comparison between the three methods is taken from Reference 4.

Predicting building damage due to an earthquake can be conceptualized in several ways. One approach is to treat total damage as the sum of damage to individual structures. This approach lends itself to a formal probabilistic and statistical development, which is particularly useful considering the highly variable nature of damage. In such an approach, damage information and relationships must first be obtained for individual structures. Once these relationships are determined satisfactorily, total damage estimation is simply a matter of statistical combination. An analogous use of individual element information combined to obtain overall information is the finite element method of stress analysis. Although only monetary damage is being mentioned here, loss of life predictions and the estimation of the social and economic impact of earthquake damage are of equal, if not greater, importance in seismic planning and risk mitigation. These prediction techniques, however, require the use of parameters that are not as easily quantified as monetary damage. Although there have been efforts to account for damage in more than monetary terms, prediction techniques in these areas remain in an embryonic stage.

In general, damage to a structure is a function of demand, capacity, and the value of the building. Damage due to ground motion occurs when the response of a building exceeds the ability of the structural and architectural components to withstand such motion, i.e., when demand exceeds capacity. The demand at a site should include ground motion amplitude, duration, and frequency content effects. Capacity should be a function of such factors as construction type, structural and dynamic properties, age, condition, and size. The result will be a relationship between damage for a building and the demand imposed on it and will reflect associated capacity and value levels. This relationship will be the focal point for comparison in this chapter.

Format for Damage Estimates

In arriving at damage estimates, several levels of sophistication may be adopted. In order of increasing complexity, these are:

1. A central value measure. This is usually manifested as mean total damage either in monetary terms or as a percentage. It represents the basic relationship between damage and demand, in which demand may be expressed in various ways, such as spectral acceleration, spectral velocity, or an intensity scale. When damage is viewed as a percentage, it is usually done with respect to replacement value. The total damage is simply the sum of individual damage.

2. A measure of scatter about the central value. Loosely stated, scatter will yield an estimate of the bounds on damage. Typically, this implies an estimate of variance, or, equivalently, standard deviation or coefficient of variation. When total damage is considered as the sum of individual damage, scatter must also include consideration of damage correlation between buildings.

3. Probabilities of damage. Ultimately, probability statements for total damage can be made by postulating probability damage distributions for individual buildings. Additionally, these statements may include consideration of time, thus yielding statements on expected damage for a given period.

Only comparisons of central values and scatter are considered here. Although comments are made concerning probability statements, detailed investigation is left for later studies.

Central Value Measure

SMM. The SMM makes use of probabilistic formulations for individual building demand and capacity in order to make damage predictions. Demand is a spectral response value assumed to be lognormally distributed. Capacity is defined as the demand level at which first yield occurs. Inelastic strength is also considered by assuming that the total energy dissipated is nearly the same as the energy stored by a perfectly elastic model. The capacity probability distribution is assumed to be a Weibull distribution.

The damage parameters in the SMM are a damage factor, defined as the ratio of repair cost to replacement value, and a damage state, defining the building as either undamaged or damaged. These parameters are defined for individual buildings and then statistically combined for classes of buildings. The damage factor is represented by a mixed probability function, while the damage state is a mass probability function. Both are functions of a normalized ratio of demand over capacity.

Seismic Element Method. In the seismic element method, demand is a spectral acceleration based on median response spectra. Statistically developed spectral shapes at 5% damping are used with estimated peak ground accelerations. No probabilistic variation in demand is considered directly.

The resulting damage estimates are in the form of damage factors defined as the ratio of repair cost to replacement value. Three types of damage factors are defined: an elastic value based on first damage with no reduction in structural strength, an inelastic value based on a change in building period, and a weighted combination of the two used to assign degrees of damage. These factors are directly obtained from demand values. Although dynamic structural behavior is not directly examined, structural capacity is implicitly considered through a multiplicative factor that takes into account both theory and empirical observation. Damage factors are determined for individual buildings and then combined to produce maps that define zones of varying damage levels.

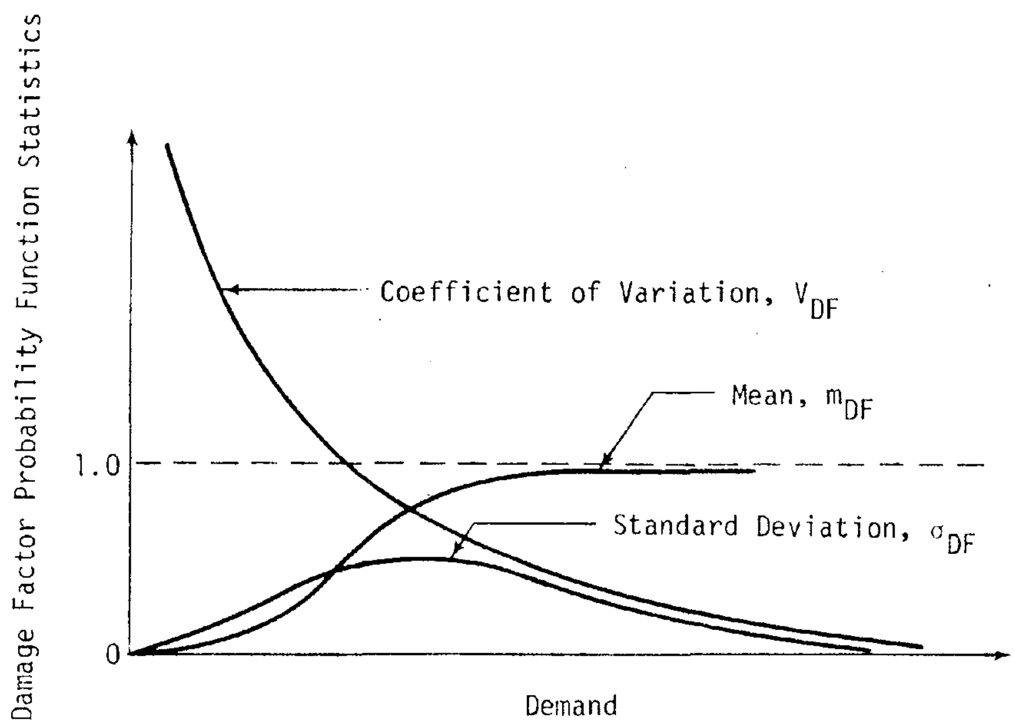
Decision Analysis Method. Unlike the other two damage prediction methods, the decision analysis procedure does not consider damage to an individual building. Instead, it is based on a statistical examination of damage data recorded from the 1971 San Fernando, 1933 Compton, and 1952 Kern County earthquakes. The damage statistics are assumed to be lognormally distributed about a median value. A conditional linear predictor is fitted to the data points, and a relationship between percentage loss and demand is developed. From this, a conditional median loss or, by extension, any other loss condition is defined.

The decision analysis method defines demand in terms of Modified Mercalli Intensity (MMI). The use of intensity as the demand parameter introduces considerable variation in the determination and interpretation of the damage versus demand relationship. However, MMI remains an important parameter because of its historical use in relating damage data to ground motion.

Scatter

SMM. In the SMM, variance of loss is considered both at the individual building level and at the total damage level. For individual buildings, the variance of the damage factor is conceptualized as near zero at low demands, increasing for moderate demands, and then decreasing to zero for high demands (Figure 3-1). The variance in turn depends on the previously developed distribution functions of demand and capacity.

The variance of loss for individual buildings is statistically combined to produce variance estimates for total loss. In addition, the variance of total damage takes into account correlation of damage between buildings. In general, damage may be correlated between demand and building capacity. As a first approximation, damage in the SMM is assumed to be correlated only through uncertainty in demand.



EXAMPLE CONDITIONAL DAMAGE FACTOR PROBABILITY
DISTRIBUTION FUNCTION STATISTICS

FIGURE 3-1

Seismic Element Method. Because the seismic element method is not based on a statistical formulation, a formal consideration of scatter is inappropriate. However, implied uncertainty in damage estimates is accounted for by use of broad classifications of damage and the use of qualitative damage levels in the final damage estimation rather than numerical values of loss.

Decision Analysis Method. As an approximation, linear variation in the conditional median loss relationship in the log domain is assumed for the decision analysis method. In the range of interest, this approximation is considered to be reasonable. Moreover, such an assumption facilitates the use of linear statistical models.

Probabilities

SMM. For the SMM, probability distributions for total damage can be constructed from assumed individual structure probability distributions. Using Chebyshev's inequality, weak probability statements can be made on the probability that damage is less than a specified damage level.

Seismic Element Method. The use of median damage factors represents a 50% probability statement on damage estimates. However, in its present form, the seismic element method is not intended for making additional probability statements.

Decision Analysis Method. Because the damage parameter versus demand curve in the decision analysis method is a median curve, it defines the 50% probability level. Using normal distribution tables, other loss conditions can be determined.

Additionally, the decision analysis method explicitly details a procedure for incorporating damage parameter versus demand predictions into long-range policy analysis. Assuming that a mass probability distribution of demand for a desired return period has been determined, the expected value of damage can be calculated using median values from the damage versus demand relationship. This value can then be adjusted to any loss probability by use of normal distribution tables.

As mentioned earlier, the decision analysis method was used to estimate damage potentials for different types of buildings in this study. Before looking into these results, the following observations and conclusions can be made regarding the three methods of damage prediction.

The most obvious source of variability in the damage estimate comparisons is the use of intensity as a common demand parameter. As a demand parameter, intensity is less than ideal because of the subjectivity inherent in its use. However, in the absence of more specific information, intensity may be the most easily derived basis for damage. As knowledge about the relationship between damage and ground motion increases, either through experimental data or theoretical developments, it would be desirable to have a more quantifiable value as the demand parameter. The seismic element method and the SMM are examples of the developing use of spectral acceleration as a demand parameter. Future damage prediction methods should seek to incorporate such developments into the prediction technique.

The seismic element method provides, within ranges, good results compared to the other methods. However, because the seismic element method is developed specifically for the San Francisco area, extension of this method to regions with significantly different types of construction or seismic history must be done cautiously. In such a case, the various damage relationships may have to be considerably modified to reflect the characteristics of the particular area.

Similarly, the decision analysis method is based on the damage history of a particular area. Inappropriate or inadequate data can produce misleading damage estimates. The different light industrial construction in San Fernando gave low damage estimates for high intensities, and the lack of data on 3- and 4-story buildings prohibited any comparison for this class of buildings. Increased and more intensive study of damage data - for example, expanding the number of classes studied - will create a more comprehensive data base and help to alleviate this problem.

An additional problem in the decision analysis method is the use of constant variance. As demonstrated earlier, this assumption does not appear to be appropriate, particularly for a MMI greater than VIII or less than VI. The errors caused by this assumption become greatest at the extremes; for high or low intensities, the decision analysis method must be used with discretion. However, in some cases, the inaccuracy caused by the assumption of constant variance may be acceptable in order to facilitate quick computation. Future studies may include an analysis of the magnitude of this error.

Between the three methods, the SMM offers the greatest flexibility in predicting expected damage behavior. By altering different parameters, many of the variabilities in structural assemblage and material properties can be taken into account. As more damage information becomes available, it is expected that this capability will enable the SMM to help identify better building design practices.

Despite the limitations of the various damage prediction methods, there is an underlying damage phenomenon that all three methods attempt to model. As information on damage becomes scarce at high demand, the variation in the damage estimates become greater. Future studies should consider procedures to incorporate new data into present methods, as well as including sensitivity analysis and quantifying demand. It can be said that no one approach, theoretical empirical, or intuitive, can wholly describe damage behavior with certainty. Instead, each can be used to complement the other.

III-2 Insurance "Risk" for Nicaraguan Cities

As can be inferred from the previous section, no single state-of-the-art prediction technique can really help in estimating precisely the damage potential for a given class of structures due to a given seismic hazard. However, it can be said that the seismic risk for monetary loss is a function of

1. Seismic hazard
2. Type of construction
3. Type of occupancy (Use Class)

For insurance risk evaluation, the overall loss potential is important. Thus, a fourth variable, the number of structures of a given type and use subjected to a given seismic hazard, is also important. If the type of construction, the type of occupancy and the population of the region are constant for two seismically active regions, then the insurance risk should be a function of seismic hazard only. Table 3-1 gives seismic hazard information for the eleven cities in Nicaragua for various return periods. It can be seen from this table that the ground shaking seismic hazard changes substantially between Managua (highest level) and Bluefields (lowest level). Thus, if the populations of these two regions were similar, then for the same type of construction and use, the total expected insurance claim should be approximately in the ratio of this seismic hazard. (Here, the effect of local site conditions are not taken into consideration. They will play an important role.) However, the population and hence the number of claims for a given use class makes the insurance risk problem much more complex. Table 3-2 taken from Reference 1 gives some expected median losses for the eleven cities mentioned in Table 3-1. It can be seen from this table that the maximum expected damage for wooden one and two story structures, built with the technology of pre 1940, would be about $2.00 per thousand dollar value per year. This would be true only when the damage is averaged over 20 years. If the expected damage is averaged over 50 years, the corresponding amount would be about $1.10. For comparison, in the San Francisco Bay region, the earthquake insurance rates for comparable construction vary between $1.50 to $3.50 per thousand dollar value per year with 5% of the value of the property deductible. Thus, for a $100,000 home the earthquake insurance premium would be anywhere from $150 to $350 per year with $5000.00 earthquake damage deductible.

Table 3-1

Seismic Hazards in Major Cities
(Ground Shaking Hazard)

Peak ground acceleration in g units

	Return Period			
Cities	1000	500	100	50
Managua	.465	.44	.345	.285
Masaya	.36	.34	.265	.235
Leon	.34	.30	.24	.22
Granada	.33	.29	.235	.215
Rivas	.29	.265	.205	.185
Chinendega	.28	.25	.20	.19
Juigalpa	.15	.14	.11	.09
Estelli	.14	.125	.09	.075
San Carlos	.15	.14	.11	.09
Matajalpa	.11	.095	.09	.065
Bluefields	.095	.08	.055	.05

Table 3-2

Expected Median Losses

per $1000.00 value

	20 Yrs.			50 Yrs.		
	All Dwell.	P-1940 Dwell.	L. I. Struct.	All Dwell.	P-1940 Dwell.	L. I. Struct.
Managua	26.2	39.7	128.4	33.9	51.6	164.4
Leon	20.0	30.0	100.0	26.2	39.7	128.6
Granada	20.0	30.3	100.0	26.6	40.3	130.3
Masaya	24.2	36.7	119.4	29.6	44.9	144.4
Chinandega	16.5	24.8	83.4	19.9	30.0	99.3
Matagalpa	6.0	9.0	33.2	6.6	10.0	36.10
Esteli	6.6	9.9	35.7	8.1	12.1	43.1
San Carlos	7.6	11.5	40.9	9.8	14.8	51.7
Rivas	16.1	24.2	81.4	21.0	31.6	104.2
Juigalpa	7.6	11.5	40.9	9.8	14.8	51.7
Bluefields	2.9	4.8	17.5	4.5	7.1	25.8

Looking at the figures of Table 3-2, the insurance risk in Managua seems to be of similar order of magnitude as the risk in the San Francisco Bay region. If a one or two story residence is constructed in Managua with modern materials such as steel, concrete, masonary or lumber and reasonably "engineered", then the "insurance risk" and hence the insurance rates should be in the range of \$3.00 to \$5.00 per thousand dollar value with some deductible clause (as an example, 5% of the value of the property as deductible). These values are suggested here from simply looking at the relative hazard and the expected damage. They do not take into account many important parameters such as total claims, distribution of risk in space, population distributions, etc. It should be pointed out that the purpose of giving numerical example here is to present an idea about the order of magnitude of the damage potential. No conclusions regarding insurance rates or damage estimates should be made without further studies. However, if the construction materials and the building practices are similar to those used in California, then for light industrial buildings or residential houses, the numerical values of expected median damages (Table 3-2) do represent realistic estimates. For a region like Managua, expected median damage averaging over a twenty year period is quite realistic.

The authors of this report strongly feel that the insurance rates, in various cities mentioned in Table 3-2, should reflect the level of seismic hazard. It can be said that the rates should be the highest in Managua and lowest in Bluefields. If the concept of space averaging is used, then for insurance purposes, the main cities of the country could be divided into two categories. Category I could be for lower seismic insurance risk and Category II could be for higher seismic insurance risk. The distribution of various cities in one of these two categories could be as follows:

Category I		Category II	
1.	Matagalpa	1.	Managua
2.	Esteli	2.	Leon
3.	San Carlos	3.	Granada
4.	Juigalpa	4.	Masaya
5.	Bluefields	5.	Chinandega
		6.	Rivas

For each of the categories, separate insurance rates could be fixed, based on type of construction, design, use and local site conditions.

In conclusion, it can be said that the problem of insurance risk is very closely tied in with the problem of damage estimation. Since the state-of-the-art in damage estimation is not very precise, the problem of insurance risk and hence insurance rates will remain imprecise.

CHAPTER IV

INTRODUCTION TO THE PROPOSED SEISMIC DESIGN PROCEDURE

SCOPE

In this chapter the general overview of the seismic design methodology developed through this research is presented. A short description of all major parameters and steps is given to provide the reader with a quick comprehension. This chapter can be viewed as a summary of work that follows in detail in succeeding chapters.

....

In order to design economical buildings which will perform adequately during strong earthquake ground motions, it is necessary for structural engineers to have a practical understanding of:

(1) The probability of occurrence of important levels of earthquakes.

(2) The acceptable risk associated with these events for different use classes of structures.

(3) The representation of earthquakes in terms of response spectra at the structure site.

(4) The dynamic response of structures to the important levels of earthquakes.

(5) The earthquake demands on the strength, stiffness, ductility, and energy dissipation capacity of various structural systems.

(6) The design of the structural elements and lateral force resisting system such that the important levels of earthquakes may be resisted with acceptable reliabilities of performance.

In the chapters which follow, a seismic design procedure is formulated which hopefully will provide the engineer with this needed understanding. In order to assist the reader in the organization of the presented material, the following general description of the design method is given.

IV-1 Design Objectives

For a given life time of a structure, an adequate design should provide acceptable reliabilities against:

(1) excessive damage due to a moderate or damage threshold earthquake.

(2) condemnation due to a major or condemnation threshold earthquake.

(3) collapse due to a catastophic earthquake.

The value of the acceptable reliabilities of protection against each level of earthquake depends on the use class or importance of the structure. The concept of cost of protection versus seismic risk should be considered in this evaluation.

Moderate, major, and catastrophic earthquakes are described in terms of the seismicity at the structure site. This seismicity is expressed in terms of probabilities of peak ground accelerations for a given time period, and also in terms of the corresponding response spectrum values.

Damage control and condemnation protection are accomplished through strength requirements and deformation limitations of the structure response to moderate and major earthquake response spectra. This requires a classification of structural systems according to their respective deformation capacity at the damage threshold and ductility at the condemnation threshold.

Collapse protection against the catastrophic event is maintained by specific restrictions on the types of allowable lateral force resisting systems. These systems all must have the characteristics of maintaining vertical load carrying capability under severe lateral deformations.

IV-2 Methodology

To achieve the above design objectives, the following methodology is formulated:

(1) Forecasting of future seismic events. Develop occurrence rate of peak ground acceleration at site and site response spectra.

(2) Select peak ground acceleration and response spectra shapes for moderate (damage threshold) and major (condemnation threshold) earthquakes according to local site conditions, structure use class and acceptable risk level.

(3) Develop structure design spectra for different types of structural systems according to deformation characteristics and reliability of the system.

(4) Develop procedures for computing the response of structures to the above design spectra (modal superposition or base shear method).

(5) Develop criteria for the design of structural systems and elements (strength, ductility, drift, P-Delta effect).

All elements of the methodology and a detailed design procedure are discussed in detail in the next chapters, in the Appendix and in quoted References. Presented below are brief summaries of the most important elements of the procedure.

IV-3 Site Response Spectra

For a given region with known (overall) geological characteristics, a sample set of past major earthquake accelerographs and their corresponding response spectra can be assembled. This data set may be from the region for which seismic design criteria are to be developed or from geologically similar regions. Each response spectrum is then scaled so as to have a unit value of peak ground acceleration (PGA), and is hence termed as a dynamic amplification factor (DAF). The resulting sample set of DAF's is then averaged to form the mean DAF (MDAF) which provides the representative spectral shape for the given region (See Appendix A). This shape may be adjusted for known hard or soft soil column effects at the site. Given any forecasted PGA value for a future earthquake, the acceleration response spectrum may be obtained by multiplying the MDAF by the PGA value.

The spectrum as obtained from the basic data of instrument time history readings is then converted to an "effective" structure response spectrum by means of a reduction factor R which is discussed in detail in Chapter VII.

IV-4 Peak Ground Acceleration

The PGA values at specific sites in Nicaragua which have a probability P of being exceeded during a given economic life time of a structure are presented in Acceleration Zone Graphs or the Iso-Contour Map discussed in detail in Part I of this series of reports and in Chapter II of this report. The PGA values for the damage threshold and condemnation threshold earthquakes are termed A_D and A_C, respectively.

A seismic event, X, having a probability of exceedance, P_X, is adequately described for design purposes by the PGA value from the Acceleration Zone Graph, A_X, and the regional spectral shape, MDAF.

IV-5 Structure Use Class and Risk Levels

Planners are able to categorize the various structure uses into classes depending on their importance and need before, during and after a strong earthquake. Since it is neither practical nor economically feasible to provide a damage resistant structure for all conceivable levels of earthquake ground motions, each use class will have to admit its own particular probability or risk of repairable damage, P_D, and corresponding risk of total condemnation P_C, during the economic life. These risks should of course be very low for essential facilities such as hospitals and may be relatively high for a purely functional structure such as a warehouse. The risk of total collapse can be virtually eliminated by code restrictions on the type and quality of the lateral force resisting system in a building.

The importance of the assigned acceptable risk values of P_D and P_C for each structure use class is that they, along with the site location, determine the corresponding values of A_D and A_C from the Acceleration Zone Graphs or the Iso-Contour Map.

The design objectives are then to assure a reliable level of damage control for earthquake levels up to a PGA of A_D, and condemnation prevention against the effects of an earthquake with a PGA of A_C. The A_D and A_C values are used to scale the mean response spectrum shape (MDAF) for design purposes.

IV-6 Types of Structural Systems

The lateral force resisting system may consist of rigid frames, bracing, or shear walls - either in combination or in pure frame or wall systems. Any permissible system must have the quality of collapse prevention - the vertical load carrying system must remain intact under catastrophic ground motions which are reasonably beyond the acceptable condemnation level.

Each structural system has its own characteristics of response to the damage and condemnation threshold earthquake loadings. The measures used to evaluate these thresholds are: extent of repairable damage, ductility and energy dissipation characteristics, redundancy of the system, quality control and construction supervision, and reliability of performance in past earthquakes. Also, each particular system has its own value of total damping as it relates to the site response spectrum.

IV-7 Structure Design Spectra

Given the structure site and use class, the risks P_D and P_C are known and the values A_D and A_C are found. Having selected the structural system type with its damping value, its reputation or reliability measure, and its ability to deform beyond its strength design level to a damage state and then further to a condemnation state, three design spectra are formed:

(1) Design Force Spectrum (DFS) - this is an appropriately modified form of the spectrum for the acceptable damage threshold earthquake with PGA level A_D. The force response from this spectrum is used as the seismic design loading for the ultimate strength design of the structural members.

(2) Damage Deformation Spectrum (DDS) - this provides the structure deformation demand of the earthquake with PGA level A_D, i.e., for the damage threshold event. The resulting deformations are used for computation of P-Delta effects, and for non-structural damage analyses (drift limitations).

(3) Condemnation Deformation Spectrum (CDS) - this is the spectrum of the acceptable condemnation threshold earthquake with PGA level A_C. The resulting structure deformation response is used to estimate local member ductility demands and hence provides an approximate test whether or not these demands are within allowable limits. P-Delta effects and structural stability may be analyzed with these deformations.

Clearly, the most important of these three is the Design Force Spectrum (DFS) since its resulting design load levels must create a complete structural system such that the structural deformation response of the earthquake with PGA level A_D and risk P_D will remain reliably below the structure damage threshold. Also, in a structure designed for the DFS forces, the deformations of the earthquake with PGA level A_C and risk P_C will remain in most practical cases reliably below the structure condemnation threshold. This spectrum also must meet the practical restrictions of economically feasible design, and as such must not differ radically from the seismic load recommendations of modern codes. For overturning moment, a special spectrum termed Design Overturning Moment Spectrum (DMS) is developed for systems with ductile shielding of the vertical load carrying members.

IV-8 Computation of Response

The basic method chosen for the computation of the structural response is the modal superposition method. The principle of superposition makes it necessary to select a linear elastic model of the structure. This also facilitates the computational effort in design offices since computer programs for linear elastic response of two and three dimensional structural configurations are readily available.

Natural frequencies and mode shapes can be computed based on the mass distribution and deformation characteristics of the lateral force resisting system, but also should include the effects of stiff elements that are not part of the lateral force resisting system. Then, for a given spectrum (any one of the three design spectra) the structure response (force or deformation) is computed as the square root of the sum of the squares of the individual modal responses to the given spectrum (SRSS response).

For the case where the computed deformations are beyond the linear elastic range of the structure, it is assumed that the deformation response in the actual non-elastic structure is given by the SRSS deformation response of the linear elastic model. It is recognized that this linear procedure can result in a certain amount of approximation error, however, this will be compensated for by an appropriate spectral confidence level and a requirement for special analysis for irregular structures.

For structures which meet certain requirements for regularity and symmetry, a simplified method will be formulated. Empirical relations for structure periods, a base shear coefficient, and lateral force distribution will be given to provide a safe upper bound of design in lieu of the more lengthy modal analysis and response spectrum method. This is a most essential step in order to assure widespread application; however, even this simplified method will contain a descriptive commentary so that the designer is aware of the essential elements: earthquake levels and their associated risks; dynamic response of structures to these earthquakes; and design provisions for adequate behavior at the damage and condemnation thresholds.

IV-9 Design Criteria

The seismic loads resulting from the Design Force Spectrum (DFS) response, together with ambient dead and live loads, determine the required ultimate strength capacity for member design. The ultimate strength design method based on elastic behavior of the structure is recommended for all types of structures, including steel structures. Load factors are suggested where deemed necessary.

Drift limitations are specified for the deformation response due to the Damage Deformation Spectrum (DDS) and secondary effects and structural stability are to be investigated at the Damage and Condemnation Deformation levels.

The ductility demand resulting from the Condemnation Deformation Spectrum response may affect the choice of the structural system and the detailing requirements for various elements such as boundary elements in shear walls and spandrel beams. In some cases, the CDS analysis may render it advisable to increase the strength of certain elements to keep the ductility demands below acceptable values.

IV-10 The Role of Dynamic Analysis in Seismic Design

Dynamic analysis, either in response spectrum or time history form, has been prescribed by various recent seismic design recommendations and codes. This analysis may be an allowable alternative (or even a necessary requirement for special structures), as in the Uniform Building Code (Reference 5). However, nowhere in these seismic provisions, is there given a definite and complete procedure of design based on a dynamic analysis. It is therefore the objective of this project to provide this very much needed complete procedure based on the response spectrum method. In addition to a more accurate determination of structure periods and lateral load distribution, the method allows the designer to have a direct physical and practical understanding of each step in the design procedure as it relates to seismicity and the related structural behavior. It is felt that this understanding is more important in a design procedure than the use of high design load values in order to create structures which can perform adequately during strong ground motion.

IV-11 Design Methodology

The design method is to be developed in terms of the following basic topics:

(1) Design objectives of damage control and condemnation prevention

(2) Seismicity in the form of an Iso-Contour Map and return periods

(3) Use classes of structures

(4) Types and behavior of structural systems

(5) Effective response spectra

(6) Design spectra

(7) Calculation of response

(8) Load combinations

(9) Member design

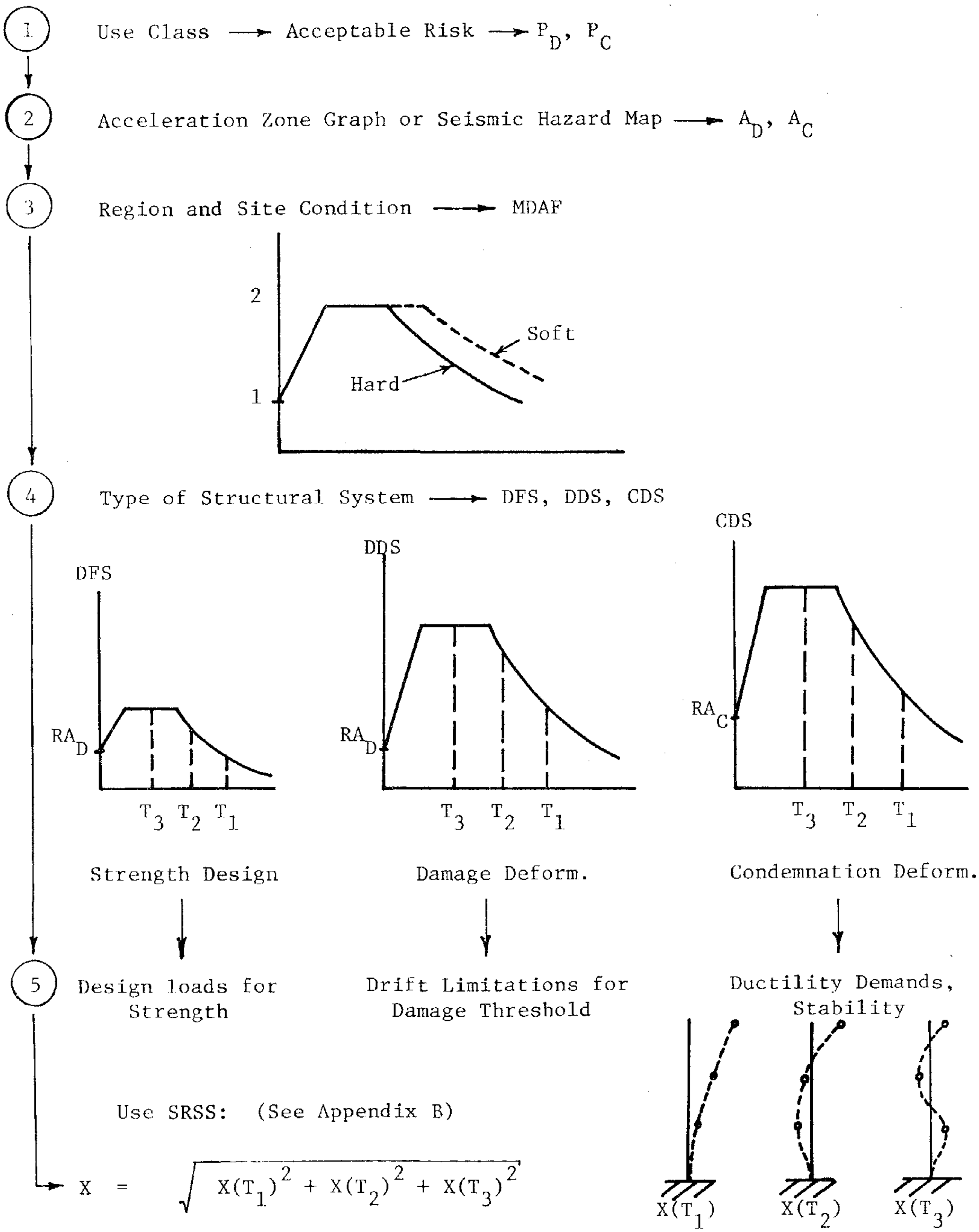
(10) Deformation analysis

A flow chart representation of the design procedure is given in Figure 4-1.

It is important to note that all the procedures presented here for seismic load levels, analysis and design of structures, are in the form of general methodology. They are meant to be used as guidelines in any future development of specific seismic code requirements.

IV-12 A Comparison of the 1974 SEAOC Recommendations and the Proposed Design Method

In order to best appreciate the proposed methodology the following summary comparison is presented between the 1974 SEAOC approach and the approach developed in this report. (Reference 6).



FLOW CHART OF DESIGN PROCEDURE

FIGURE 4-1

1974 SEAOC Recommendations

The base shear for ultimate strength design according to this code is given by,

$$V_B = Z\ I\ U\ C\ S\ K\ W \qquad 4\text{-}1$$

V_B = Base shear to be distributed to each story according to a linear "empirical" version of dynamic analysis.

Z = Seismic Zone Factor based on magnitudes of earthquakes in a region - but not on their frequency or chance of occurrence.

I = Structure Importance Factor. This value is greater than unity for essential facilities - however it is not related to a definite acceptable value of risk.

U = Load Factor to convert from a working stress level to an ultimate strength design basis for proportioning structural members.

C = An empirical shape factor for an inelastic multi-mode acceleration response spectrum. This is only a rough approximation of the statistical average of spectral shapes for the given region.

S = Site Response Factor for the influence of the underlying soil column and structure interaction on the spectral shape as represented by C. It is a number larger than unity when the site period is near the structure period.

K = A reciprocal measure of the ductility of a given lateral force resisting system. This value adjusts the inelastic response spectrum shape C so as to represent a reduction of lateral loads for ductile system and an increase for non-ductile system.

W = Weight of the structure taken as dead load only - with no ambient live load.

Within the actual design procedure, the following observations can be made.

- Strength Design for Members is for the Force effects of V_B together with factored dead plus live load effects.

- There is no specific requirement for a verification of stability and condemnation protection at the major earthquake level (except for a special requirement for vertical load carrying members at about 4 times working stress design deformation).

- There is no consideration of modal participation and effect of mode shapes on lateral load distribution.

Proposed Design Procedure

Base shear and Lateral Design Load are given by the SRSS Modal response to the Design Force Spectrum

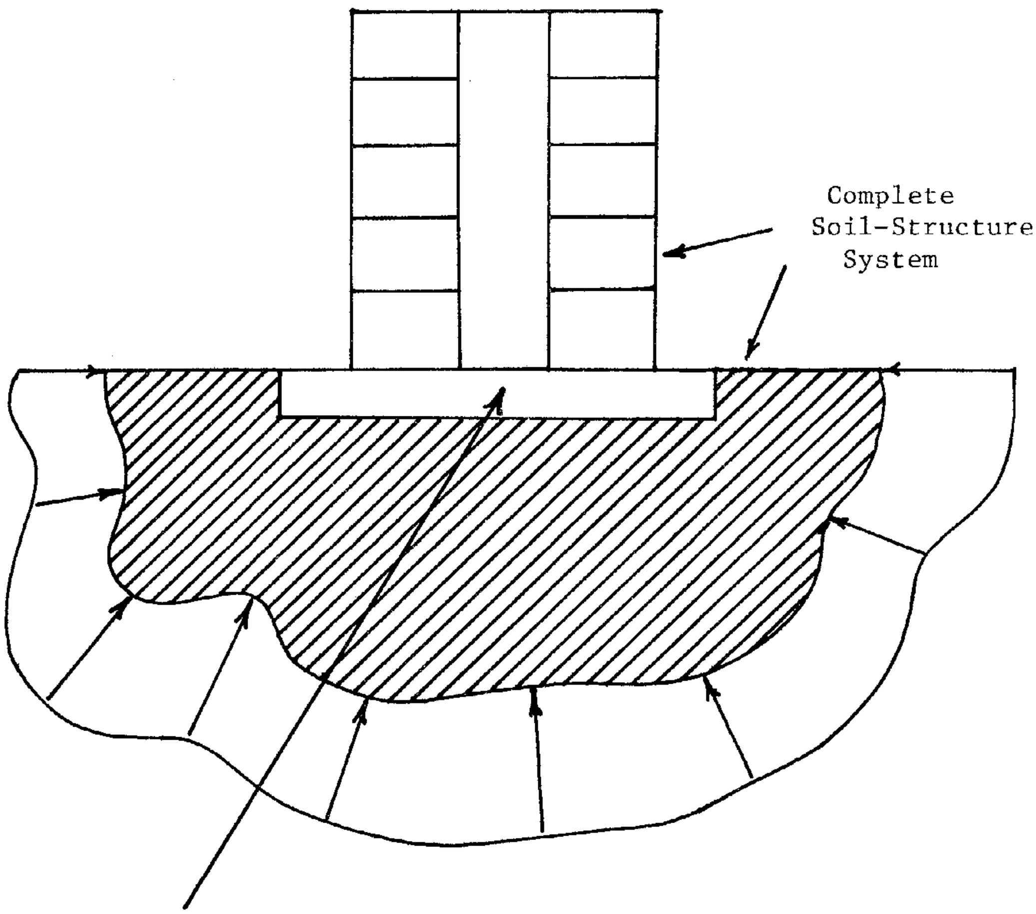
$$DFS = R \cdot A_D \cdot (MDAF) \frac{1}{d_T} (1 + k_T V_S) \qquad 4\text{-}2$$

R = A Peak Acceleration Reduction Factor to represent the Effective Acceleration on the Structure. It represents the spacial average of Peak Accelerations on the effective soil-structure system. See Figure 4-2 and Chapter VII.

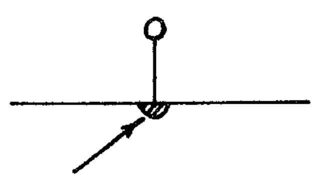
A_D = Peak Ground Acceleration at Structure Site - having acceptable risk of being exceeded. If A_D is exceeded, then extensive structure damage may occur. See Chapter V.

MDAF = Mean or Statistical Average of Acceleration Response Spectrum Shapes for the region. The shape can include any soil-column response effects, and together with R can represent soil-structure interaction effects. See Figure 4-3 and Chapter VI.

d_T = Damage Deformation Factor for a given lateral force resisting system. It represents the ratio between the maximum acceptable deformation at

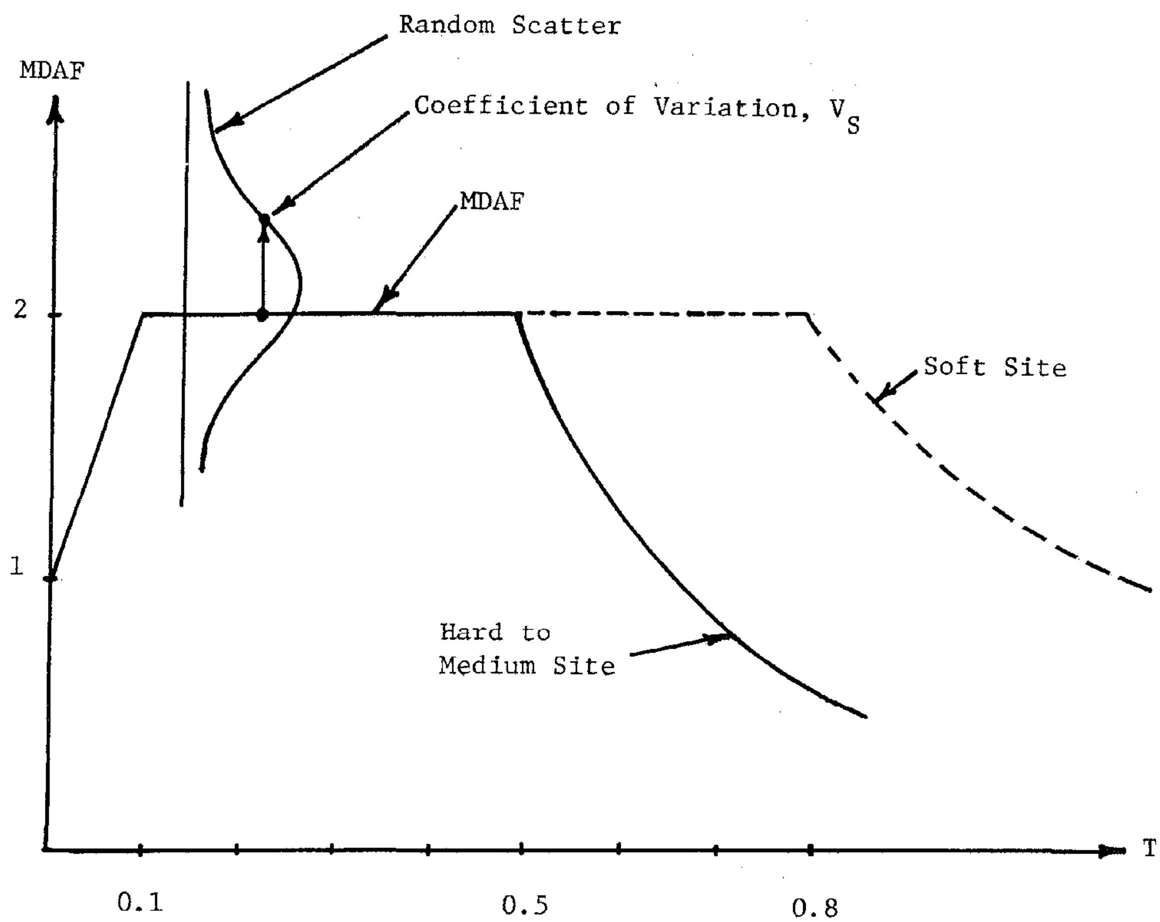


R · PGA = Surface Average of the Distributed Peaks at all points of the soil-structure system.



PICTORIAL REPRESENTATION OF R-FACTOR

FIGURE 4-2



STATISTICAL PROPERTIES OF THE DAF SPECTRAL SHAPE

FIGURE 4-3

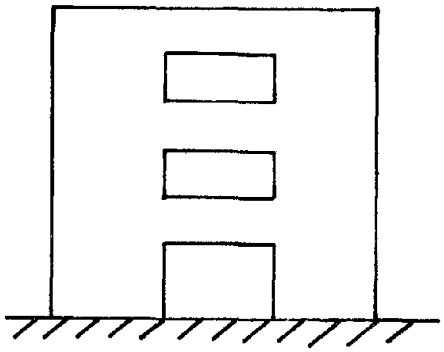
d_T (cont.) the damage earthquake level and the design deformation in the highest stressed member. The d_T value depends on the K-factor type of the system. See Figure 4-4 and Chapter VIII.

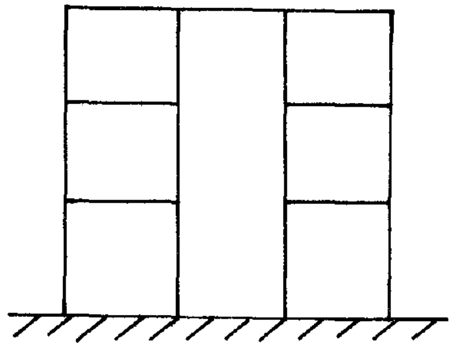
$(1 + k_T V_S)$ = Spectral Confidence Interval Factor, where V_S is the coefficient of Variation of the spectral shape, and k_T sets the confidence level. The factor k_T allows for the degree of reliability, inherent in a system, of attaining the given d_T distortion value without excessive damage. If a system is very reliable then k_T may be zero. See Figure 4-5 and Chapter IX.

The k_T value depends on the quality or grading of A, B, or C of a given structural system. See Figure 4-5 for relation of confidence levels and the system grade of reliability.

Member seismic design forces are found by the SRSS value of the individual mode response to the DFS. In the formulation of the dynamic model the full dead load and some reasonable fraction of the live load (0.4L) is considered.

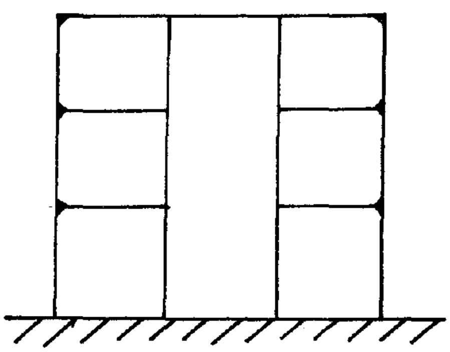
Type K = 1.33 | 1.00

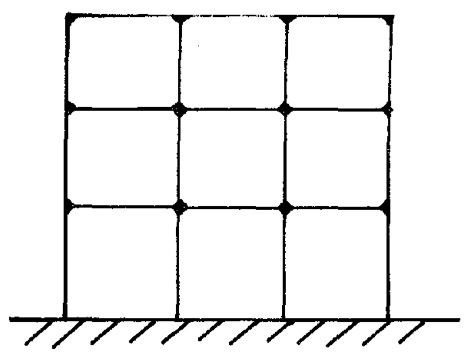




d_T = 1.5 | 2.0

Type K = 0.80 | 0.67

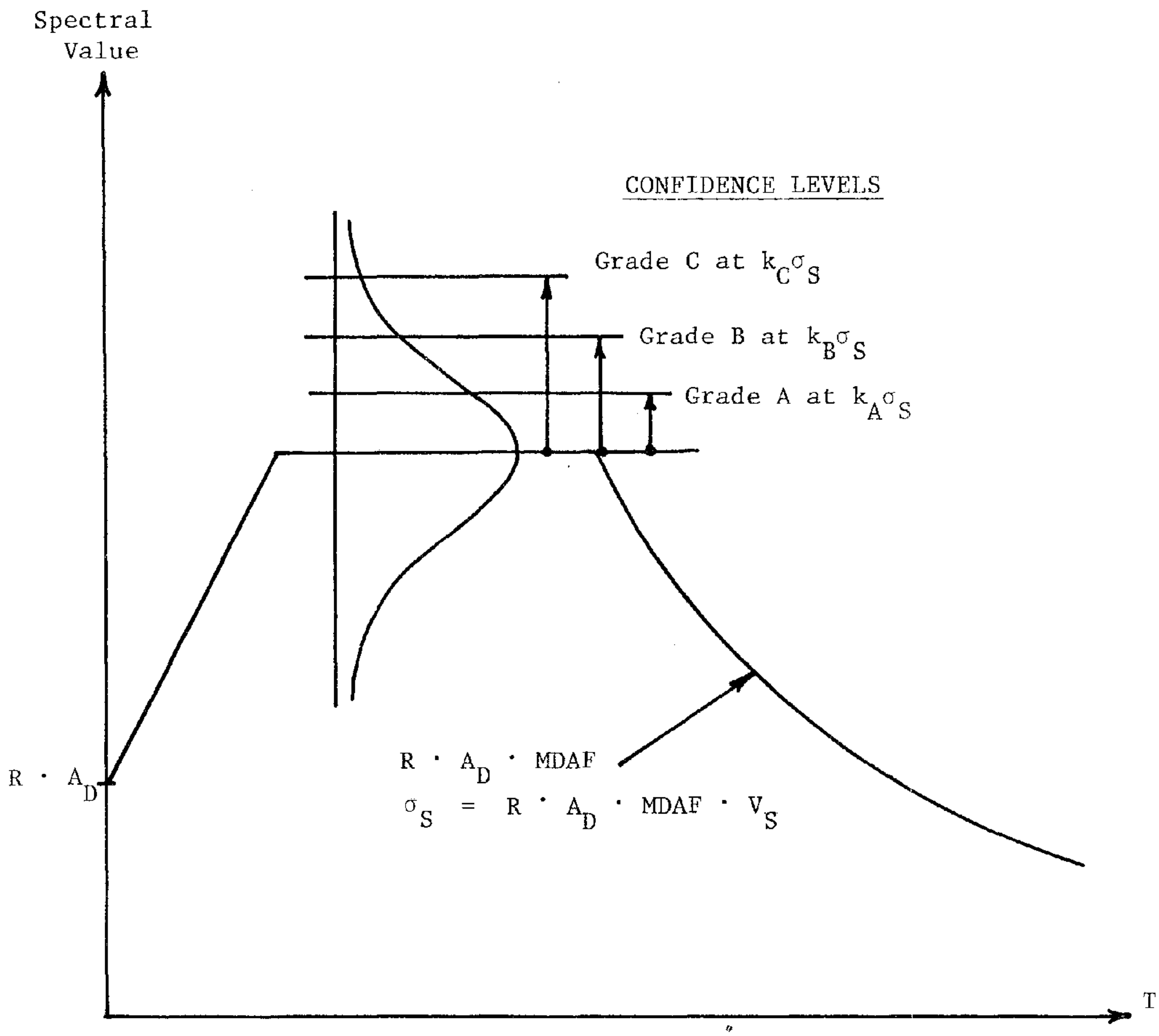




d_T = 2.5 | 3.0

d_T FOR THE VARIOUS TYPES
OF LATERAL FORCE RESISTING SYSTEMS

FIGURE 4-4



RELATION OF GRADE TO DESIGN LEVEL

FIGURE 4-5

Within this proposed approach, the following comments are pertinent.

- Strength Design for Members is the Force Response of the DFS plus dead load and a reasonable fraction of ambient live load (0.4L).

- Non-Structural Damage Control is verified at the SRSS modal deformation response to the Damage Deformation Spectrum

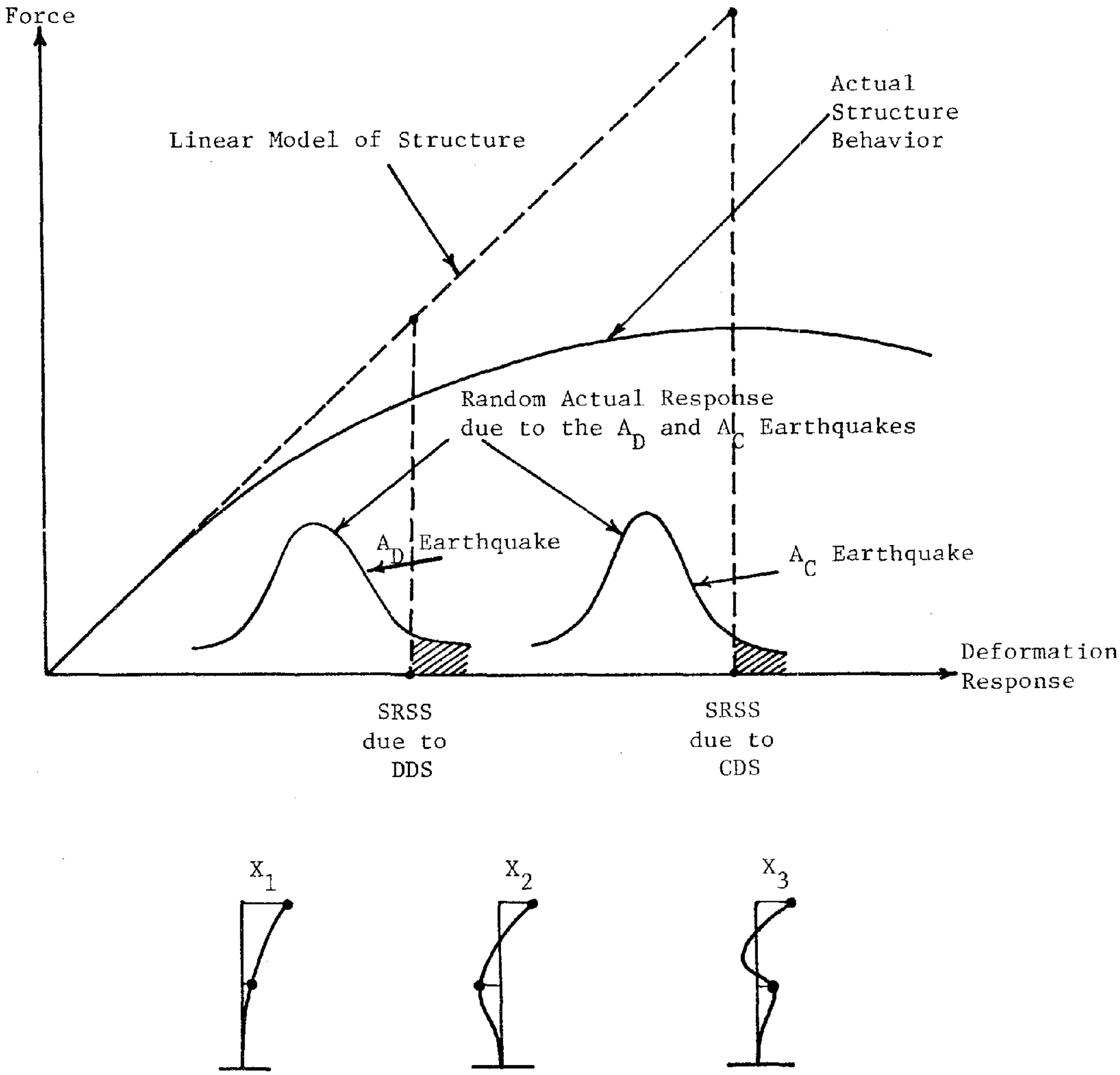
$$DDS = R \cdot A_D \cdot (MDAF)\ (1 + k_T V_S) = d_T DFS \qquad 4\text{-}3$$

See Figure 4-6, for the relation of the linear model method of calculating SRSS response - to actual unknown random response to a given earthquake.

This is a most important phase of the design procedure - since it requires the designer to consider the flexibility of the structure with respect to damage to the architectural, utility, and service facilities. These items represent a considerable portion of the structure value, and may be necessary for life safety.

- Local Member Ductility Demand and Structure Stability verified at the SRSS modal deformation response to the Condemnation Deformation Spectrum,

$$CDS = R \cdot A_C \cdot (MDAF)\ (1 + k_T V_S) = \frac{A_C}{A_D}\, d_T DFS \qquad 4\text{-}4$$



$$SRSS = \sqrt{X_1^2 + X_2^2 + X_3^2}$$

RELATION OF LINEAR MODEL COMPUTED RESPONSE TO ACTUAL STRUCTURE RESPONSE

FIGURE 4-6

A_C = PGA value corresponding to the condemnation level seismic event. See Figure 4-6. Local member deformations are compared against their yield level deformations to assess whether ductility demands are within allowable limits.

A numerical comparison of the two methods summarized above can be found in Appendix D.

Basic Philosophy of the Proposed Seismic Design Procedure

In the design spectra, such as

$$DFS = R \cdot A_D \cdot (MDAF) \frac{1}{d_T} (1 + K_T V_S) \quad \text{4-2 (repeated)}$$

it should be noted that a very simplistic and approximate representation is given for some very complex phenomena. For example

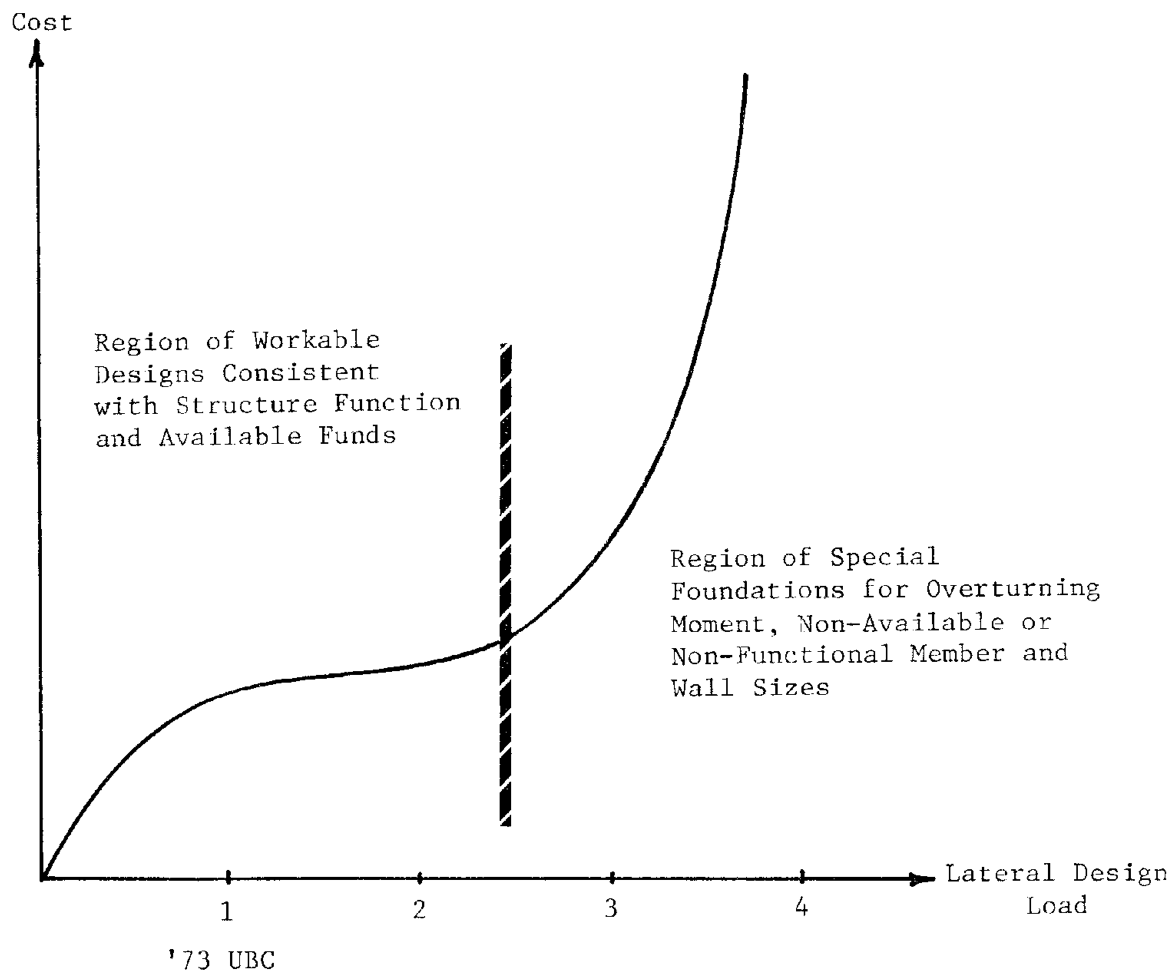
- R represents all soil-structure interaction effects
- d_T and the β_T of the MDAF account for both damping and the non-linear system effect of "tuning out" of harmonic response.
- The MDAF has two simple shapes to allow for the soil column response effects.

Obviously a more complex representation of these and other structure response phenomena could have been proposed in order to better predict the effects of a future seismic event - the net result would be higher or lower design load levels based on the specific structure and site conditions.

However, for this proposed design method, the following general philosophy has been adopted - given realistic seismic design load levels at the ultimate strength level the accuracy in prediction of future seismic loads is not particularly necessary for the attainment of the design objectives of damage protection and condemnation prevention. The insensitivity to the cost of providing lateral load resistance within a certain range is illustrated in Figure 4-7. The principal element of the design philosophy is to provide procedures which will create a good seismic resistant system having:

- at the damage threshold earthquake response
  - adequate strength and stiffness for damage control
- at the condemnation threshold earthquake
  - no excess of inelastic deformations beyond the failure capacity of members, and
  - no large unbalance of inelastic deformation in any story level of the elevation, or in any wall or frame line of the structure plan.

The proposed design procedure is based on this "good system" (rather than "precise load") philosophy and can attain the objectives by following the basic criteria of a response spectrum method.



ECONOMIC AND ENGINEERING CONSEQUENCES OF LATERAL LOAD CRITERIA ASSUMING EQUAL DESIGN PROCEDURES AT MULTIPLES OF THE 1973 UBC

FIGURE 4-7

CHAPTER V

DESIGN PHILOSOPHY AND ACCEPTABLE RISK

SCOPE

In this chapter, the design philosophy based on the concept of acceptable risk for different uses of structures is presented. Specifically, PGA values for damage level A_D and condemnation level A_C are suggested for an "acceptable" risk level.

....

V-1 Introduction

From the information as developed in the preceding chapters, peak ground acceleration values may be established for a given structure location. These values have selected probabilities P of not being exceeded during a given economic structure life L. The purpose of this chapter is to show how these acceleration values are to be incorporated into load criteria for seismic design provisions. Basically, acceleration values must be converted to seismic load information, such that structures, as designed for these load levels, will have a desired reliability R_D of damage protection and a much higher reliability R_C against total building condemnation or incipient collapse during the economic structure life.

While at first thought a building owner may desire full protection against both the hazards of damage and condemnation, a consideration of the complete set of his objectives will show the necessity for acceptance of some level of risk. For a given site location, structure life, and Use Class or Function, these objectives of the building owner are:

- Low construction cost
- Low Operating cost
- Functional configuration
- Attractive configuration
- Damage protection
- Condemnation prevention

Perfect and certain fulfillment of all of these objectives is not possible due to the uncertainties in earthquake demands and in structural capacities and behavior. Practical fulfillment of the first four objectives requires the acceptance of a moderate probability of damage P_D (equal to $1-R_D$) and a small probability of structural condemnation P_C during the building's economic life, L. Planners, therefore, must agree to a definite set of values for P_D, P_C, L for the given value, and Use Group of the building. In Reference 1, a discussion on this aspect of risk, economic life and return period was presented in Chapters V & VI.

For these given values of P_D, P_C, and L, the Acceleration Zone Graphs (AZG) or the Iso-Contour Map provide the Peak Ground Acceleration values A_D and A_C which have the moderate P_D and small P_C probabilities of exceedence during the structure life L at a given site location.

The use or function of structures may be organized into the following classes which depend on the desired reliabilities of operation and damage protection in the event of a large earthquake.

Class 1: Critical facilities necessary for life care and safety; hospitals; penal and mental institutions; gas, water, electric, and waste water treatment facilities; communications facilities; police and fire departments; and disaster control centers.

Class 2: Family residences; hotels; recreational and entertainment structures; churches and schools; commercial and industrial structures necessary for normal commerce.

Class 3: Facilities which are relatively non-essential for normal commerce and where damage will not create a life safety hazard. An example of such facilities would be warehouses.

The Vice Ministry of Urban Planning in Managua has recommended an alternate use classification scheme. This scheme is primarily intended as a planning matrix for land use. However, the use group can be developed from the categories mentioned in that table. See appendix C for this table.

Example values of the peak ground accelerations A_D and A_C, at sites in Managua and Leon, are given in Table 5-1, 5-2, 5-3, and 5-4. These are based on structure lives of 20, 50 and 100 years, and on reasonable values for P_D and P_C corresponding to the structure Use Class. Note that the damage risk per year for class 1 structure is one fifth the damage risk for class 2 structure. Similar statements can be made for condemnation risk for all three suggested use classes. The values given in these table are strictly for demonstrating the concepts, and are not meant to be final. As can be seen from these four tables, the same facility and risk in Leon and Managua requires different A_D and A_C values. Obviously, Leon has a lower seismic demand than Managua.

Table 5-1. Managua Region

Suggested Damage "risk" levels

Class	Economic life Yrs.	RP_D Yrs.	P_D	"Risk"/yr.	A_D g units
1	100	500	0.20	.002	.45
2	50	100	.40	.01	.35
3	20	50	.40	.02	.30

Table 5-2. Managua Region

Suggested Condemnation "risk" levels

Class	Economic life Yrs.	RP_C	P_C	"Risk"/yr.	A_C g units
1	100	1000	.1	.001	.47
2	50	500	.1	.002	.45
3	20	100	.2	.01	.35

Table 5-3. Leon Region

Suggested Damage "Risk" Levels

Class	Economic Life Yrs.	RP_D	P_D	"Risk"/Yr.	A_D g units
1	100	500	.20	.002	.30
2	50	100	.40	.01	.25
3	20	50	.40	.02	.21

Table 5-4. Leon Region

Suggested Condemnation "Risk" Levels

Class	Economic Life Yrs.	RP_C	P_C	"Risk"/Yr.	A_C g units
1	100	1000	.1	.001	.35
2	50	500	.1	.002	.30
3	20	100	.2	.01	.25

V-2 Design Objectives

With these known values of A_D and A_C at the structure site, the primary objectives of the structural designer are to:

- Provide a structure with sufficient rigidity such that no significant non-structural damage will occur due to earthquake ground motions of a level represented by A_D.

- Provide a structure with sufficient strength capacity such that no significant structural damage will occur due to deformation demands caused by earthquake ground motions of a level represented by A_D.

- Provide a structure with sufficient strength, stability, and deformation capacity such that condemnation of the structure will not result from the effects of earthquake ground motions of a level represented by A_C.

- While the possibility of significant damage is admissible with the moderate probability P_D, and the possibility of building condemnation is admissible with the small probability P_C, every prudent effort is to be made to prevent serious injury or death of the building occupants. This life safety objective requires that the details of both the structural and non-structural elements, and the complete structural system are such that neither injurious system failures, injurious falling debris, nor structural collapse will result from ground motions of a level represented by A_C.

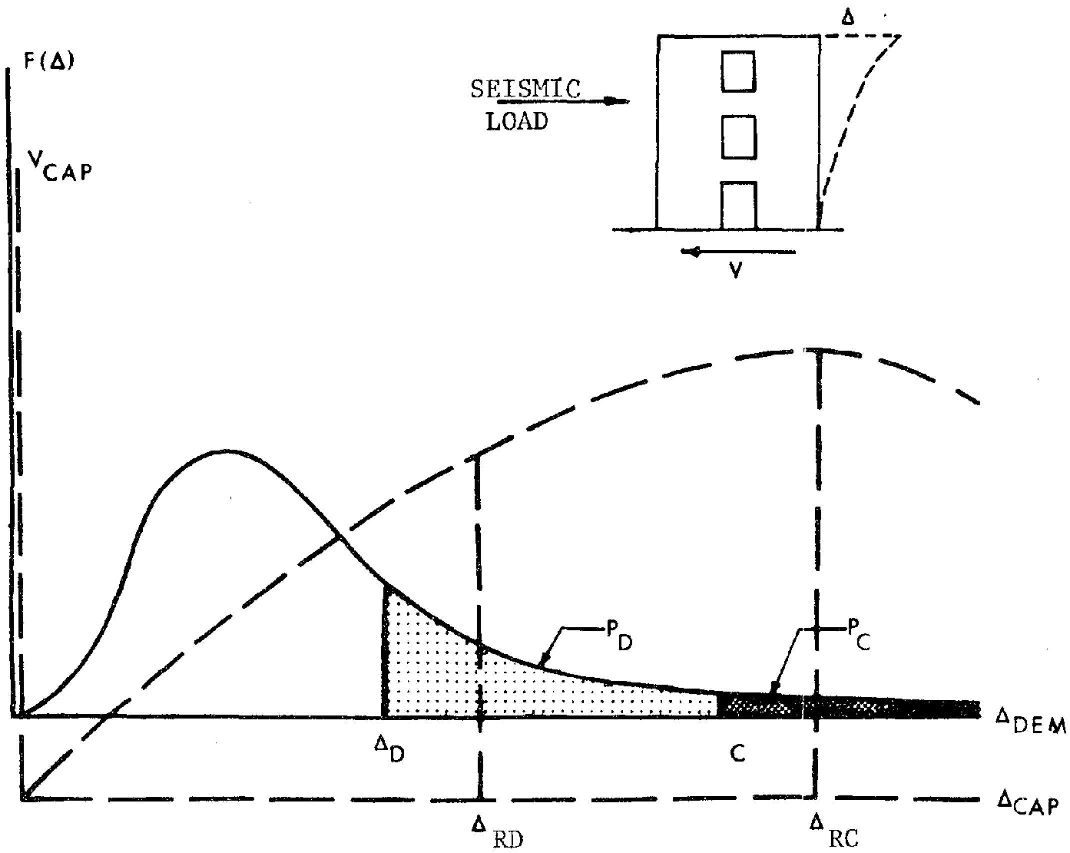
The practical consequence of this last objective is that only those types of structural systems which are capable of retaining their integrity and stability at deformations at and beyond the A_C level are to be used.

Within these systems, the details of the connections between structural elements must tie the structure together, and the elements themselves must not have brittle or sudden buckling modes of failure. Multiple systems of frames, or back-up systems in the form of shear walls or vertical bracing must provide a series of lateral force resisting systems such that vertical load capacity is maintained for earthquake deformation demands at and reasonably beyond the Δ_C level.

The complete set of structural design objectives is shown in Figure 5-1. Since the demands of earthquake ground motions create nonlinear structural behavior, this figure indicates the critical design thresholds of damage Δ_D and condemnation Δ_C in terms of structure deformation Δ rather than forces. The solid line coordinate system represents the probability density function f (Δ) of Earthquake Deformation Demands Δ_{DEM} which may occur on a given structure during a life L. The dotted line system indicates the load V versus deformation capacity Δ_{CAP} curve of a given structure which satisfies the stated design objectives. Specifically, the structure has been designed such that its deformation capacities are equal to or greater than the earthquake demands at the damage and condemnation threshold levels. (Note that Δ_{RD} is greater than Δ_D and Δ_{RC} is greater than Δ_C) The earthquake of level A_D with probability of exceedence P_D does not exceed the damage capacity level Δ_{RD} and the earthquake having the condemnation level A_C with probability P_C, does not exceed the condemnation capacity level Δ_{RC}. Further, the structure load-deformation curve maintains a reasonably constant level for even those highly improbable deformations which might reasonably exceed the condemnation level. This latter characteristic insures the stability of the structure against collapse. Methods for achieving these objectives are discussed in later chapters.

V-3 Structure Use Classification

The classification of structures according to their use or function as stated in the introduction to this chapter as Class 1, 2, and 3,



Δ_{CAP} = Deformation Capacity

V_{CAP} = Shear Capacity

Δ_D = Demand of Earthquake of level A_D

Δ_C = Demand of Earthquake of level A_C

Δ_{RD} = Damage Threshold Deformation

Δ_{RC} = Condemnation Threshold Deformation

P_D = Probability of $\Delta_{DEM} > \Delta_D$

P_C = Probability of $\Delta_{DEM} > \Delta_C$

FIGURE 5-1

determine the acceptable risk levels for damage control and condemnation protection. The following values are suggested for these acceptable risks, economic lives, and return periods. Appendix G provides some risk data on natural and man-made hazards.

Table 5-5. Suggested Return Periods

Use Class of Structures	Suggested Economic Life (years)	Suggested Return Period (years)	
		Condemnation	Damage
1	100	1000	500
2	50	500	100
3	20	100	50

Thus, for values suggested in Table 5-5, the risk levels for different classes are:

<u>Class 1</u>

(i) Risk of exceeding condemnation level loading per year = 0.001

Risk of exceeding condemnation level loading during 100 year economic life is 0.10.

(ii) Risk of exceeding damage level loading per year = 0.002.

Risk of exceeding damage level loading during 100 year economic life is 0.20.

Class 2

(i) Risk of exceeding condemnation level loading per year = 0.002.

Risk of exceeding condemnation level loading during 50 year economic life is 0.10.

(ii) Risk of exceeding damage level loading per year = 0.01.

Risk of exceeding damage level loading in 50 years = 0.40.

Class 3

(i) Risk of exceeding condemnation level loading per year = 0.01.

Risk of exceeding condemnation level loading in 20 years of economic life is approximately 0.20.

(ii) Risk of exceeding damage level loading per year = 0.02.

Risk of exceeding damage level loading in 20 years of economic life is approximately 0.40.

As an example, consider the Managua Region. The PGA values corresponding to different return periods are given in the following table. (Obtained from AZG for Managua).

Table 5-6

RP years	PGA in g units
1000	.47
500	.45
100	.35
50	.30

Similarly, the values for Leon would be as given in the following table.

Table 5-7

RP Years	PGA in g units
1000	.35
500	.30
100	.25
50	.21

It should be emphasized that the values suggested in Table 5-5 should be used for the whole country. The level of the PGA corresponding to these suggested return periods (and hence risk) will change from region to region, based on its seismicity or seismic hazard. This concept of consistent risk is very important in developing a rational design and code formulation.

V-4 Response Spectrum Analysis

Referring back to Figure 5-1, it is necessary for the designer to have some analytical method of computing the earthquake demands of Δ_D and Δ_C. The method to be employed is modal analysis as described in Appendix B. Briefly, this consists of the following steps:

- A linear elastic dynamic model of the structure is formulated, and the characteristic mode shapes and frequencies are evaluated.
- For any given Response Spectrum, the force and displacement response of the linear model are assumed to be

given by the square root of the sum of the squared response of each mode. This is termed as SRSS response.

- Design spectra are to be formulated (in a following chapter) such that: the SRSS response to the Damage Threshold Spectrum provides the demand Δ_D, and the SRSS response to the condemnation Threshold Spectrum provides the demand Δ_C. Since both Δ_D, and Δ_C, may be inelastic deformations, it is necessary to employ the assumption that inelastic structure deformations may be predicted by the elastic dynamic model response to the specially formulated inelastic design spectra.

With the stated design philosophy and the response spectrum method of analysis, the basic objectives are that when the design spectra are employed as input to the method of analysis and with the element design procedure, the acceptable reliabilities of damage protection and condemnation prevention will be achieved in the as-designed structure. These design spectrum levels are functions of:

- structure use class with its particular set of desired reliabilities (as discussed in this chapter).

- structural system type with its particular damping and inelastic deformation characteristics at the damage and condemnation thresholds; along with its reliability and quality control in terms of its subjective or actual performance record in resisting strong motion earthquakes. These parameters will be discussed in the following chapters.

CHAPTER VI

DEVELOPMENT OF THE DYNAMIC AMPLIFICATION FACTOR SHAPE STATISTICS

SCOPE

Having described the general philosophy and summary of approach for the proposed method, a detailed commentary for each individual parameter is presented in the following chapters. In this chapter MDAF and V_S are defined and evaluated. These two factors appear in the Design Force spectrum equation 4-2.

$$DFS = R \cdot A_D \cdot (MDAF) \frac{1}{d_T} (1 + k_T V_S) \qquad \text{4-2 (repeated)}$$

....

VI-1 Introduction

The PGA value given by the Acceleration Zone Graphs or Iso-Contour Map for a given return period is a prediction or forecast of a future seismic event. This future event will have an accelerogram or acceleration time history characterized by the particular PGA value given by the graph or map. However this PGA value by itself does not provide sufficient information concerning the future time history or accelerogram. This required information is most practically represented in the form of a response spectrum. The method of obtaining this predicted spectrum is as follows.

As mentioned in Chapter IV, for a given region with known (overall) geologic characteristics, a sample set of past major earthquake accelerograms and their corresponding response spectra can be assembled.

This data set may be from the region for which seismic design criteria are to be developed or from geologically similar regions. Each response spectrum is then scaled so as to have a unit value of peak ground acceleration (PGA), and is hence termed as a dynamic amplification factor (DAF). This sample data is then statistically analyzed to obtain the mean and the variance of the DAF shape. From this sample mean shape, a simplified practical shape (MDAF) is then adopted. This practically usable shape may be adjusted for known hard or soft soil column effects at the site. Given any forecasted PGA value, the acceleration response spectrum may be obtained by multiplying the MDAF by the PGA value. The variance information regarding the DAF shape can be represented in terms of the coefficient of variation V_S (V_S = [standard deviation]/[mean value]). Later, when design spectra are formulated, this parameter V_S is used to establish the spectral confidence level corresponding to the type of structural system. This will be further explained in Chapters VIII and IX.

VI-2 Sample Mean Dynamic Amplification Factor (SMDAF)

The statistical analysis of the normalized (to PGA = 1g) response spectra for selected appropriate earthquake histories is given in Appendix A. See Figures 6-1, 6-2 and 6-3 for the resulting sample mean shapes (SMDAF) for the indicated damping values. Another important statistical quantity resulting from that analysis is the measure of the scatter of the individual normalized spectral ordinates about their sample mean value. See Figures 6-4, 6-5 and 6-6 for coefficient of variation behavior. It should be noted that the coefficient of variation (V_S) of the DAF shape

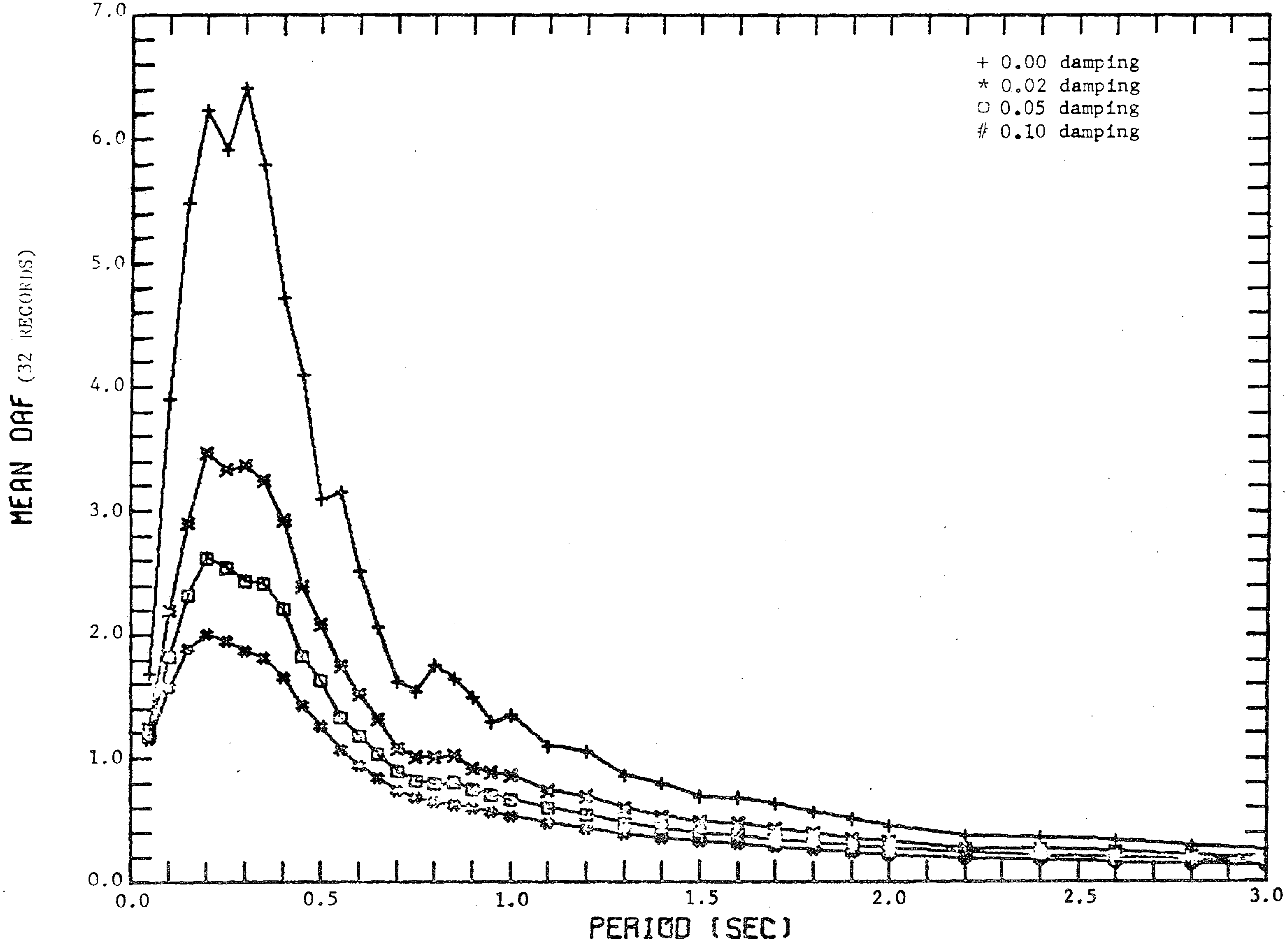

+ 0.00 damping
* 0.02 damping
□ 0.05 damping
0.10 damping
MEAN DAF (32 RECORDS)
PERIOD (SEC)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0

FIGURE 6-1

MEAN DAF (COMPLETE,[+] U.S.[*], MANAGUA[□]) VS. PERIOD, 0.05 DAMPING

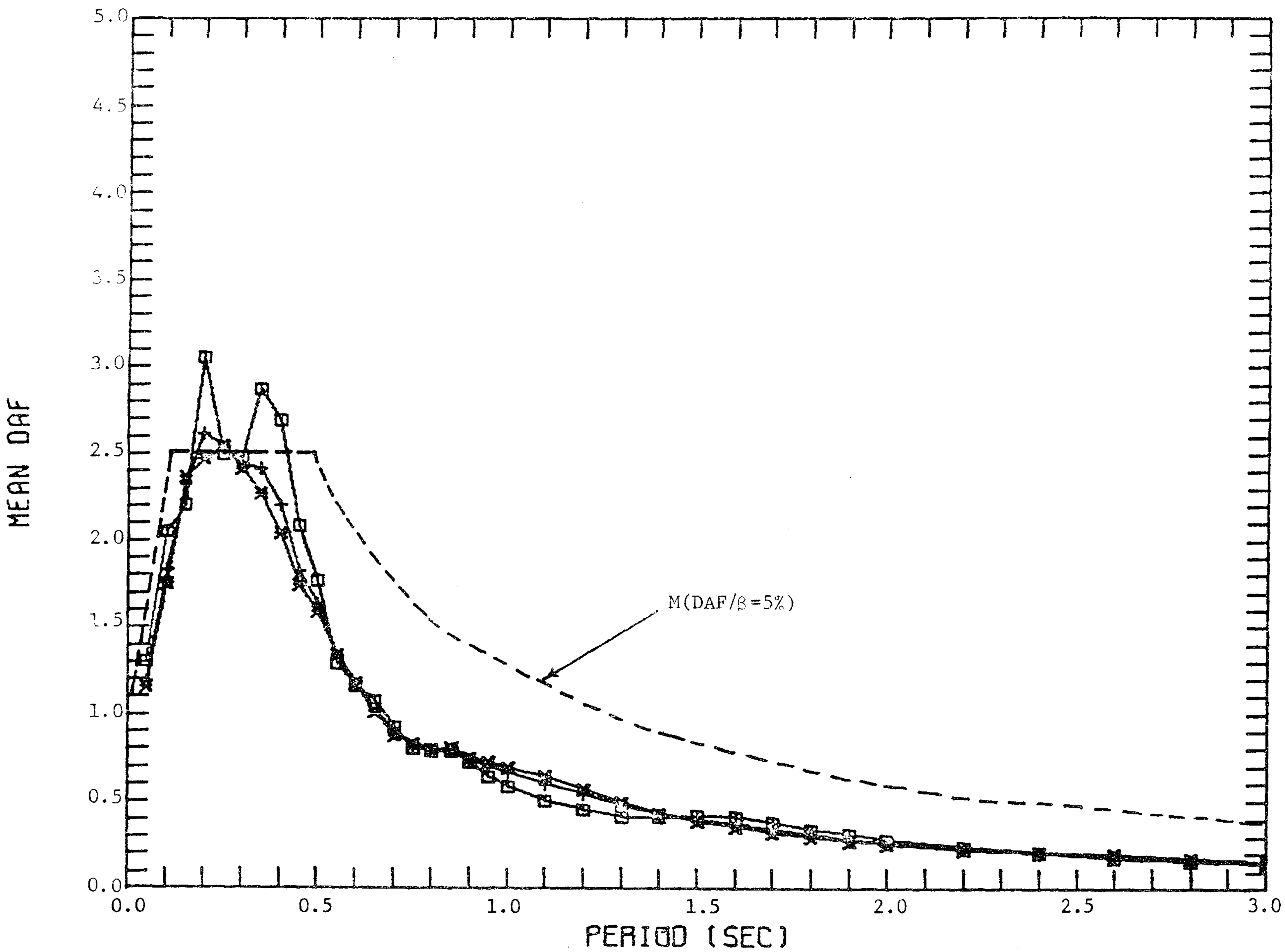


FIGURE 6-2

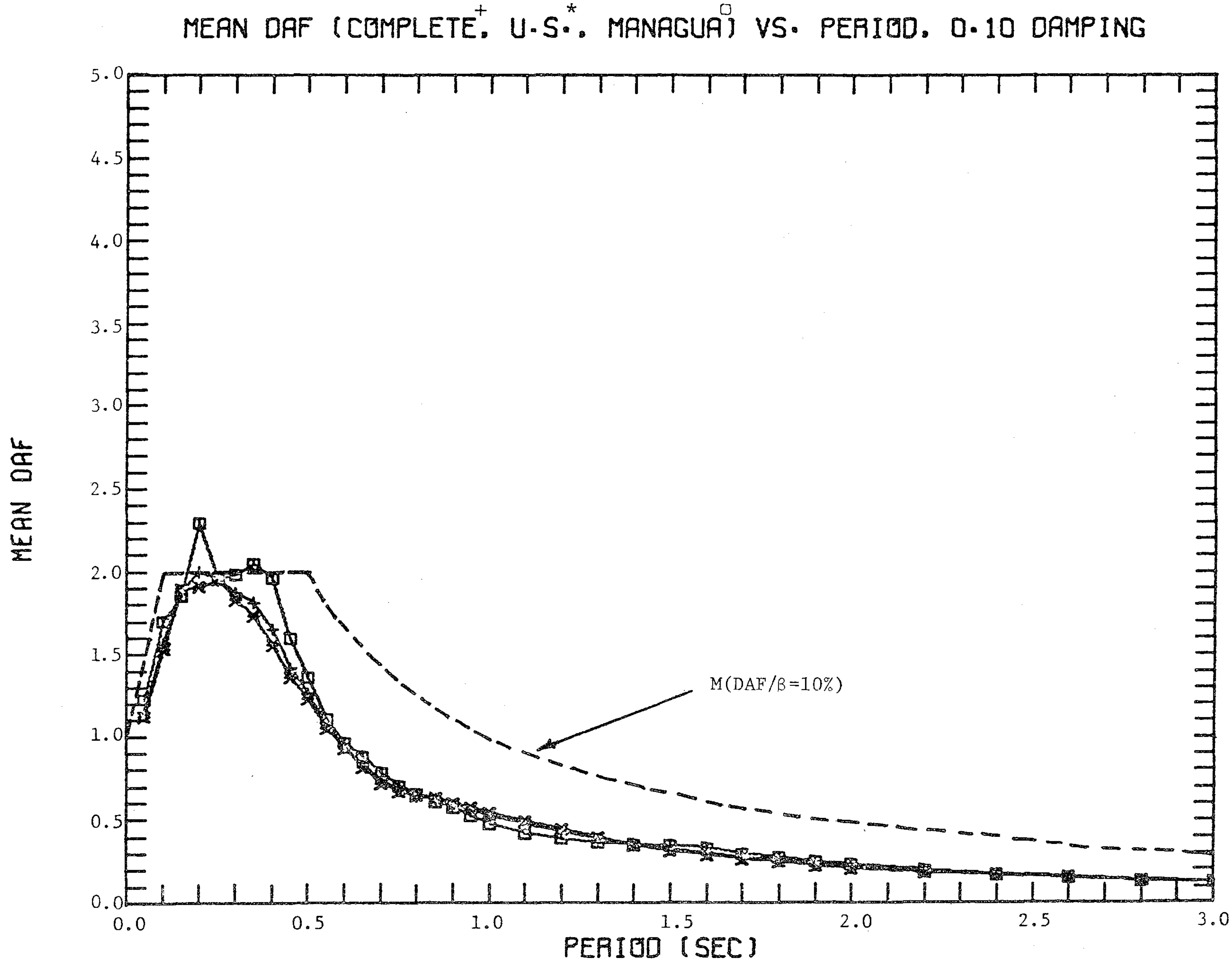

MEAN DAF (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.10 DAMPING
MEAN DAF
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.0
0.5
1.0
1.5
2.0
2.5
3.0
PERIOD (SEC)
M(DAF/β=10%)

FIGURE 6-3

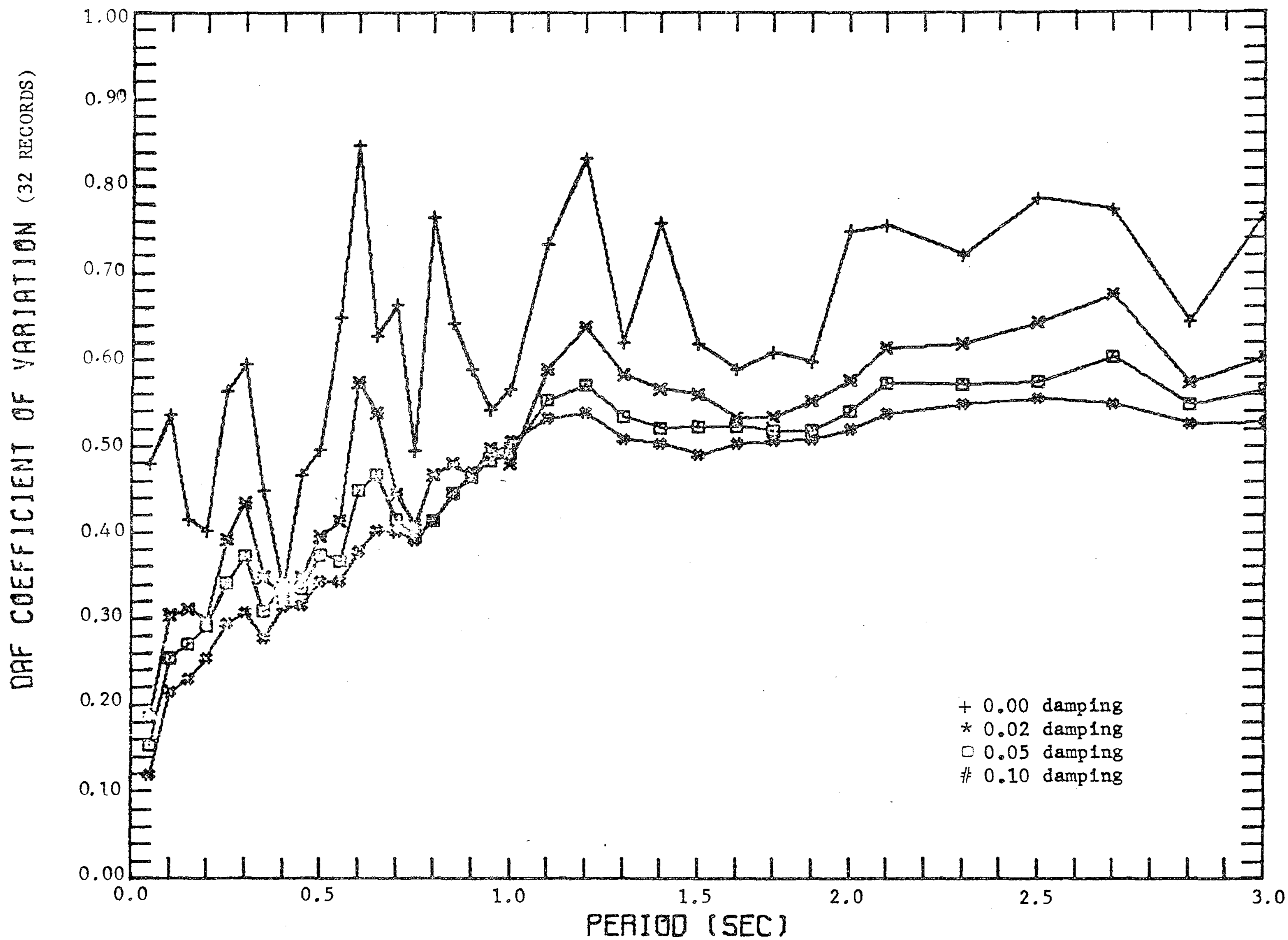

DAF COEFFICIENT OF VARIATION (32 RECORDS)
PERIOD (SEC)
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0
+ 0.00 damping
* 0.02 damping
□ 0.05 damping
0.10 damping

FIGURE 6-4

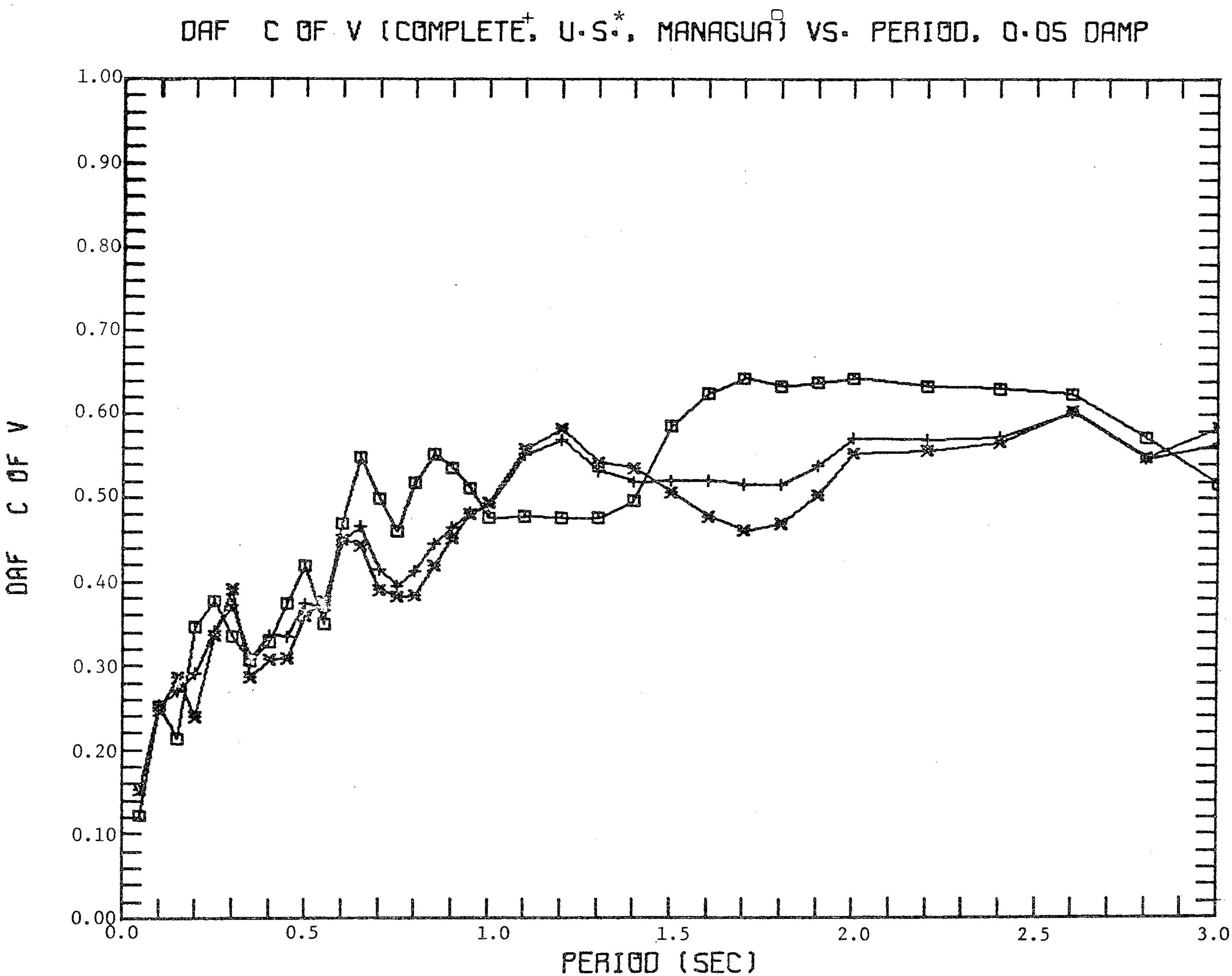
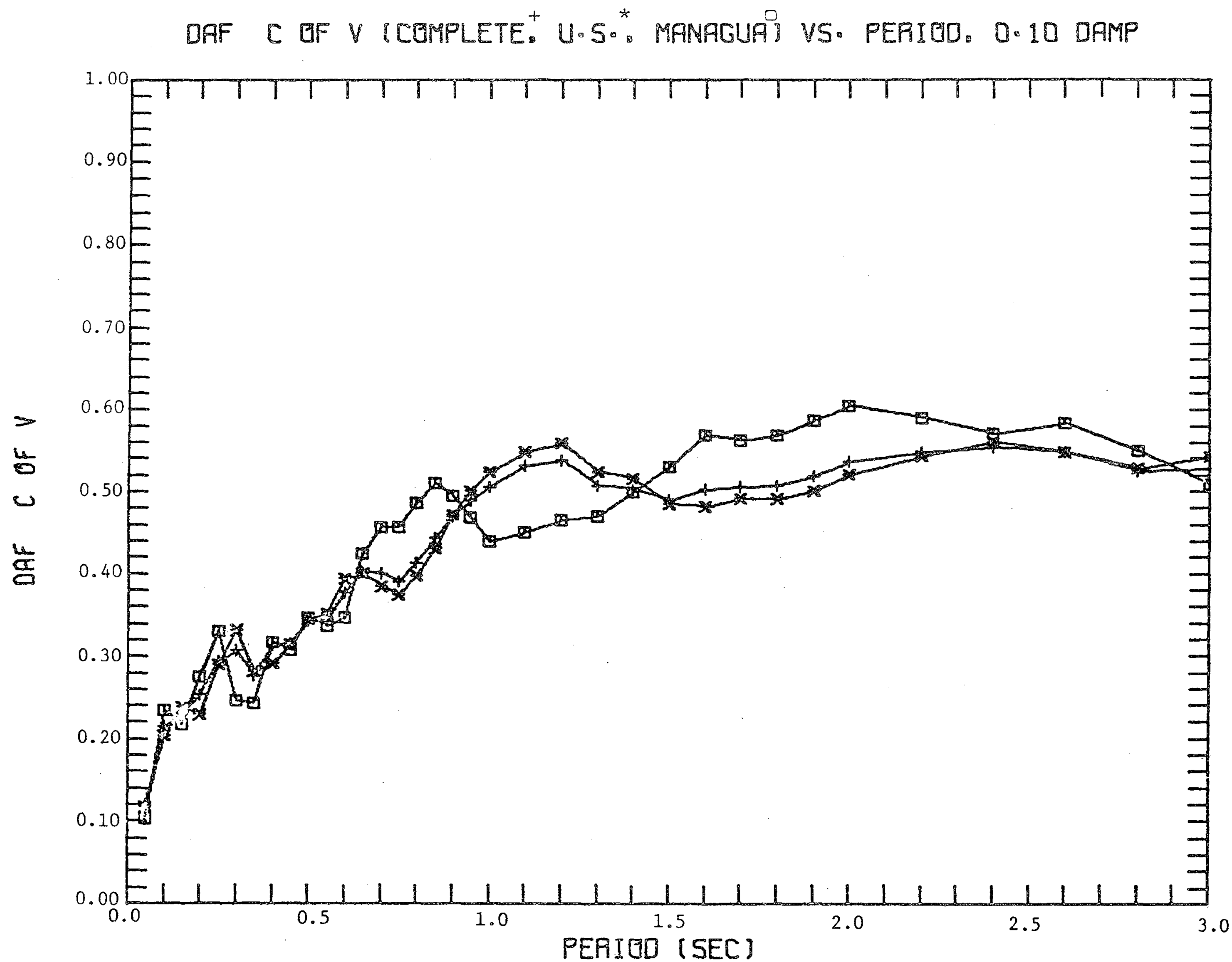

DAF C OF V (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.05 DAMP
DAF C OF V
PERIOD (SEC)
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0

FIGURE 6-5



DAF C OF V (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.10 DAMP
DAF C OF V
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0
PERIOD (SEC)

FIGURE 6-6

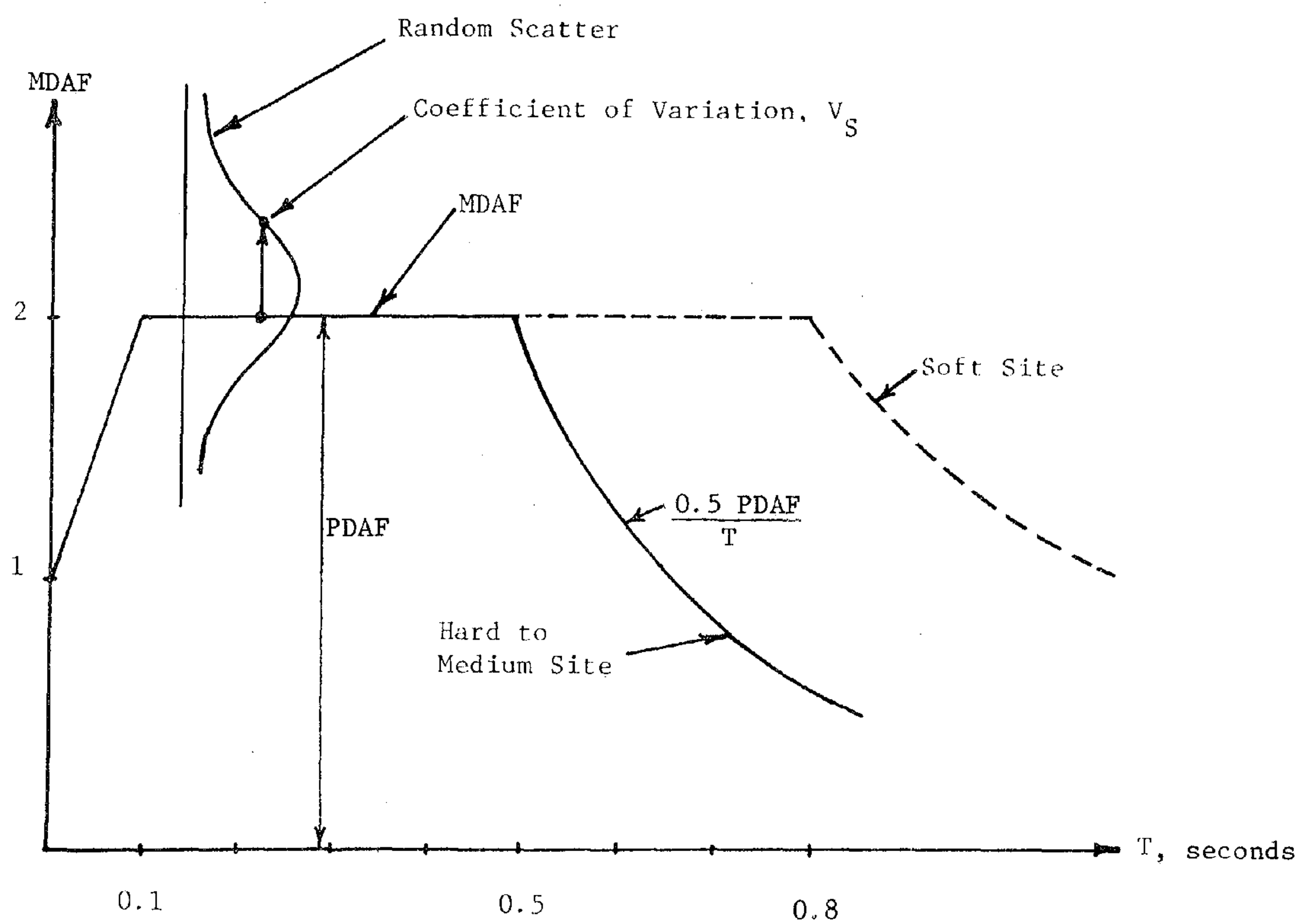
changes with period and damping. However, for practical purposes, it is necessary to select a constant value of this coefficient for a given damping. From Figure 6-6, it can be seen that for the range of periods of interest, (0.1 to 1 sec), an approximation of V_S equal to 0.4 is reasonable. (Future improved techniques of normalizing and defining spectra may lead to a lower value of V_S).

VI-3 Mean of the Dynamic Amplification Factor (MDAF)

The Statistical Analysis of Appendix A has some bias or weighting of local source and site behavior due to the ESSO refinery records. If distant major source records, such as from the Benioff Zone, were to have been available, then it is estimated that the sample mean shape would have been higher in the longer period region.

Just as the peak ground acceleration values at a given location represent the probabilistic combination or union of events from each of the possible earthquake sources, the response spectrum shape must similarly represent the effects of the events from each source. For a given PGA, a near shallow focus source would contribute to the short period region of the shape, and a distant deep-focus source would dominate the long period shape.

Therefore, with some judgment concerning the rounding of peaks which may be unique characteristics of the ESSO records, and recognition of the possible long period effects of the Benioff Zone source, the simplified shape as shown in Figure 6-7 was adopted. It is of a type that will allow simple modification for local site response (or S factor)



MEAN NORMALIZED RESPONSE SPECTRUM SHAPE

FIGURE 6-7

effects. Further refinement for special site conditions is needed. It is visualized that for special cases, detailed local site investigations will be conducted to arrive at the appropriate MDAF shape.

The shape presented in Figure 6-7 is termed as the best estimate of the true mean normalized spectral shape MDAF, and the values of the plateau or peak PDAF values are given below for the important structure damping values. (See Appendix A).

β	PDAF
5%	2.5
7%	2.3
10%	2.0
12%	1.9

It should be noted in Figure 6-7 that the shape of the MDAF reflects a linear rise in amplification from the ground motion at a zero period value to the PDAF value at a 0.1 second period. This will help establish reasonable response values for very stiff structures.

References 7. and 8. contain statistical studies of spectral shapes and therefore provide additional illustrations of the technique employed in this chapter.

CHAPTER VII

THE EFFECTIVE STRUCTURAL RESPONSE SPECTRUM

SCOPE

In this chapter, the definition for the R factor in the design spectrum equations is presented. For example, in the equation for Design Force Spectrum,

$$DFS = R \cdot A_D \cdot (MDAF) \frac{1}{d_T} (1 + k_T V_S) \qquad \text{4-2 (repeated)}$$

the parameter R appears as a multiplier for the PGA resulting in a modification of the spectrum that will account, in an approximate manner, for the difference between recorded instrument acceleration and the effective acceleration acting on the structure.

. . . .

VII-1 The Relation Between Instrument Records and Structural Response

The Acceleration Zone Graphs and the Iso-Contour Map provide values of Peak Ground Acceleration (PGA) for given return periods. It is most important to recognize that these PGA values represent instrument records rather than peak acceleration values on real buildings. For clarity, the PGA is as shown in Figure 7-1: the peak value of an instrument record of ground acceleration for a given earthquake.

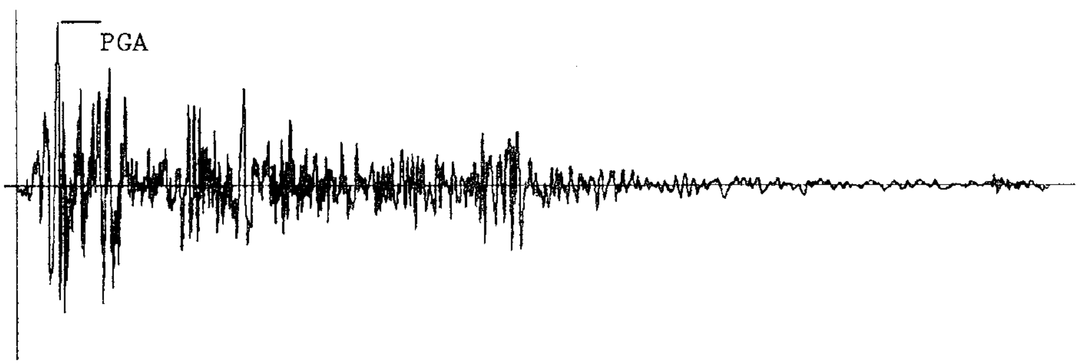


FIGURE 7-1

All actual records used for the data base, all empirical relations for PGA in terms of magnitude, and all attenuation relations are in terms of this instrument record value of PGA because of the precise nature of its definition and its direct availability.

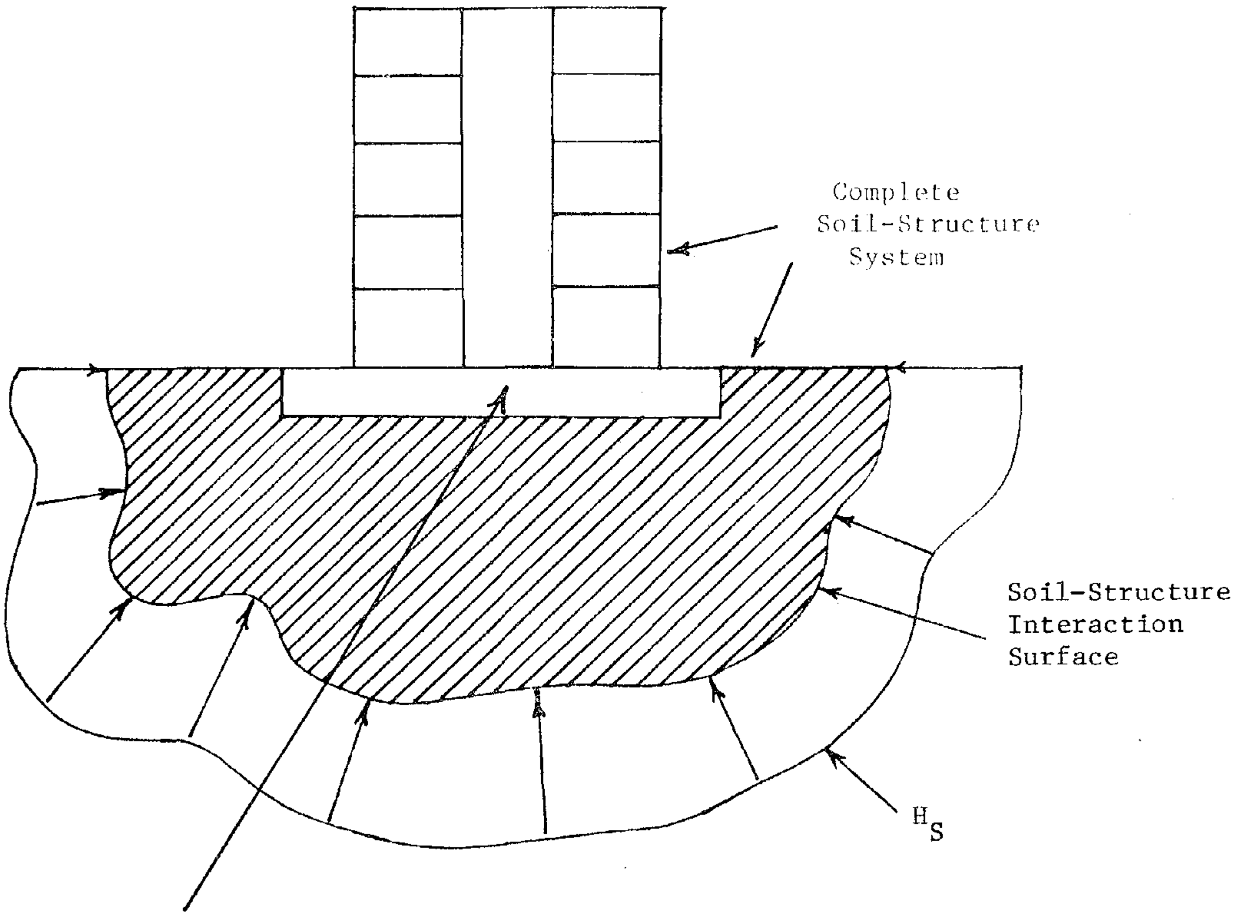
When a value of PGA is taken from the graph or map at a given return period, this value implies that there is a corresponding accelerogram for a given seismic event, representing the response of the instrument system to essentially a point application of time history H_I shown in Figure 7-2. For the purpose of computing the response of an actual building structure, it is necessary to transform the response spectrum representation of the history H_I to the effective structure response spectrum representation of the structure time history H_S. This history H_S is not a point application - but a distributed effect which is applied over the total area of the soil-structure interaction surface. In order to account for this distributed effect, which should include the four factors listed below, it is estimated that the <u>effective</u> structure response spectrum is equal to 0.7 (PGA) · MDAF. The calculated deformation response of the dynamic model due to this spectrum is essentially the same as that of the real structure due to the event that creates H_I on the instrument and H_S on the structure.

The reduction factor of R = 0.7 which converts the peak ground acceleration into the effective ground acceleration represents the combined effects of

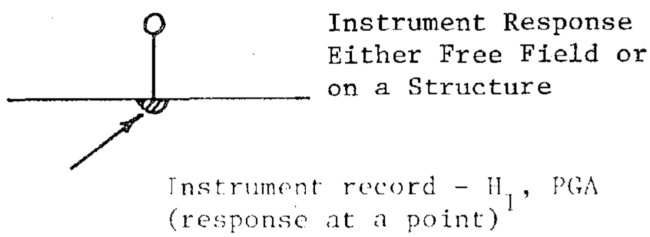
(1) Soil-structure interaction

(2) Foundation flexibility and rocking

(3) The averaging of peaks over the complete inter-action surface.



R · PGA = Surface Average of the Distributed Peaks at all points of the soil-structure system. (H_S; R · PGA)



PICTORIAL REPRESENTATION OF R-FACTOR

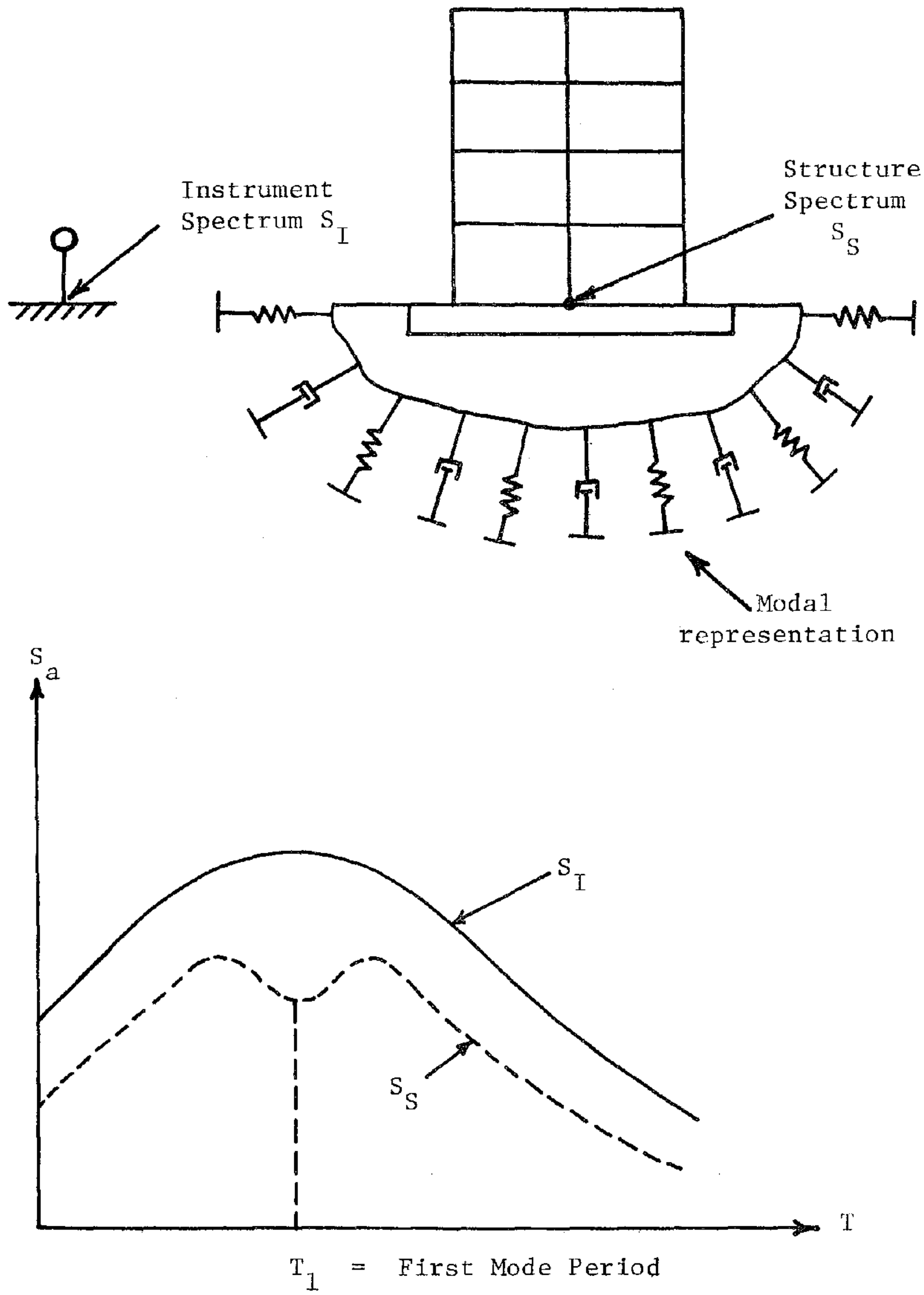
FIGURE 7-2

(4) The filtering of high frequency components that will not effect the response of the structure.

It is assumed that the reduction factor R is due to two general sources: difference between instrument and structure input and difference between the behavior of the SDOF model as employed for spectral evaluation and the individual modal behavior of the structure for motions recorded at the basement or foundation.

In many spectra obtained through accelerograms in buildings, there has been an observed dip in the spectral shape near to the first-mode period. See Figure 7-3. This dip could be explained by feed-back or rocking effects in the total soil structure system. This, of course, needs further research. However, for the present utilization, the R factor presented here is quite sufficient for reliable design.

Selection of the value of R to be 0.7 is somewhat arbitrary. However, the value selected is within rational and reasonable bounds. It could be 0.8 or 0.6; also, it may vary significantly with the type of soil-structure system. Based on the adopted philosophy of simplicity coupled with rationality, an average value of 0.7 is reasonable. Also, since a major component of the R-factor is the insensitivity of the actual structure to the short duration acceleration peaks of the time history, it may be necessary that the R value should vary with the geological region. For example the high peaks of an earthquake source region with shallow focal depths may justify a low (0.7) R value; however a distant or deep focal region should possibly use a higher R value.



EFFECT OF SOIL-STRUCTURE INTERACTION ON SPECTRUM SHAPE

FIGURE 7-3

In Appendix F, the statistics of peaks from the 32 earthquake records which were used to develop MDAF shape, are given. Note that for all the earthquakes, the majority of peaks (more than 99%) lie below the 70% level of PGA. The implication of this phenomenon is not used in developing the value of R.

CHAPTER VIII

TYPES AND BEHAVIOR OF LATERAL FORCE RESISTING SYSTEMS

SCOPE

In this chapter a classification of structural systems, based on their past performance, their deformation properties and on the type of quality control and inspection is suggested. Definition and concept of the damage deformation factor d_T appearing in the Design Force Spectrum

$$DFS = R \cdot A_D \cdot (MDAF) \frac{1}{d_T} (1 + k_T V_S) \qquad \text{4-2 (repeated)}$$

is presented. Numerical value for the (MDAF), - discussed in Chapter VI - based on the effective total damping β_T is also given.

....

VIII-1 Introduction

The Design Force Spectrum (DFS), which will provide the lateral earthquake forces for member design,is very much dependent upon the damping and deformation properties of the particular type of lateral force resisting system employed in a structure. This chapter serves to define the various types of lateral systems and their properties as they govern the formation of all design spectra.

The standard UBC K-factors (0.67 to 1.33) provide the basic format for describing the allowable lateral force resisting systems. Then, depending upon redundancy, reliability, and quality control, a grade of A, B, or C is assigned. This grading method provides a much needed reward or penalty system for good or bad structural systems. Also, it fulfills the need to allow new structural systems. For example, if new construction methods or materials are proposed, these are not arbitrarily prohibited - however, because of their unproven performance they must be subject to high design levels or to a more detailed analysis. In general, the system grade method requires the designer to be fully aware of either the good or bad characteristics of his particular lateral force resisting system.

VIII-2 Seismic Force-Deformation Behavior

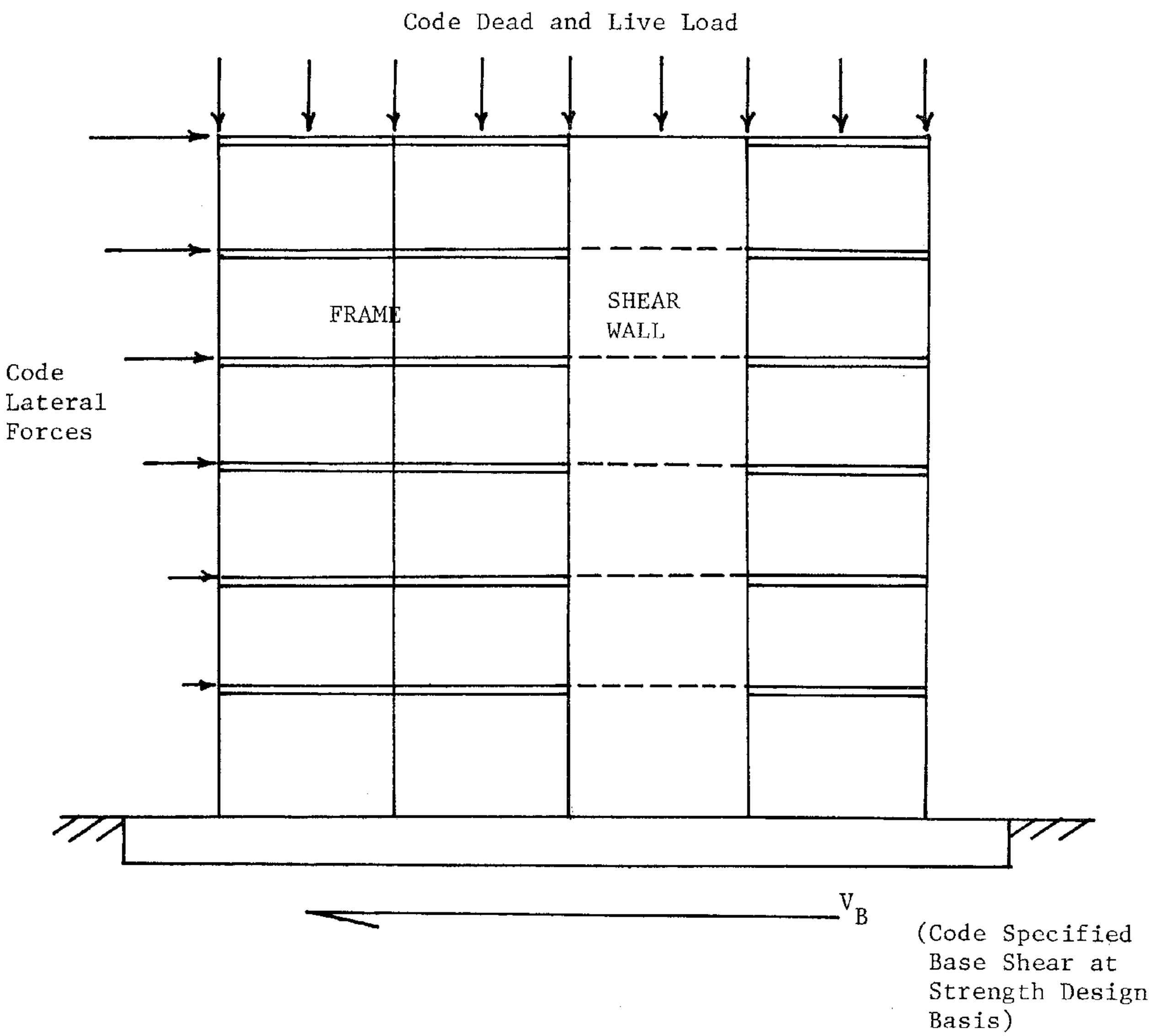
Figure 8-1 shows a typical building structure. Assume that the members have been designed according to the Uniform Building Code for vertical dead and live load, and for a reasonable lateral seismic load.

The purpose of this section is to define and discuss the important seismic load-deformation states of this "designed" structure as it is subjected to increasing levels of earthquake ground motions that may cause structural deformations beyond the code design strength level. The structure carries an ambient live load along with the lateral load.

The following definitions will be useful in the discussion:

(1) Highest Stressed Member, or Member Section with the Highest Stress-Ratio: where the effects of vertical load and seismic lateral load combine to produce the maximum load demand on the section.

(2) Member Section Strength, R_u: the ultimate strength capacity of a reinforced concrete section, and an appropriate



TYPICAL CODE DESIGNED BUILDING

FIGURE 8-1

Θ_{RD} is the damage threshold deformation for the member - beyond this value a significant amount of damage occurs in the member.

Θ_{RC} is the condemnation threshold deformation for the member - beyond this value the member is beyond repair and its ability to carry load is questionable.

Δ_{RD} is the structure damage threshold deformation where a significant number of members are at or beyond Θ_{RD}. At this threshold,

(1) wide cracks and spalling occurs in concrete beams and joints, and in shear wall chords.

(2) extensive diagonal cracking exists in shear walls.

(3) visible distortion and/or plastic rotations are present in steel members.

(4) story drifts are such as to cause damage and loss of function in non-structural elements - unless design precautions are taken for their protection.

This Δ_{RD} may be reasonably larger than the deformation at which the first or highest stressed member reaches Θ_{RD} because in the actual three dimensional, statically indeterminate structure, many members must attain Θ_{RD} in order to create a total structure damage state.

Δ_{RC} is the structure condemnation threshold deformation where a significant number of members are at the condemnation state Θ_{RC}. At this level,

(1) Local member ductility demands u_C as measured by the ratio of Θ_{RD} to Θ_{Des} are at or beyond established allowable values.

(2) Extensive diagonal cracking and/or chord damage has deteriorated shear walls beyond repair.

(3) Important columns, frames, or piers are near to buckling failure.

(4) Member distortions and or drifts are non-correctable.

Similar to the damage state, Δ_{RC} may be larger than the first Θ_{RC} deformation state, since many members must be involved in order to constitute the condemnation state.

VIII-3 Types of Allowable Lateral Force Resisting Systems

For the purposes of the proposed design method, the same general classification of lateral force resisting systems is used as is given in the 1973 UBC and the 1974 SEAOC Recommendations (References 5 & 6). These are termed as "allowable" systems since they all have the quality of collapse resistance - that is, the vertical load carrying system is shielded by beam yield hinges, bracing or shear walls so as to reliably withstand the effects of an earthquake without loss of stability. The general definitions of the system types are as follows; some minor changes have been made (from SEAOC) in order to better assure the collapse resistance.

Definition of Structure Types According to K-Factors

K = 0.67 Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: The ductile moment resisting space frame shall have the capacity to resist the total required lateral force.

K = 0.80 Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls designed in accordance with the following criteria:

1. The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames.
2. The shear walls acting independently of the ductile moment resisting space frame shall resist the total required lateral force.
3. The ductile moment resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force.

K = 1.00 Buildings with a complete vertical load carrying frame together with either shear walls or bracing that resists the total lateral force.

1. Same as criterion 1. for K = 0.80.
2. The frames need not qualify as "ductile moment resisting". However, it is recommended that details for ductility be employed in elements having the largest stress ratios. These details include continuous longitudinal steel, stirrups over beam lengths, tied splices, and compact steel sections.

K = 1.33 Buildings with shear walls or braced frames capable of resisting the total required lateral force. These buildings are distinguished by the fact that a significant portion of the vertical load is carried by the lateral force system.

While these definitions provide a common and familiar starting point, there is a definite need for better description of the various forms of system configuration and the various degrees of quality or reliability of performance. Therefore, a practical method of recognizing these variations is to be developed in terms of a grading system.

VIII-4 A Proposed Grading System for Structural Types

Each of the standard types of structural systems is to be assigned a grade of A, B, C depending on its particular qualities of stability, redundancy, dependability, and reliability of performance at the damage and condemnation thresholds. These respective qualities will be rated as Excellent, Good, or Fair for any given system as follows.

Reliability and Dependability

Structures in the Code K factor categories (0.67, 0.80, 1,00, 1.33) can have ratings of excellent, good, or fair in terms of their as-constructed reliability of satisfactory performance during strong ground motion. These ratings depend on the accuracy of analysis, degree of construction supervision, labor skill, type of details, and method of construction. Items to be considered are:

(1) Available established methods of design of members and connections.

(2) Performance experience during past large ground motion earthquakes, or generally accepted estimates of good performance if experience is not available.

(3) Estimated agreement of actual behavior with analysis procedures.

(4) Presence of back-up systems or redundancies.

(5) Ease of good construction without rigorous inspection.

(6) Degree of inspection.

Ratings may be accomplished according to the following suggested rules:

Excellent = Structural configuration can be modeled and analyzed according to standard accepted procedures. Materials and construction inspected under supervision of Engineer. Standard construction procedures with well-trained workmen.

Good = Average conditions with occasional inspection by Engineer.

Fair = Unknown conditions with no direct inspection by Engineer. Possible untrained workmen. Doubtful quality of materials.

Redundancy and Stability

Structures in the Code K factor categories (0.67, 0.80, 1.00, 1.33) can have ratings of Excellent, Good, or Fair in terms of the inherent redundancy and stability (both vertical and torsional) of their configurations in plan and elevation. It is suggested that criteria such as the following be employed:

Frames of K = 0.67, 0.80, 1.00 Systems

Excellent = 4 or more rows of frames, together with 3 or more bays per frame. Bay widths should not differ by more than a ratio of 1.5. Torsional plan eccentricity no larger than 10 percent of the structure width normal to loading.

Good = Same as Excellent except that there can be less than 3 bays per frame, and plan eccentricity no larger than 20 percent.

Fair = All other system configurations with the exception that systems with large plan eccentricity, grossly nonsymmetrical plan shape, and/or large changes in stiffness will require a more detailed analysis.

Walls or Vertical Bracing in K = 0.80, 1.00, 1.33 Systems

Excellent = 4 or more rows of walls or bracing in 2 or more bays of a frame. In K = 1.33 systems the wall panels in any story should provide either 1 pier with height to width H/D less than 1/4 or 2 or more piers with H/D less than 1/2. Torsional plan eccentricity no larger than 10 percent.

Good = 4 or more rows of walls or bracing in 1 or more bays. In K = 1.33 system, 1 pier with H/D less than 1 or 2 piers of H/D less than 2. Eccentricity no larger than 20 percent.

Fair = All other configurations except for gross irregularities or eccentricities which require a more detailed analysis.

Having these rating descriptions, any given system and its configuration can be assigned a grade by the following rules.

A requires Excellent in both Stability and Reliability

B requires at least Good in both Stability and Reliability

C requires at least Fair in both Stability and Reliability.

Table 8-1 shows the general characteristics of each grade, and Table 8-2 shows a suggested form of summarizing the grading method for the purposes of a future building code format.

TABLE 8-1

GENERAL GRADING CHARACTERISTICS

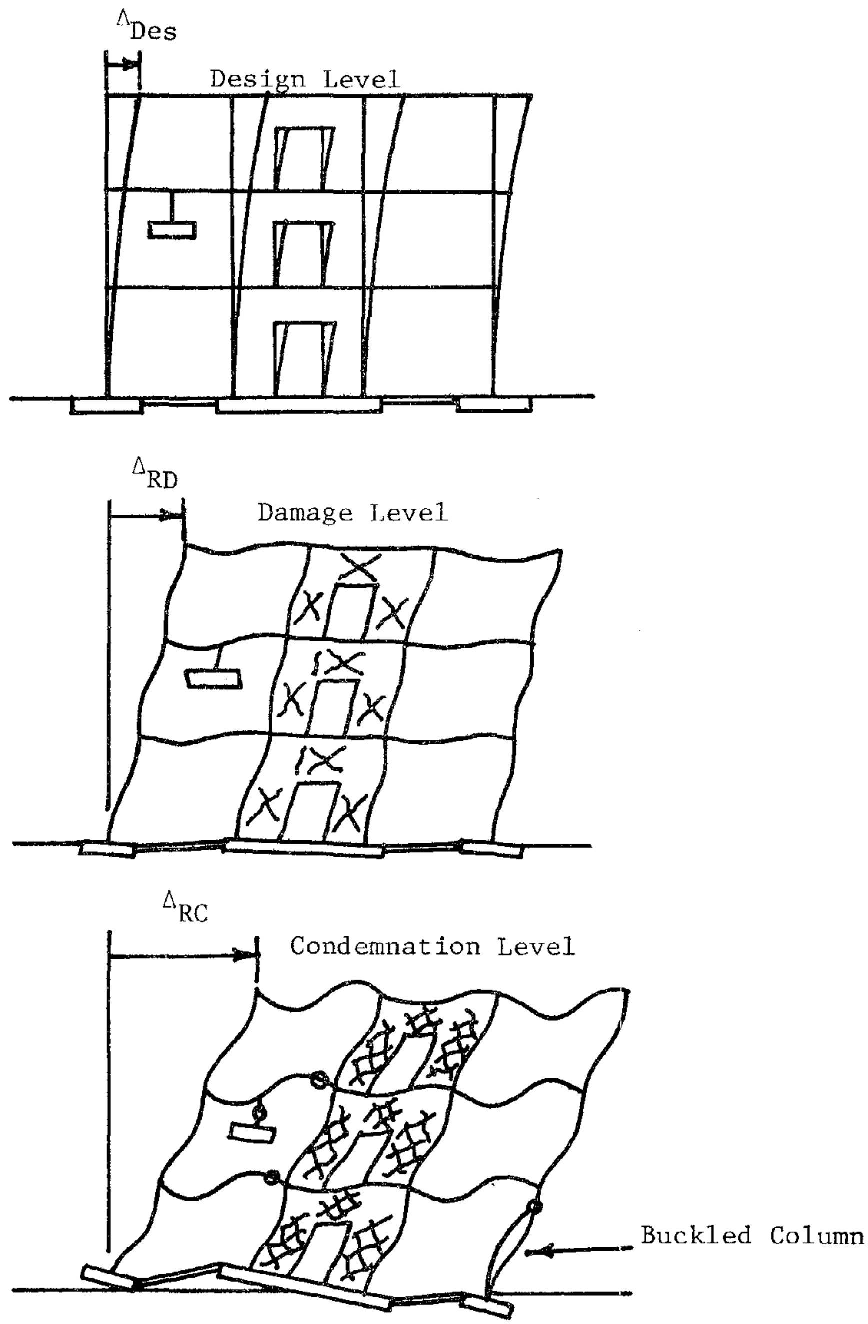
GRADE	Stability and Redundancy	Proven Reliability of System Performance	Inspection and Quality Control
A	Symmetrical Many frames and or walls with many bays	Standard conventional systems with good seismic details	Inspection by engineer Good construction personnel
B	Intermediate configurations	Intermediate system	Intermediate conditions
C	Non-symmetrical Two frames or walls or bracing with one or two bays	New types of construction with no earth-quake experience record	Remote or no inspection Doubtful materials and workmanship

TABLE 8-2

STRUCTURE TYPE AND GRADE REQUIREMENTS

Type	Grade A	Grade B	Grade C
All Types	. Plan Eccentricity $\leq$ 10% . No precast or prestressed elements in the diaphragms or in lateral force resisting system . "Exact" Linear Stress Analysis . Inspection under supervision of engineer . Dynamic Analysis for buildings with 4 or more stories	. Plan Eccentricity $\leq$ 20% . No precast or prestressed elements in the diaphragms or in lateral force resisting system	All others
K = 0.67	. 4 or more Frames . 3 or more Bays . Steel Frame	. 4 or more Frames	All others
K = 0.80	. 4 or more Frames . 3 or more Bays . 4 or more shear walls . Shear wall chords must qualify as ductile columns	. 4 or more Frames	All others
K = 1.00	. 4 or more Shear Walls . 3 or more Bays . No Braced Frames . Shear wall chords must qualify as ductile columns . Height limit of 16 Stories	. 4 or more Frames . No Braced Frames . Shear wall chords must qualify as ductile columns . Height limit of 16 Stories	All others
K = 1.33	. 4 or more walls . No braced frames . Shear wall chords must qualify as ductile columns . Height limit of 16 Stories	. 4 or more walls . No braced frames . Shear wall chords must qualify as ductile columns . Height limit of 16 Stories	All others

While the descriptions of the qualities required for a certain grade are rather brief and certainly not comprehensive - the exercise of a grading procedure has the purpose of making designers aware of the general characteristics of good or bad systems and hence influence their design decisions.



DEFORMATION STATES OF A GIVEN STRUCTURE

FIGURE 8-4

VIII-5 Parameters in Design Spectra

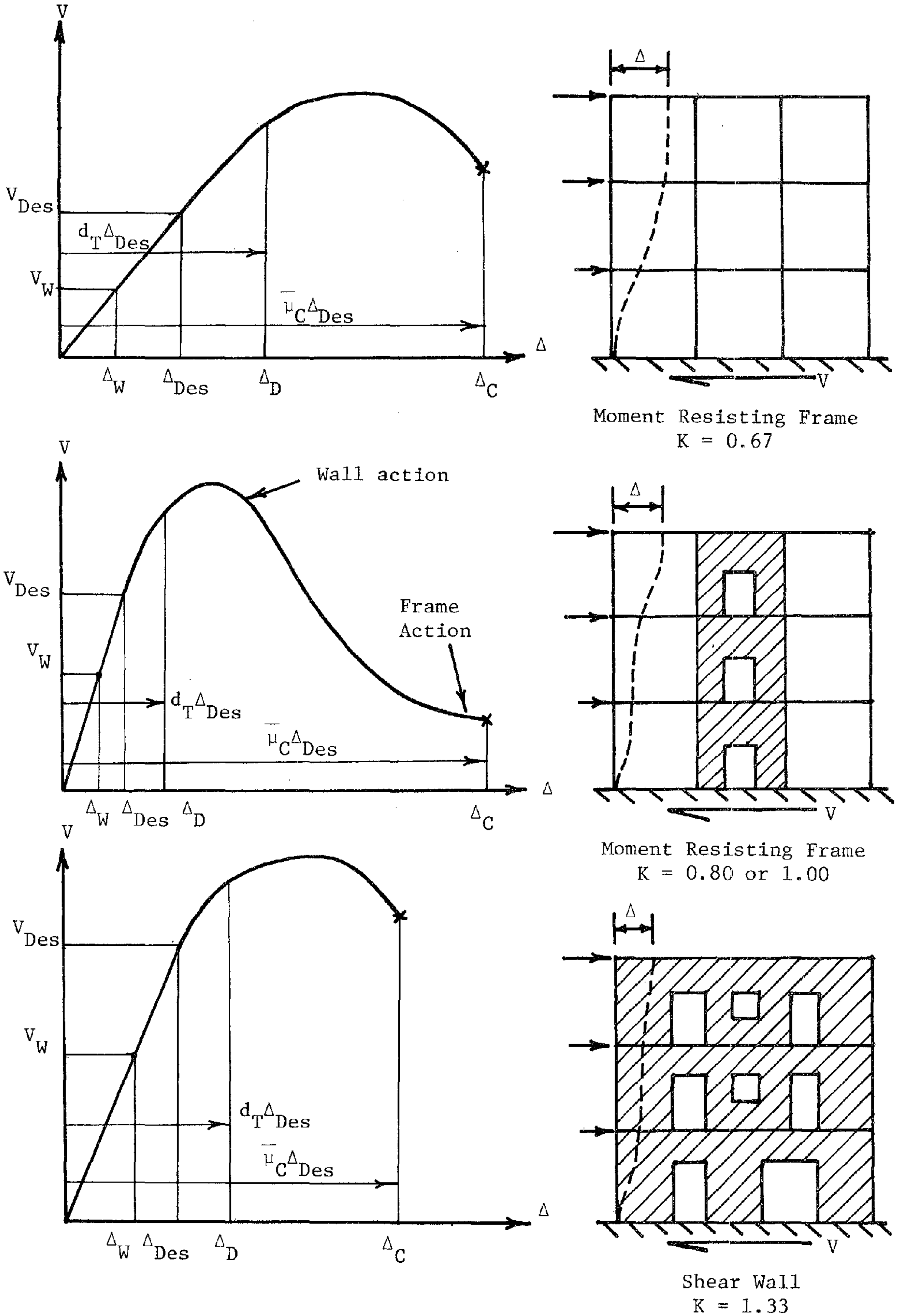
The load-deformation behavior for various types of lateral force resisting systems is shown in Figure 8-5. Each graded type of system, such as a K = 1.00 B, will have its particular values of total structural damping β_T, damage deformation factor d_T, and response spectrum confidence level factor k_T. Also, since design values for overturning moment are highly sensitive to the ductility and damage resistance of walls, columns and foundation structures, a special design overturning moment factor (d_{OT}) needs to be formulated. These parameters will be used to form the Design Force Spectrum for a given structure type. The factor d_T, d_{OT} and damping β_T are discussed in the next sections of this chapter, and k_T is developed in Chapter IX.

VIII-6 Damage Deformation Factor (d_T)

The structural deformation characteristic d_T, termed as the damage deformation factor is a most important quantity in the formation of a Design Spectrum. This factor d_T is a numerical representation of the fact that a real building structure is not at the significant damage threshold level when the highest stressed member reaches its design strength capacity. The deformation at the damage state is substantially beyond the design state. The value of d_T depends on the type of structural system and it increases with the degree of redundancy. It is not only a measure of material ductility, but also represents the ability of the slightly non-linear structure to "fall out" of resonance and thereby not reach the spectral peaks of the perfectly linear system.

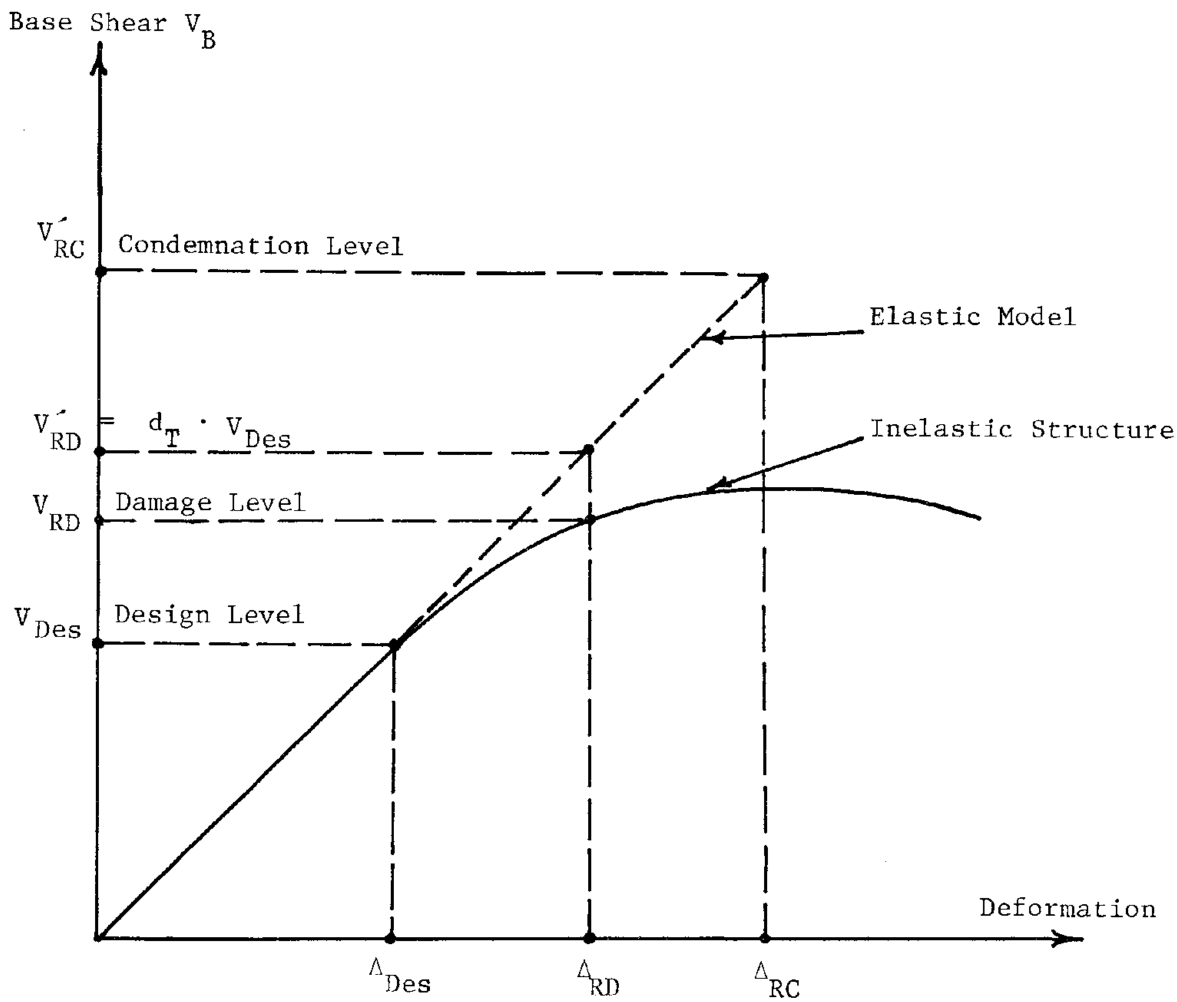
Referring to Figure 8-6,

$$d_T = \frac{\Delta_{RD}}{\Delta_{Des}} = \frac{V'_{RD}}{V_{Des}}$$



LOAD-DEFORMATION BEHAVIOR ACCORDING TO TYPES OF STRUCTURAL SYSTEMS

FIGURE 8-5



LOAD-DEFORMATION CURVE

FIGURE 8-6

where, again,

Δ_{RD} = Structural Deformation at the damage threshold - beyond which a significant amount of structural damage will occur.

Δ_{Des} = Structural Deformation at the Member Design Level - at which design strength capacity is reached in members having the highest stress-ratio due to seismic design load F_{Des} and ambient vertical loads (Figure 8-7)

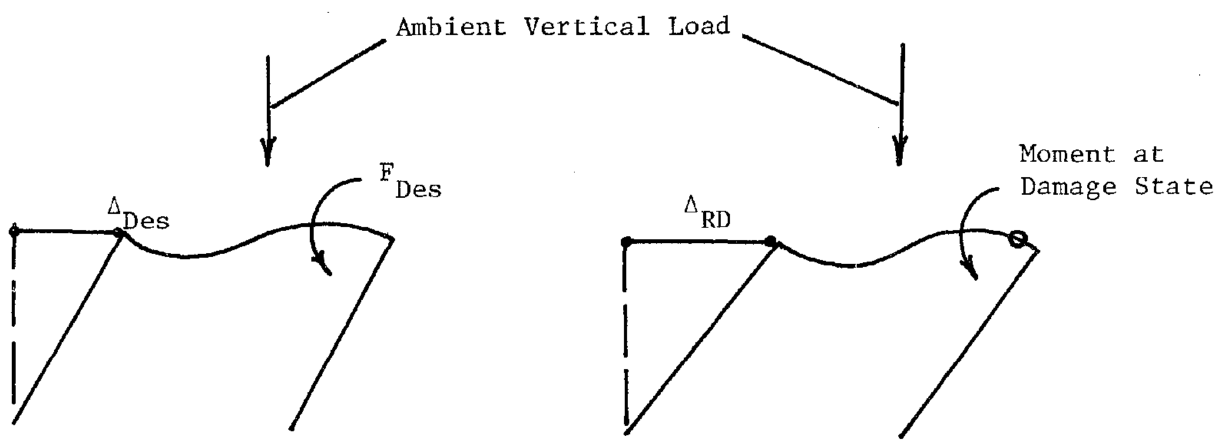


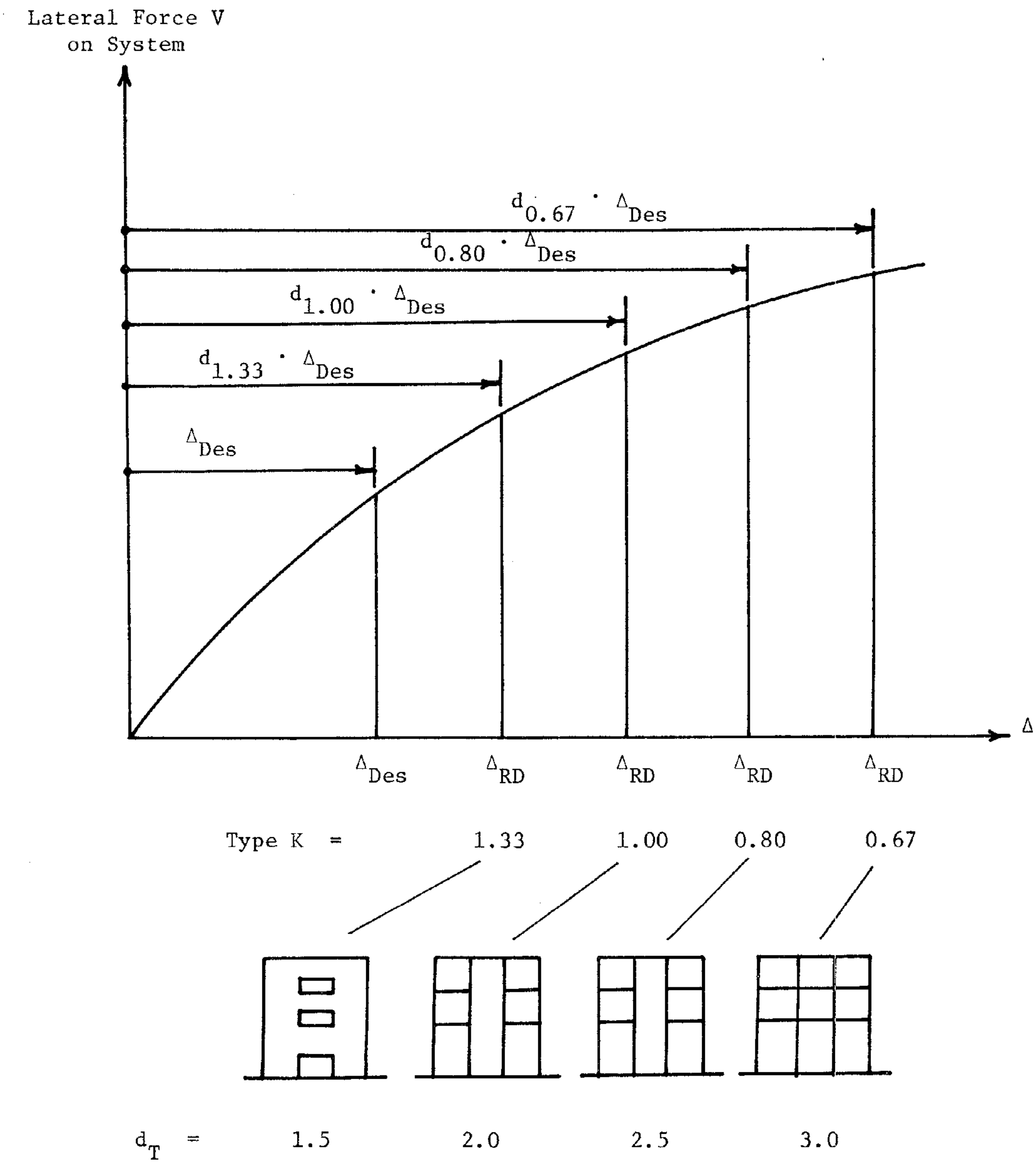
FIGURE 8-7

The proportionality of base shears in this relation may be used because of the relatively small amount of inelastic behavior in the total structure at the damage threshold; it is assumed that Δ_{RD} can be predicted by the response of the linear elastic structure model to the Damage Threshold Spectrum DDS with damping corresponding to this threshold. As a consequence of this assumption, the force V_D' in the linear model at the damage threshold is proportional to $\Delta_{RD} = d_T \Delta_{Des}$, and therefore is equal to $d_T V_{Des}$. If the DDS is known and the value of d_T is assigned subjectively according to the type of structural system, then the Design Force Spectrum DFS which provides V_{Des} is given as the DDS divided by d_T. The basic concept is that when the members with the highest stress ratio are designed at the ultimate strength basis for forces due to the DFS, then the structure damage threshold will be at deformations equal to or greater than those caused by the DDS.

The value of the d_T factor is assigned subjectively based on a judgemental evaluation of the damage resistance of a given system type. Some example values are given in Figure 8-8. Later, in the chapter on Design Spectra, a discussion will be given concerning the method of subjective assignment of all parameter values (d_T, d_{OT}, β_T, etc.)

VIII-7 Design Overturning Moment Factor (d_{OT})

Given a shear wall with its shear reinforcement designed for ultimate strength resistance to V_{Des}, and with chord steel designed for the corresponding overturning moment effects of V_{Des}, and with the qualification of having confinement ties as required for a ductile column (Figure 8-9), the shear damage threshold Δ_{DV} in the wall panel is reached before the overturning moment flexural damage threshold Δ_{DM} occurs in the chords (Figure 8-10). This is because the confined and contained (by closed ties) concrete in the chords does not suffer a significant strength reduction



d_T RELATIONS FOR THE VARIOUS TYPES
OF LATERAL FORCE RESISTING SYSTEMS

FIGURE 8-8

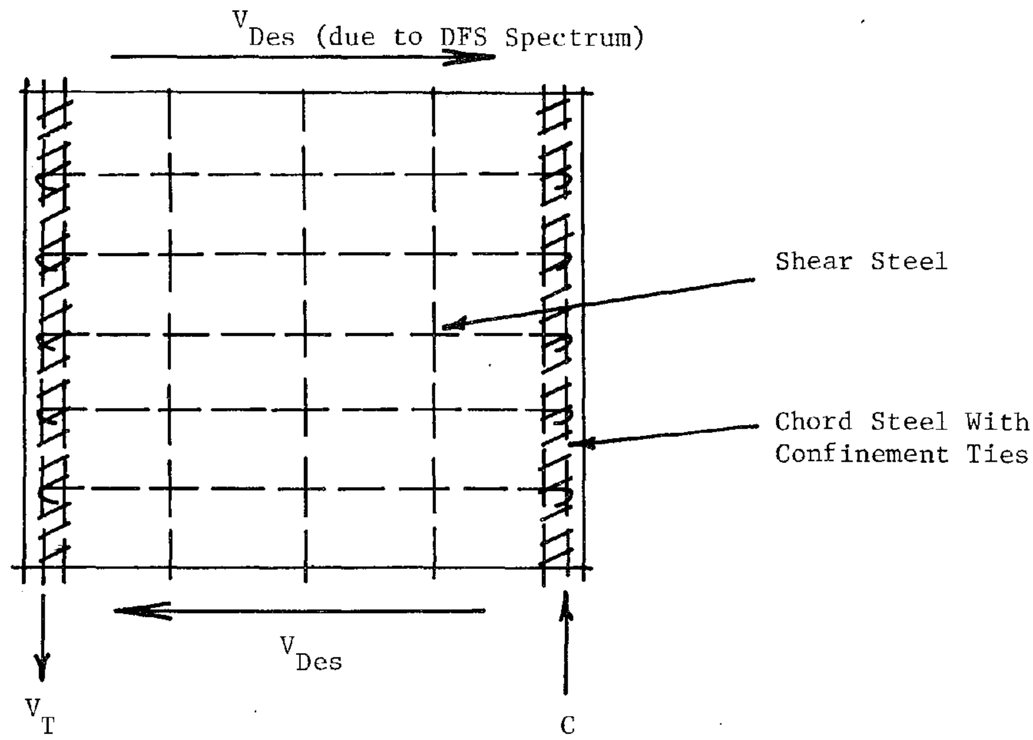


FIGURE 8-9

or damage at the cyclic load levels that do produce the significant damage state of orthogonal diagonal shear cracking and strength deterioration in the panel. Although the required grid of horizontal and vertical shear reinforcing steel can distribute and control this shear cracking so as to maintain the integrity of the wall - the physical appearance of the grid of orthogonal cracks constitute the damage threshold.

Therefore, in order to provide a wall design in which both the shear and flexural damage thresholds would occur at the same deformation it is necessary to set the design level for flexure at a level lower than that for shear.

In terms of design method, this requires two design spectra. For strength design - except overturning moment effects - the design spectrum is given by

$$DFS = \frac{DDS}{d_T} \qquad 8\text{-}1$$

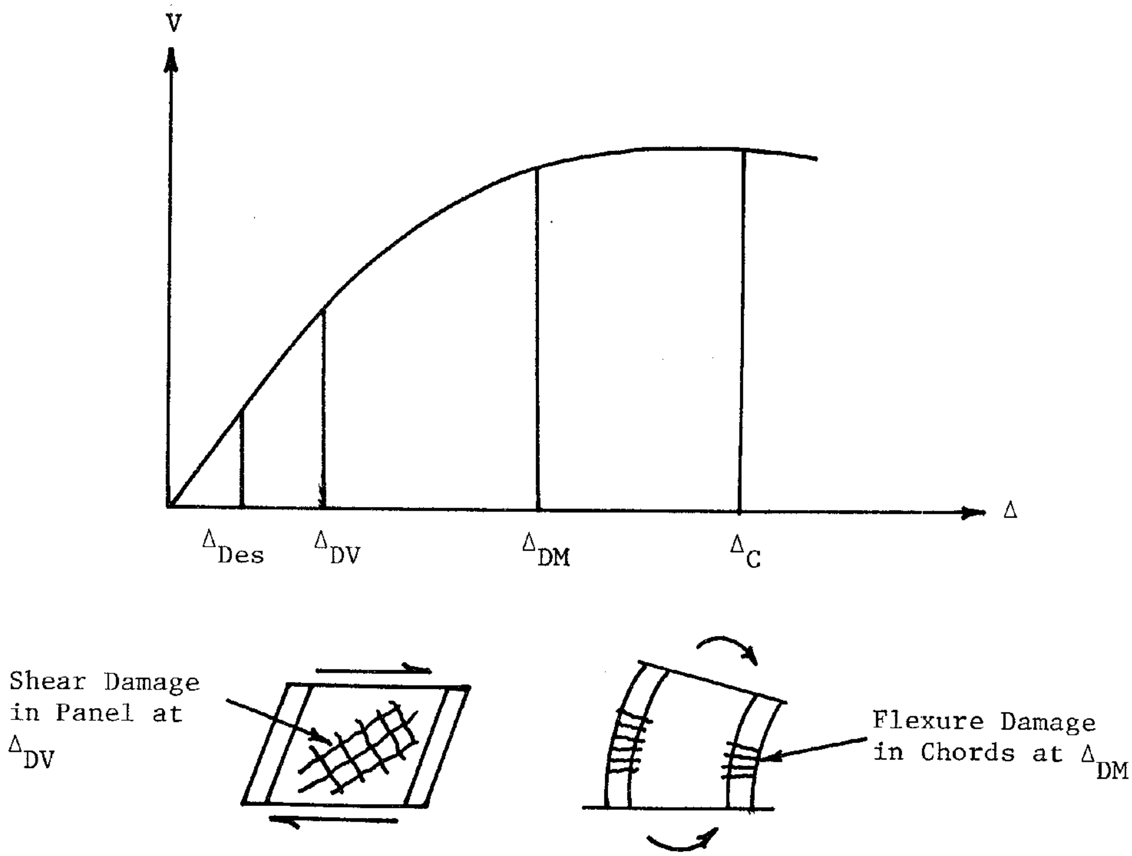


FIGURE 8-10

For overturning moment effects a new spectrum based on

$$DMS = \frac{DDS}{d_{OT}} \qquad 8\text{-}2$$

should be used. This is an attempt to make the damage threshold for flexure due to overturning coincide with the damage threshold due to shear effects. Thus, d_{OT} is larger for walls which do have ductile damage resistance in their chords. For all other walls d_{OT} is equal to d_T.

Similarly, if footing uplift occurs prior to the development of flexural damage in the chords, then an appropriate degree of this flexural uplift rotation corresponds to the damage and condemnation threshold as shown in Figure 8-11

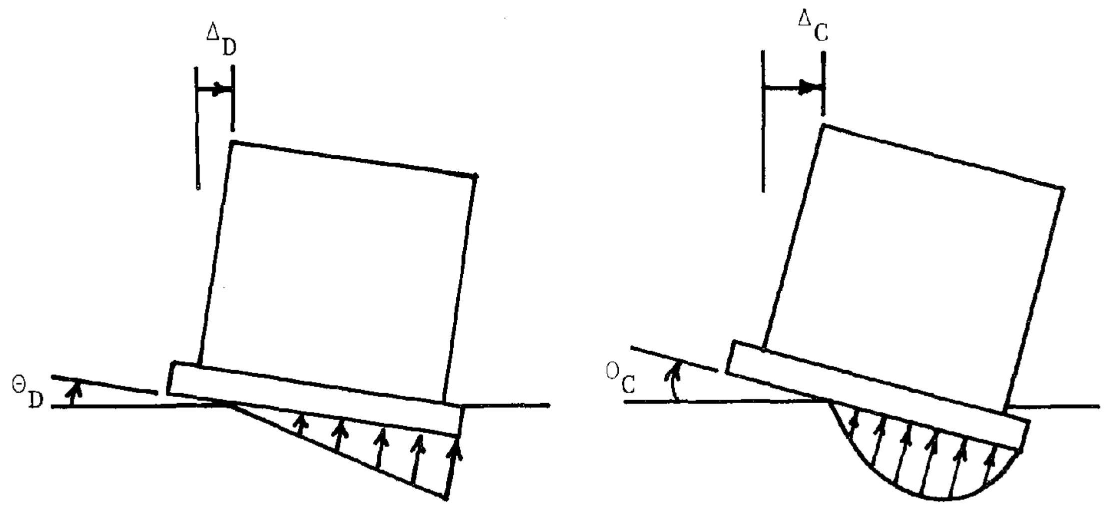


FIGURE 8-11

If flexural damage is catagorized as either chord damage or uplift, as shown in Figure 8-11, then a balanced design (representing a simultaneous occurrence of flexural and shear damage) results from the use of d_{OT} (for flexure) greater than for d_T (for shear) as shown in Figure 8-12.

VIII-8 The Damping Factor β_T and its Corresponding PDAF

The total damping β_T assigned to a given type of structural system will determine the appropriate peak dynamic amplification factor (PDAF).

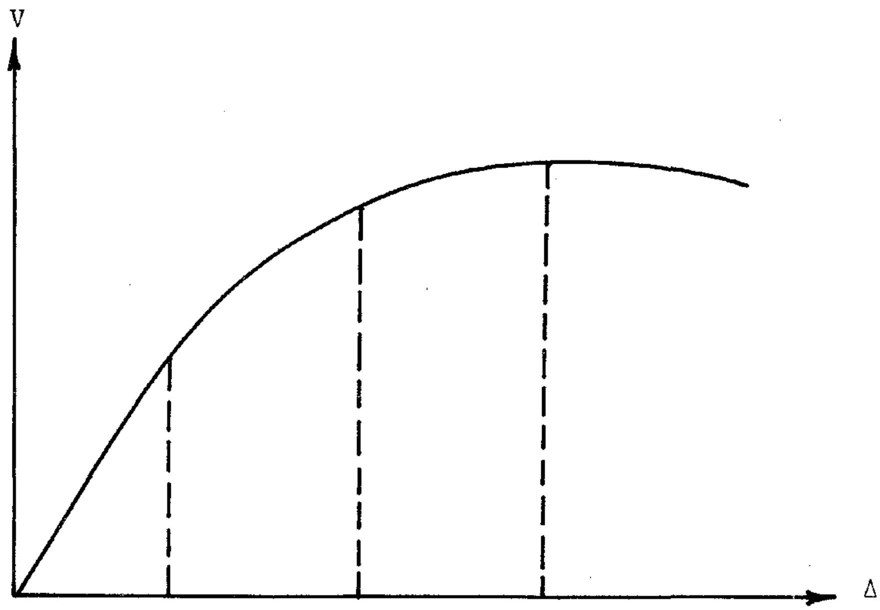


FIGURE 8-12

For example $\beta_T = 5\%$ gives PDAF = 2.5, and $\beta_T = 10\%$ gives PDAF = 2.0 (see Appendix A). The term "total damping" is employed in order to represent the fact that the real structure can contribute three sources of damping:

(1) Ordinary structural or internal friction damping $= \beta_S$

(2) Hysteritic damping due to mildly non-linear hysterises, primarily in ductile frame structures $= \beta_H$

(3) Foundation - soil interface distortion damping, primarily in wall footings $= \beta_F$

Total damping $\beta_T = \beta_S + \beta_H + \beta_T$ may vary somewhat from system to system. However, for simplicity, a value of 10% is suggested for all structural systems. This could be justified from the estimates of damping presented in the following table.

TABLE 8-3

Type	β_S	β_H	β_F	β_T
0.67	5%	4%	1%	10%
0.80	6%	2%	2%	10%
1.00	6%	2%	2%	10%
1.33	7%	1%	3%	10%

Therefore, all structure types have identical PDAF value of 2.0. It should be noted that the values of β_S, β_H and β_F given above in Table 8-3, are strictly subjective. A detailed evaluation and discussion is needed before any numerical value can be adopted for practical use.

In general, as a guide for assignment of damping, the following properties and conditions should be considered.

Damping Measures for β_S

(1) Material or Member Damping.

(2) Connection Damping.

(3) Floor System.

(4) Exterior Cladding and Interior Partitions.

(5) Step Changes in System Stiffness and Period Due to Cracking During the Time History.

(6) Multiple Frames or Walls.

(7) Ambient Live Load Effects.

Ductility Measures for β_H

(1) Material or Member Ductility.

(2) Connection Ductility.

(3) Presence of a Back-up System for Support of Vertical Loads.

(4) Multiple Frames.

(5) Multiple Bays or Number of Redundancies.

Structure-Foundation Interaction Damping Measures for β_F

(1) Type of Foundation.

(2) Size of Foundation.

(3) Foundation Stiffness.

(4) Structure Stiffness.

(5) Foundation Uplift Effects.

(6) Type of Soil.

CHAPTER IX

RELIABILITY OF DESIGN OBJECTIVES

SCOPE

This chapter develops the theory related to the purpose and evaluation of the spectral confidence level factor k_T as it appears in design spectrum equations, for example

$$DFS = R \cdot A_D \cdot (MDAF) \frac{1}{d_T} (1 + k_T V_S) \qquad 4.2 \text{ (repeated)}$$

The k_T value depends on the type of lateral force resisting system and its quality grade of A, B, or C. This confidence level factor is meant to provide a high enough design load such that the resulting structure can reliably resist the damage threshold earthquake without significant damage, and the condemnation threshold earthquake without condemnation.

....

IX-1 Reliability of Design Objectives for a Given Seismic Event

When a structure use class is defined, then the acceptable life time risks (P_D, P_C) or return periods (RP_D, RP_C) for damage and condemnation are known. (See Chapter V). The Iso-Contour Map or Acceleration Zone Graph provides the PGA values (A_D, A_C) for the seismic events having these acceptable risks of exceedance and the structural design can proceed with this basic seismic load level information as input for the response spectrum method. However with the recognition that actual structure deformation response Δ_D is random for a given seismic event, and structural resistance Δ_R is random for a given design level Δ_{Des}, how can it be assured that the actual risks of damage and condemnation will be essentially equal to the acceptable values of P_D and P_C? The answer exists in the appropriate choice of an upper confidence limit for the calculated, or selected design value for demand. For simplicity, this concept will be explained in terms of the damage demand Δ_D. Design parameters are to be assigned such that the risk or chance that the actual demand Δ_D will exceed the structure damage threshold Δ_{RD} will be made small enough, such that the risk of damage threshold exceedance will be essentially equal to the P_D value associated with PGA of A_D. Similar philosophy is applied for condemnation level reliabilities.

IX-2 <u>The Random Description of Seismic Demand</u>

In order to discuss a random phenomenon such as the response demand on a structure due to a given earthquake event, it is necessary to begin with the concept of the best estimate or mean value. It will be assumed that the mean response deformation demand $\bar{\Delta}_D$ for the given seismic event as represented by A_D is given by the SRSS response of the linear structure model to the <u>mean damage threshold spectrum MDS = $R \cdot A_D \cdot$ MDAF.</u>

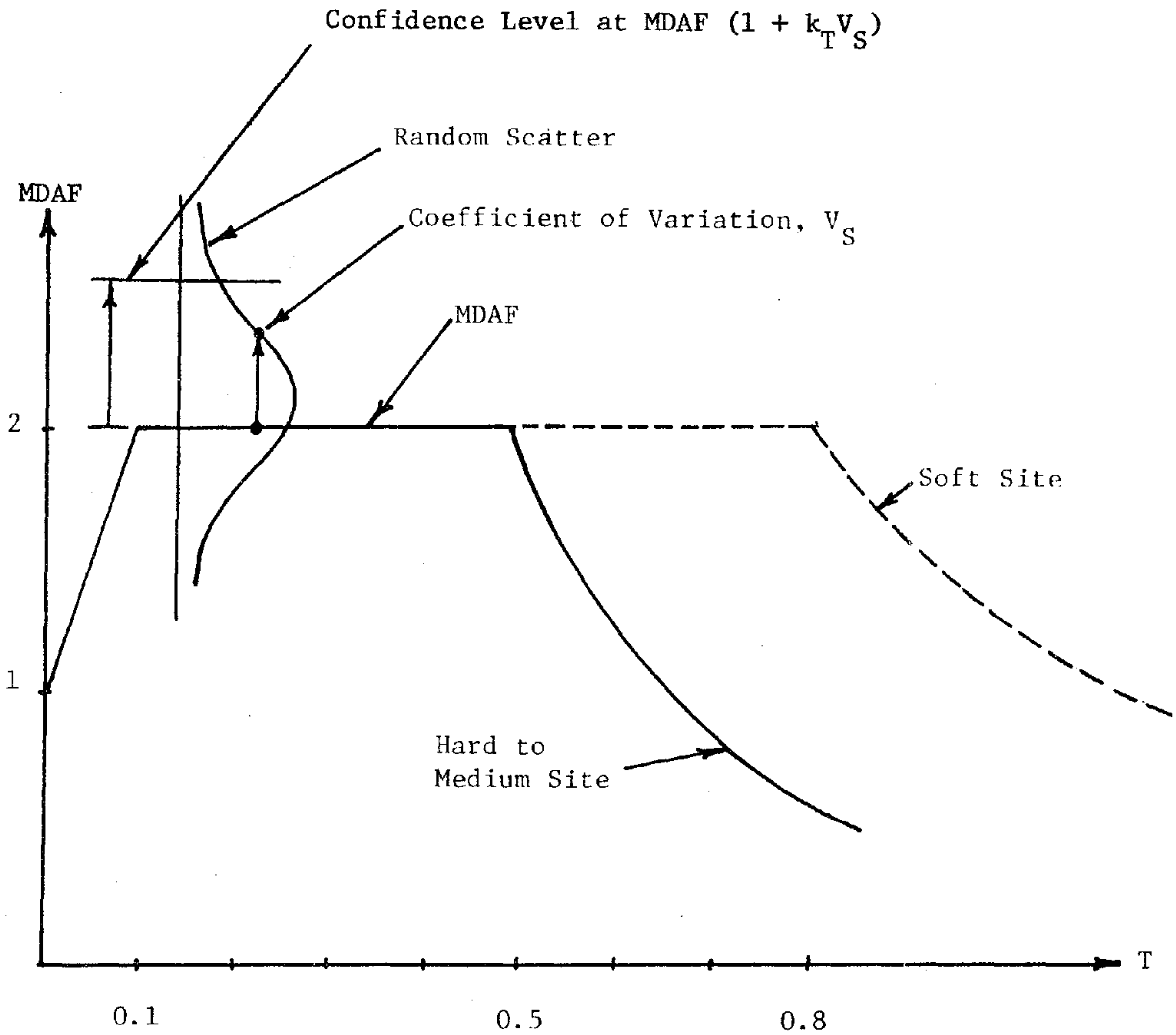
Because of the imperfect knowledge of the seismic input, the structure model, and its actual response, the true response due to the given event is a random variable $\tilde{\Delta}D$. This value is assumed to be scattered about the mean value $\bar{\Delta}_D$ with a standard deviation value of σ_D. The components of uncertainty which contribute to this σ_D value include:

- Statistical prediction error in the Iso-Contour Map value for A_D and in the choice of the R-Factor.
- True response spectrum shape as it is scattered about the MDAF with coefficient of variation V_S. (See Figure 9-1).
- Modeling approximations and uncertainties in the description of the real structure; including stiffness, choice of damping, and foundation restraints.
- Approximation error in the modal superposition of response by the SRSS method.

Except for the spectral variation value of V_S, no specific values can be assigned to these components of uncertainty, and therefore it should be realized that subjective judgment and rough calibration with existing code values are to be employed for establishing safe design values rather than a statistical or mathematical approach based on an acceptable probability of failure. The random description is shown in Figure 9-2.

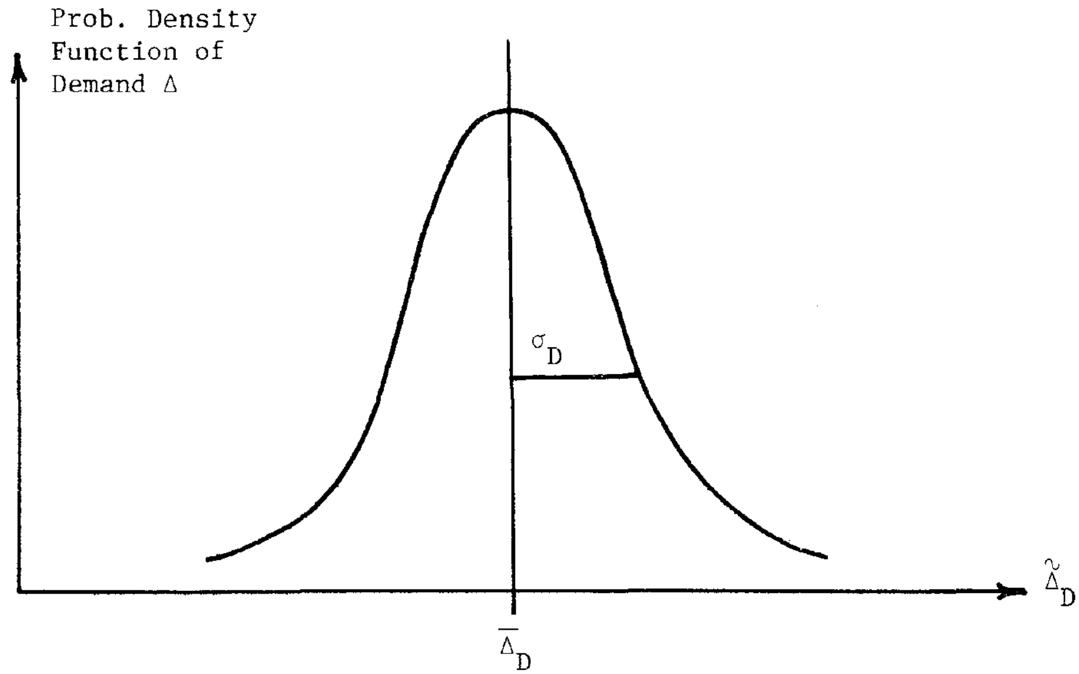
IX-3 The Random Description of Structure Resistance or Capacity

For a given structure with its particular lateral force resisting system that has been designed at a certain lateral strength level and deformation state (V_{Des}, $\tilde{\Delta}_{Des}$), the actual damage threshold deformation is a random variable Δ_{RD}. For the allowable and adequate types of systems, materials, and details this random quantity is substantially above the design state Δ_{Des}. For descriptive purposes the mean threshold value $\overline{\Delta}_{RD}$ is assumed to be a multiple of $\overline{d}_T$ times Δ_{Des}. The mean amount of deformation or excursion $\overline{d}_T$ is dependent on the overall deformation capabilities



STATISTICAL PROPERTIES OF THE DAF SPECTRAL SHAPE

FIGURE 9-1

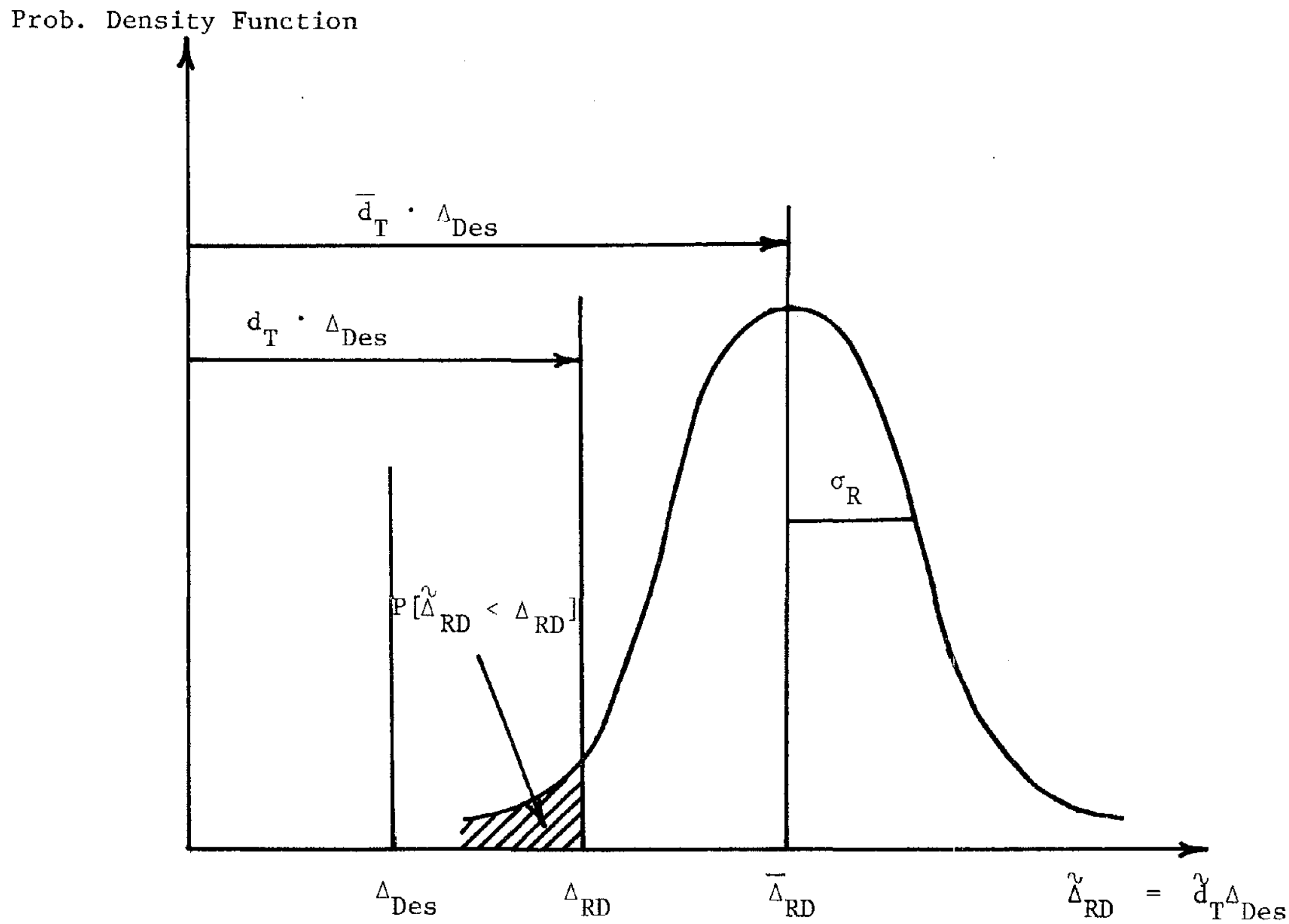


DAMAGE THRESHOLD DEMAND

FIGURE 9-2

and redundancy of the structural system. The actual damage threshold $\tilde{\Delta}_{RD}$ can be represented as a random $\tilde{d}_T$ times Δ_{Des}, and it is scattered about $\bar{\Delta}_{RD}$ with standard deviation σ_R as shown in Figure 9-3. The components of σ_R are:

- Uncertain empirical knowledge concerning the member design strength value and its relation to the damage state.
- Uncertain member strengths and system behavior due to construction variabilities in the as-built structure.



DAMAGE THRESHOLD DEFORMATION

FIGURE 9-3

- Uncertain system behavior due to non-calculated redundancies in the real three-dimensional structure and its foundation structure.
- Uncertain definition of the physical and economic conditions which correspond to the damage threshold as assessed by the owner or inspecting agency.

As in the case of demand, not much in the way of numerical value can be assigned to these sources of random variation. Perhaps the most practical approach is to assign, by judgement, a conservative value for the damage deformation factor d_T such that the value of $d_T \cdot \Delta_{Des}$ is a safe or reliable lower-bound on the damage threshold. The complete random description is shown in Figure 9-3.

IX-4 Relation of the Random Demand and Resistance for Reliable Performance

All of the listed sources of uncertainty and variation in both demand and resistance contribute to their respective σ_D and σ_R values. The amount of each contribution depends upon both the type (K = 0.67 to 1.33) and the quality grade (A, B, C) of the lateral force resisting system. If we review the grading criteria given in Table 8-1 of Chapter VIII, we see that as grades go from A to B to C then:

- Quality or accuracy of analysis decreases.
- Predictability of Response decreases (due to torsion effects of non-symmetry).
- Predictability of response and damage behavior decreases because of lack of experience with new systems.

- Redundancy decreases and hence sensitivity to damage increases.
- Construction quality decreases.

Therefore, while we have no real quantitative knowledge of the σ_D and σ_R values, we do have at least a system of qualitative measures in terms of grading

$$\sigma_{DA} < \sigma_{DB} < \sigma_{DC}$$

$$\sigma_{RA} < \sigma_{RB} < \sigma_{RC}$$

for grades A, B, and C respectively, as shown in Figure 9-4. As a result, it will be seen that the confidence level factor k_T is effected by the type of structural system (K = .67 to 1.33) and its grading A, B or C.

If σ represents combined uncertainty from the load and the resistance side, then this uncertainty can be reflected by a quantity $k_T\sigma_S$ above the mean spectra MDAF discussed in Chapter VI. σ_S is the uncertainty in the spectra shape. Thus, conservatism in design is achieved by obtaining a response spectrum $k_T\sigma_S$ above the MDAF. Thus, the design level of the dynamic amplification factor would be

$$\text{Design level of DAF} = \text{MDAF} + k_T\sigma_S$$

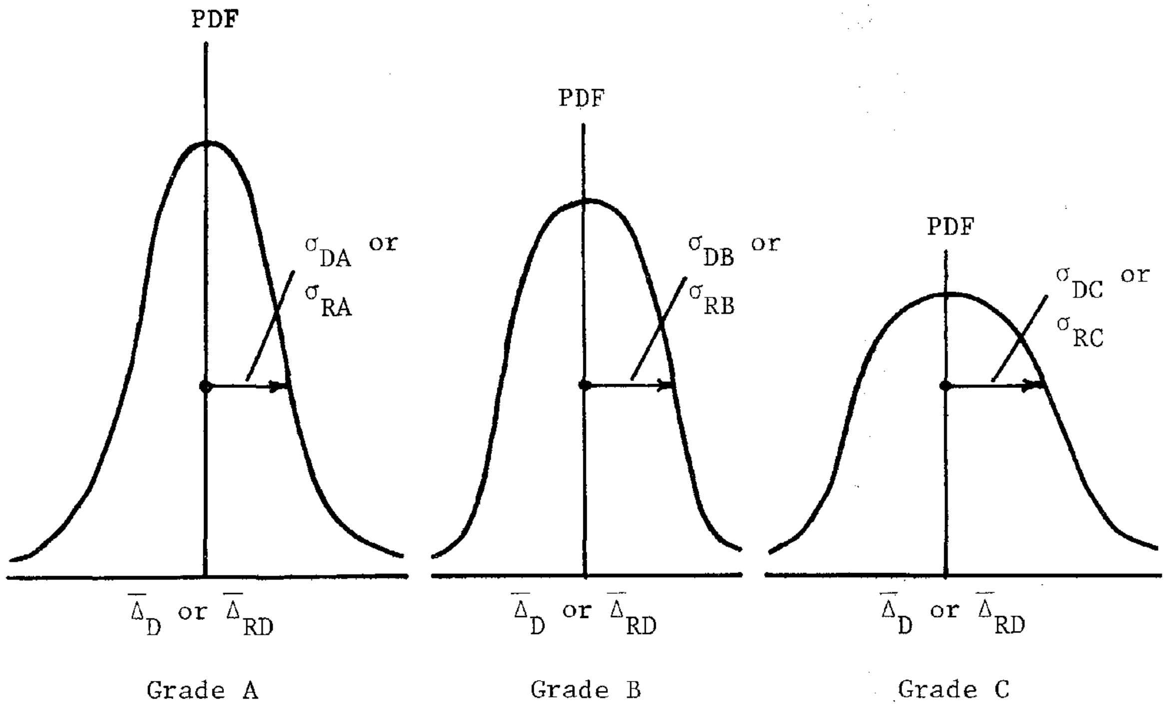
$$= \text{MDAF}\left(1 + k_T \frac{\sigma_S}{\text{MDAF}}\right)$$

$$\text{However } \frac{\sigma_S}{\text{MDAF}} = \text{Coeff. of variation of the spectral shape}$$

$$= V_S$$

$$\text{Design level DAF} = \text{MDAF}(1 + k_T V_S) \qquad \text{9-1}$$

Note that all uncertainty is represented by V_S, since this can be evaluated by statistical analysis of available response spectra.

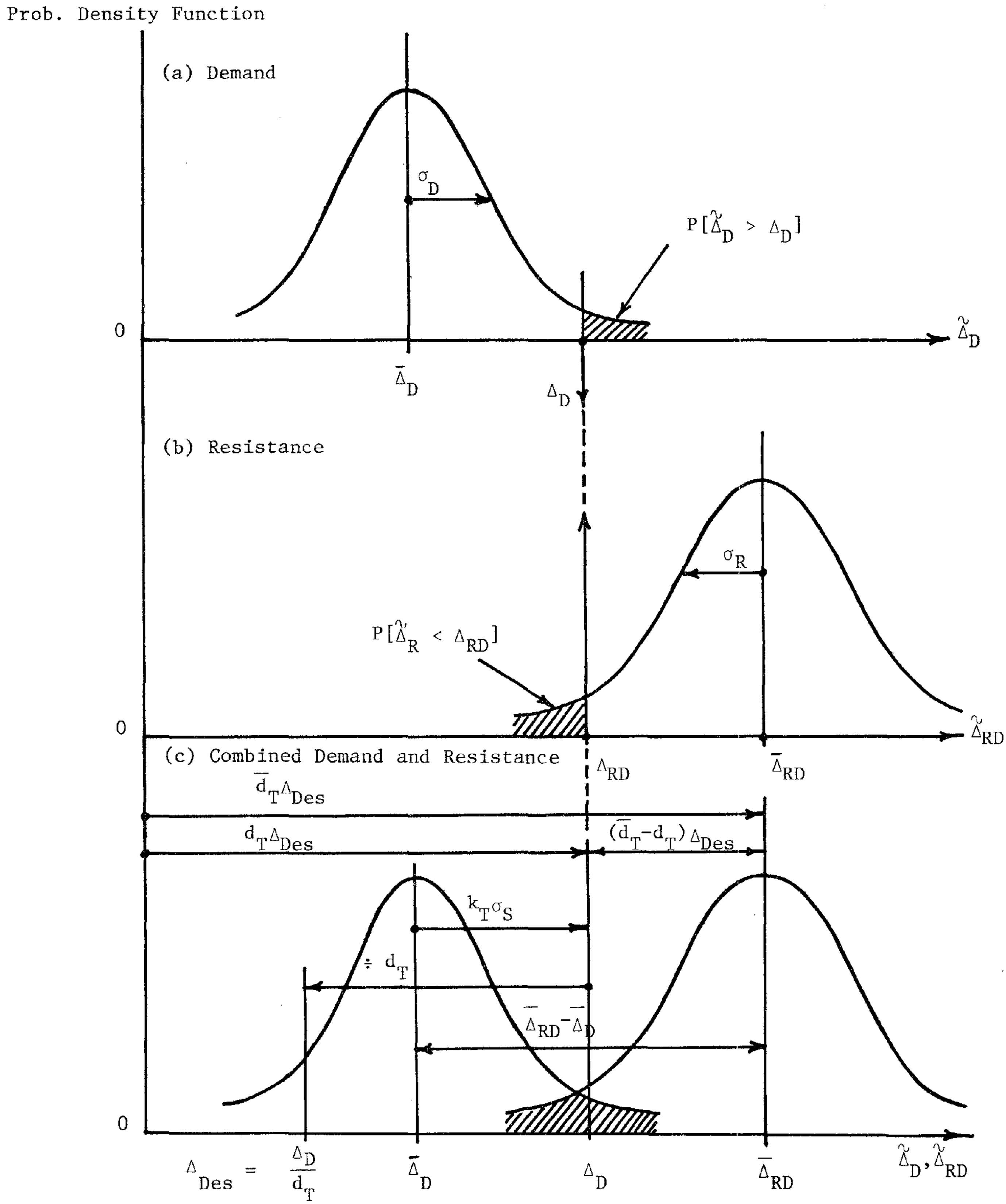


RELATIVE BEHAVIOR OF σ_D AND σ_R WITH RESPECT TO SYSTEM GRADE

FIGURE 9-4

Figure 9-5 shows the individual and combined random behavior of seismic demand $\tilde{\Delta}_D$ and structure damage resistance $\tilde{\Delta}_{RD}$. Reliable performance requires that the chance of the event that demand is greater than resistance ($\tilde{\Delta}_D > \tilde{\Delta}_{RD}$) be an acceptably small value. This chance is measured approximately by the shaded tail intersection area in Figure 9-5(c).

Because of the unknown values of the standard deviations σ_D and σ_R, the desired reliability corresponding to the small chance of ($\tilde{\Delta}_D > \tilde{\Delta}_R$) cannot be found mathematically, but it is most practically obtained by an adequate subjective separation of the mean values ($\bar{\Delta}_{RD} - \bar{\Delta}_D$). This separation is accomplished within the design procedure as follows:



Prob. Density Function
(a) Demand
σ_D
$P[\tilde{\Delta}_D > \Delta_D]$
0
$\tilde{\Delta}_D$
$\bar{\Delta}_D$
Δ_D
(b) Resistance
σ_R
$P[\tilde{\Delta}_R < \Delta_{RD}]$
0
$\tilde{\Delta}_{RD}$
(c) Combined Demand and Resistance
Δ_{RD}
$\bar{\Delta}_{RD}$
$\bar{d}_T \Delta_{Des}$
$d_T \Delta_{Des}$
$(\bar{d}_T - d_T)\Delta_{Des}$
$k_T \sigma_S$
$\div\, d_T$
$\bar{\Delta}_{RD} - \bar{\Delta}_D$
0
$\Delta_{Des} = \frac{\Delta_D}{d_T}$
$\bar{\Delta}_D$
Δ_D
$\bar{\Delta}_{RD}$
$\tilde{\Delta}_D, \tilde{\Delta}_{RD}$

FIGURE 9-5

- For a given system type the d_T value is selected so that the probability $P\left[\tilde{\Delta}_{RD} \leq \Delta_{RD}\right]$ is acceptably small ($\Delta_{RD} = d_T \cdot \Delta_{Des}$). Note that d_T is a reliable lower bound value of the random $\tilde{d}_T$ value of a given structure in Figure 9-3.
- For the given $R \cdot A_D$ value and the MDAF and coefficient of spectral variation V_S, an upper confidence level is used for the spectral input

  $$DDS = R \cdot A_D \cdot MDAF\,(1 + k_T V_S)$$

  where the confidence limit factor k_T assures that the computed SRSS response Δ_D has an acceptably small chance $P\ \tilde{\Delta}_D > \Delta_D$ of being exceeded.

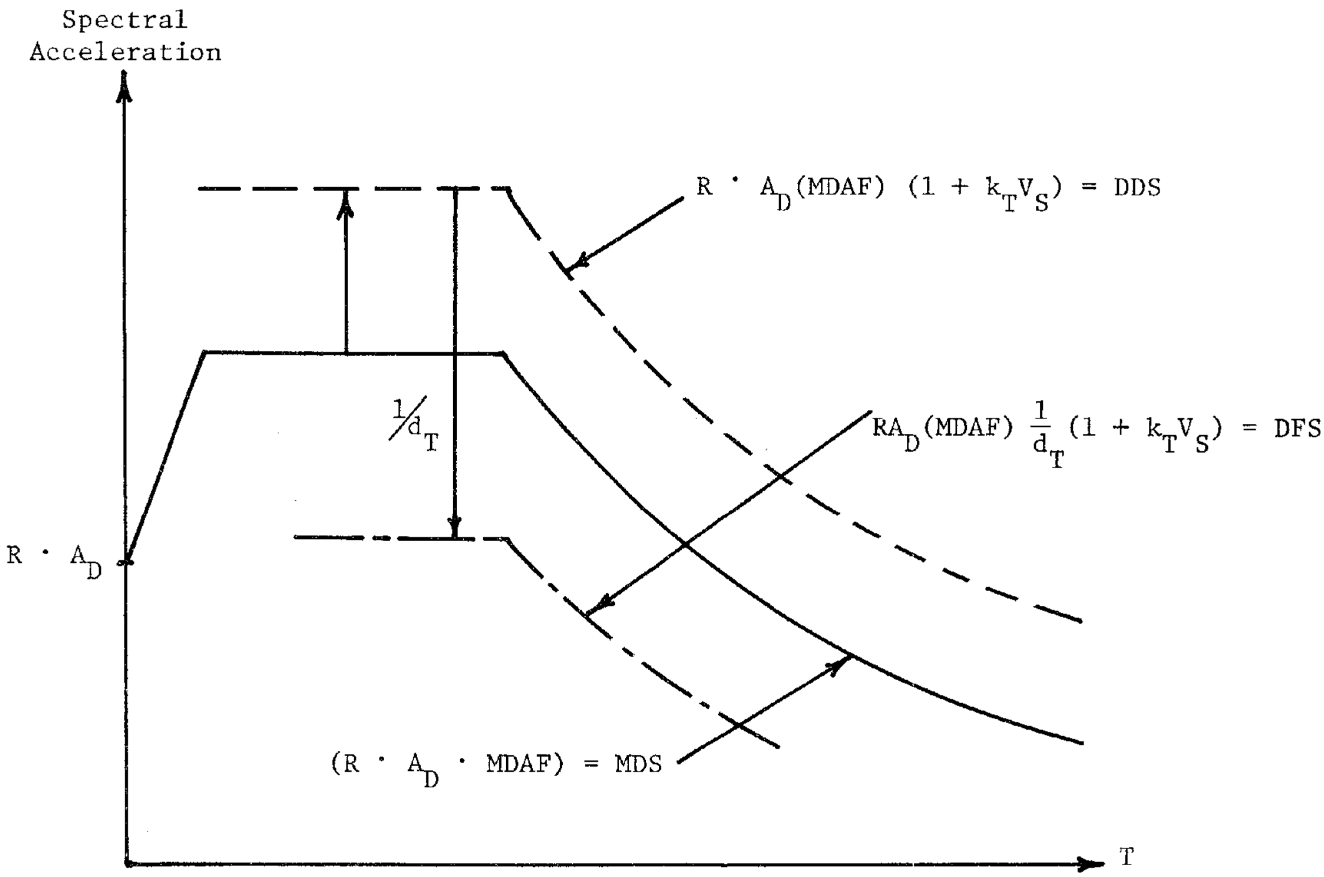
Engineering judgment is therefore applied to the assignment of the safe d_T and k_T values so that the combined chances of $P\left[\tilde{\Delta}_D > \Delta_D\right]$ and $P\left[\tilde{\Delta}_{RD} < \Delta_{RD}\right]$ are small enough to assure the reliable performance of a design based on the SRSS force response of the Design Force Spectrum,

$$DFS = R \cdot A_D \cdot (MDAF)\,\frac{1}{d_T}\,(1 + k_T V_S) \qquad \begin{matrix}4.2\\ \text{(repeated)}\end{matrix}$$

Figure 9-6 shows a summary of these spectral terms and relations. Different k_T values can provide for a constant reliability for each of the system Grades of A, B, or C. See Figure 9-7. Perhaps a more realistic representation of behavior would require that the reliable d_T value be also a function of the system grades - however, for simplicity the d_T is held constant for all grades of a given system type.

IX-5 A Period Dependent Confidence Level

This section will discuss an alternative proposed confidence level which should be given consideration - depending on the subjective



DAMAGE SPECTRUM RELATIONS

FIGURE 9-6

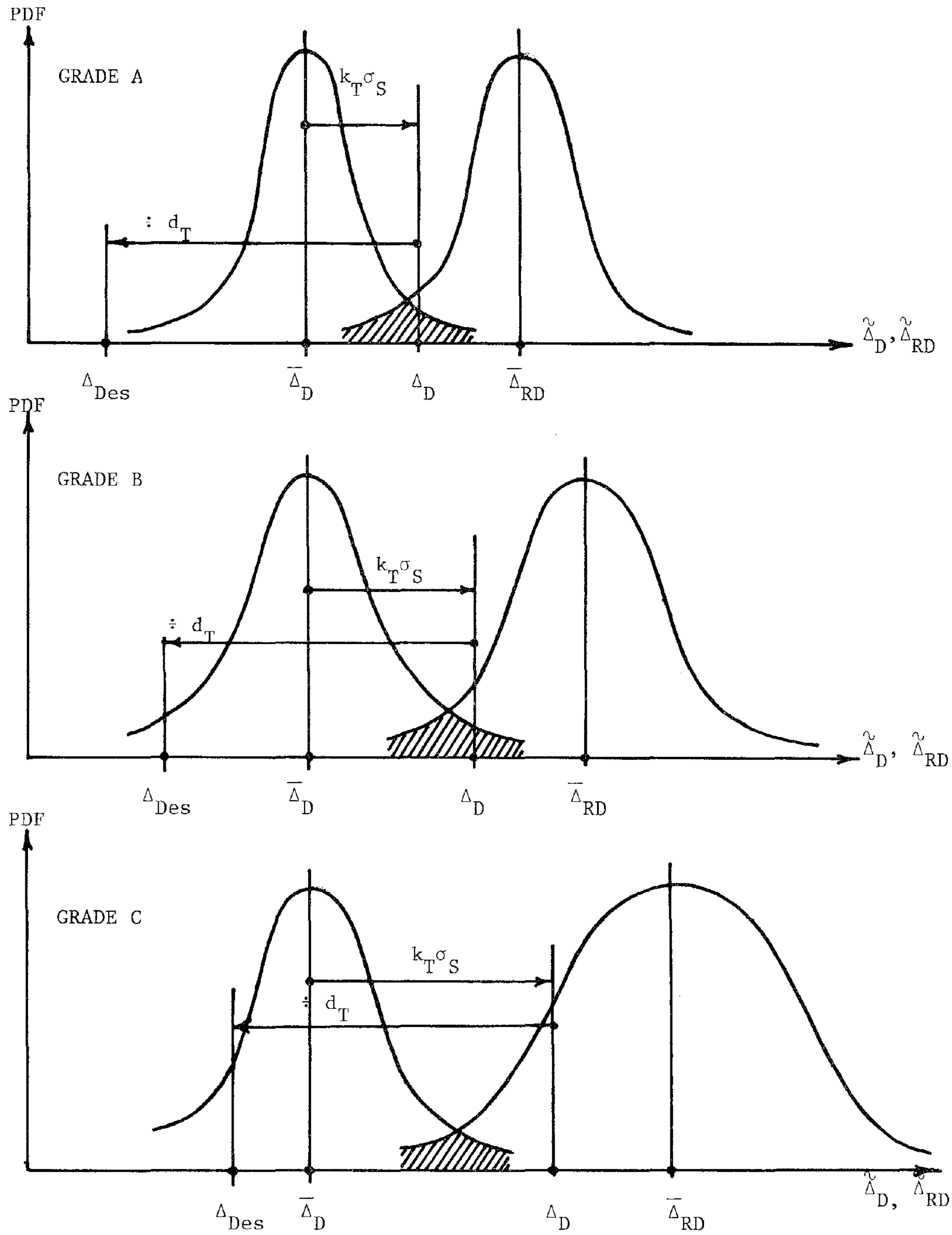

PDF
GRADE A
$k_T\sigma_S$
$\pm\ d_T$
Δ_{Des}
$\bar{\Delta}_D$
Δ_D
$\bar{\Delta}_{RD}$
$\tilde{\Delta}_D, \tilde{\Delta}_{RD}$
PDF
GRADE B
$k_T\sigma_S$
$\pm\ d_T$
Δ_{Des}
$\bar{\Delta}_D$
Δ_D
$\bar{\Delta}_{RD}$
$\tilde{\Delta}_D, \tilde{\Delta}_{RD}$
PDF
GRADE C
$k_T\sigma_S$
$\pm\ d_T$
Δ_{Des}
$\bar{\Delta}_D$
Δ_D
$\bar{\Delta}_{RD}$
$\tilde{\Delta}_D, \tilde{\Delta}_{RD}$

FIGURE 9-7

evaluation of local site conditions and high rise construction types and practices. The alternative is a long period dependent confidence level of the form (See Figure 9-8)

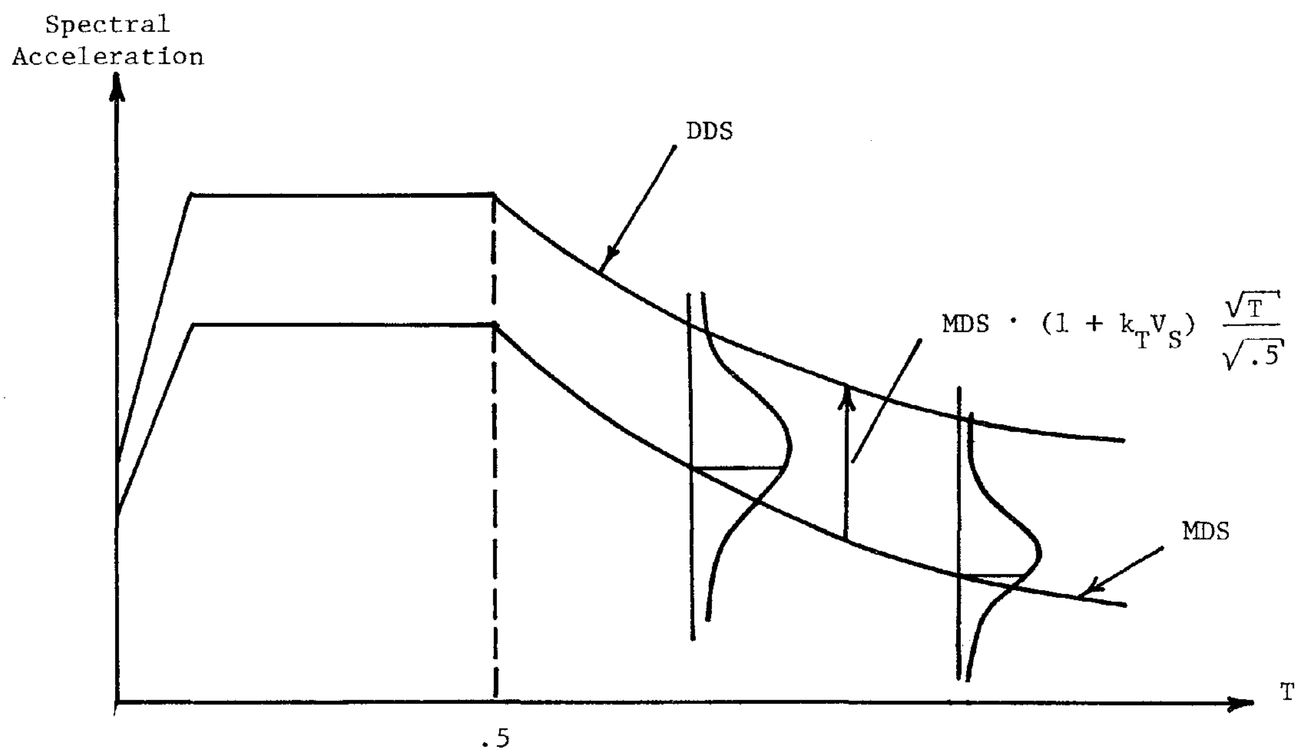
$$(1 + k_T V_S) \frac{\sqrt{T}}{\sqrt{0.5}} \quad \text{for } T > 0.5$$

which would result in spectra that would decrease with $\frac{1}{\sqrt{T}}$ (rather than 1/T): For example,

$$DFS = R \cdot A_D \cdot \frac{1}{d_T} \cdot MDAF \; (1 + k_T V_S) \frac{\sqrt{0.5}}{\sqrt{T}}$$

Some reasons for adopting of a $1/\sqrt{T}$ Design Spectrum Shape are:

- representation of multi-mode response within a simplified base shear format.
- engineers feel very uncomfortable if any method would produce loads less than UBC. In some cases, the 1/T format could result in design values below those recommended by UBC.
- uncertainty in the (long period) behavior of high rise structures
- actual increase in the Statistical V_S value as period increases. (See Appendix A).
- A form that is less sensitive to the different values of calculated structure periods.



PROPOSED PERIOD - DEPENDENT CONFIDENCE LEVEL

FIGURE 9-8

CHAPTER X

DESIGN PROCEDURE

SCOPE

In the preceding chapters, all of the parameters (R, A_D, A_C, (MDAF), β_T, d_T, d_{OT}, k_T, V_S) of the design spectra have been defined and developed. This chapter will assemble this information and incorporate it into the complete seismic design procedure. The important elements to be discussed are:

- Evaluation of Spectral Parameters.
- Construction and Purpose of each of the Design Spectra (DDS, DFS, DMS, CDS).
- Modeling of the structure for the Dynamic and Stress Analysis.
- Structural Weights, Loads and Load Factors for Ultimate Strength Design.
- The Design Procedure for Structural Elements and Related Deformation Evaluations.

....

Before proceeding to the discussion and evaluations related to these listed topics, it is important to emphasize that all methods and values are in the form of preliminary recommendations subject to review and adaptation to Nicaraguan Practice. With the realization that acceptable risk levels for structure use classes, structure types and materials, methods of analysis and member design, and construction methods may have unique characteristics for a given region, the complete design procedure must be finalized by the actual users - the Nicaraguan Planners and Engineers. Any required assistance is of course available from the John A. Blume Earthquake Engineering Center at Stanford University in order to ensure the fullest practical and widespread use of the proposed method. For example, a most useful and important type of work which can greatly assist in the finalization of design values is the conduct of actual building analyses with the proposed values. In this manner the proposed design results can be compared with past design experience and judged for adequacy and reasonableness.

X-1 Spectral Parameters

Code seismic load levels - as they have been developed - have always been subjective alterations to previously existing load levels. While theoretical analyses, tests, and earthquake experience may provide important information, the final improved code coefficients are always based on subjective acceptable values which are only indirectly related to theoretical computations.

The base line for seismic load level judgment employed in this proposed method was about double the 1973 UBC Design Level. (This is near to the response of a realistic damage threshold level of earthquake ground motion). Some upward or downward adjustment in the levels was made in order to account for higher or lower regional seismicity as measured from the Iso-Contour Map developed in this study. For a given structural system (K-Factor) type, this base line was applied to the B grade of quality. For A grade the design levels were reduced about 15% and for C grade the design levels were increased by about 15%. While damping, inelastic action, soil structure interaction, and reliability were all considered, it would not be all together candid to claim that the design levels were based on entirely quantitative calculations involving these behavior characteristics. The final seismic design level values and the associated spectral parameters represent an acceptable marriage between what may be termed as "theoretical" from the dynamic analysis viewpoint and "empirical or judgemental" from historical viewpoint of codes and engineering practice. Suggested values of β_T, MDAF, d_T, d_{OT}, and $(1 + K_T V_S)$ are given in Table 10-1. These are given here as "reasonable" numerical values, with the above statement of acceptable marriage between theory and practice in mind.

For practical use, further study should be made to refine these values. It should be noted that values given in Table 10-1 are independent of the use class of structures and the seismic region in which the structures are located. The values given are only functions of the type and quality grade of structures.

As an example, for Managua region and use group 2, Table 10-2 gives the values of spectral shape parameters. From this table, it can be deduced that for 0.67B type of structures (as an example), the value of the base shear derived from the plateau value H is about twice the 1973 UBC value, as discussed previously. A more detailed comparison between the proposed numerical values and the 1973 UBC values is given in Appendix D.

TABLE 10-1

Factors for Design Spectra

Type	β_T	Plateau Value of MDAF	d_T	d_{OT}	$(1 + k_T V_S)$
0.67A	10%	2.0	3.0	3.0	1.0
0.67B	10%	2.0	3.0	3.0	1.2
0.67C	10%	2.0	3.0	3.0	1.4
0.80A	10%	2.0	2.5	3.0	1.2
0.80B	10%	2.0	2.5	3.0	1.4
0.80C	10%	2.0	2.5	3.0	1.6
1.00A	10%	2.0	2.0	3.0	1.2
1.00B	10%	2.0	2.0	3.0	1.4
1.00C	10%	2.0	2.0	2.0	1.6
1.33A	10%	2.0	1.5	3.0	1.2
1.33B	10%	2.0	1.5	3.0	1.4
1.33C	10%	2.0	1.5	1.5	1.6

Values suggested here are preliminary.

TABLE 10-2

Factors for Design Spectra

Managua - Class 2 Structures

Type	H	H_{OT}	$\bar{\mu}_C$	$\bar{\mu}_{COT}$
0.67A	0.163	0.163	3.86	3.86
0.67B	0.196	0.196	3.86	3.86
0.67C	0.229	0.229	3.86	3.86
0.80A	0.236	0.165	3.22	3.86
0.80B	0.275	0.197	3.22	3.86
0.80C	0.317	0.229	3.22	3.86
1.00A	0.294	0.197	2.57	3.86
1.00B	0.343	0.229	2.57	3.86
1.00C	0.392	0.262	2.57	2.57
1.33A	0.391	0.195	1.93	3.86
1.33B	0.456	0.229	1.93	3.86
1.33C	0.520	0.520	1.93	1.93

$$H = (0.7)A_D \frac{(MDAF)}{d_T}(1 + k_T V_S)$$

$$H_{OT} = (0.7)A_D \frac{(MDAF)}{d_{OT}}(1 + k_T V_S)$$

Spectrum $= H$ for $T \leq 0.5$ sec
$= .5H/_T$ for $T > 0.5$ sec

For Hard to Medium soil conditions

$= H$ for $T \leq 0.8$ sec
$= 0.\frac{8H}{T}$ for $T > 0.8$ sec

For soft sites

H_{OT} is the spectral plateau corresponding to the design overturning moment spectrum (DMS). Note that the knowledge of H and H_{OT} is sufficient to describe the shape of the response spectrum. (See Figure 10-1). Also note that d_{OT} is larger than d_T and hence H_{OT} is lower than H for the structure types and grades that have ductile damage resistant chord details in their shear walls. Otherwise, no overturning moment reduction is permitted and the DMS is equal to DFS (see 1.33C type of structure).

The quantities $\bar{\mu}$ and $\bar{\mu}_{COT}$ can be considered as an overall measure of the ductility demand of the condemnation level earthquake (CDS).

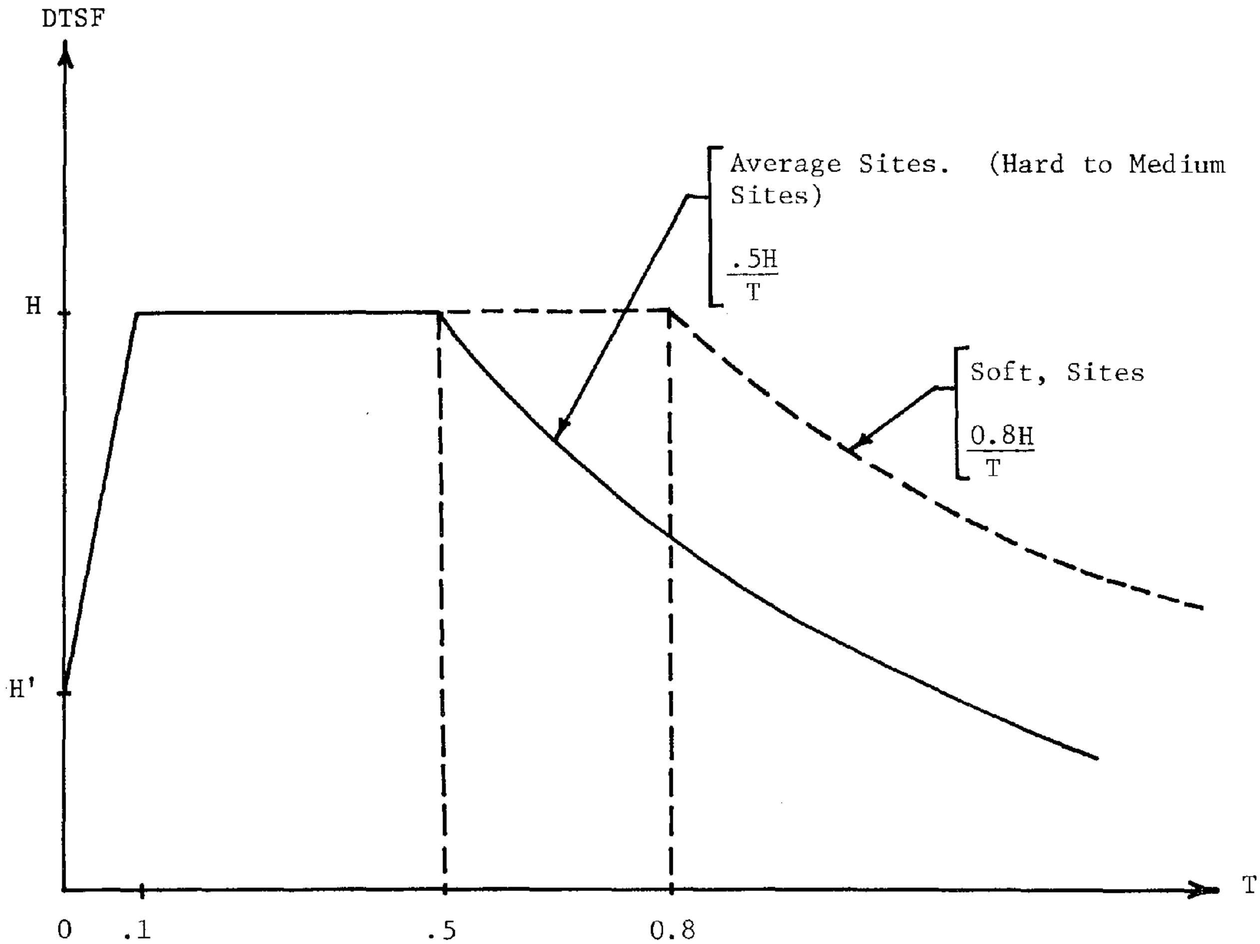
$$\bar{\mu}_C = \frac{CDS}{DFS} , \quad \bar{\mu}_{COT} = \frac{CDS}{DMS} \qquad 10\text{-}1$$

These factors can serve as multiplying factors to convert available calculated forces and deformations at the DFS (or DMS) level to corresponding elastic modal forces and deformations at the CDS level.

X-2 Construction of Design Spectra

In Chapter VI, the mean shape of the dynamic amplification factor (MDAF) for medium to hard and soft sites was developed. To obtain an effective shape of the response spectrum, consistent with the local seismicity, we multiplied the MDAF by RA_D for damage level earthquake and RA_C for condemnation level earthquake. Thus, RA_D(MDAF) gives the mean response spectrum shape for damage level earthquake and RA_C(MDAF) gives the mean response spectrum shape for condemnation level earthquake. This is shown in Figure 10-2.

For a given lateral force resisting system (such as K = 1.00B), the damping value β_T, the damage deformation factor d_T and the confidence



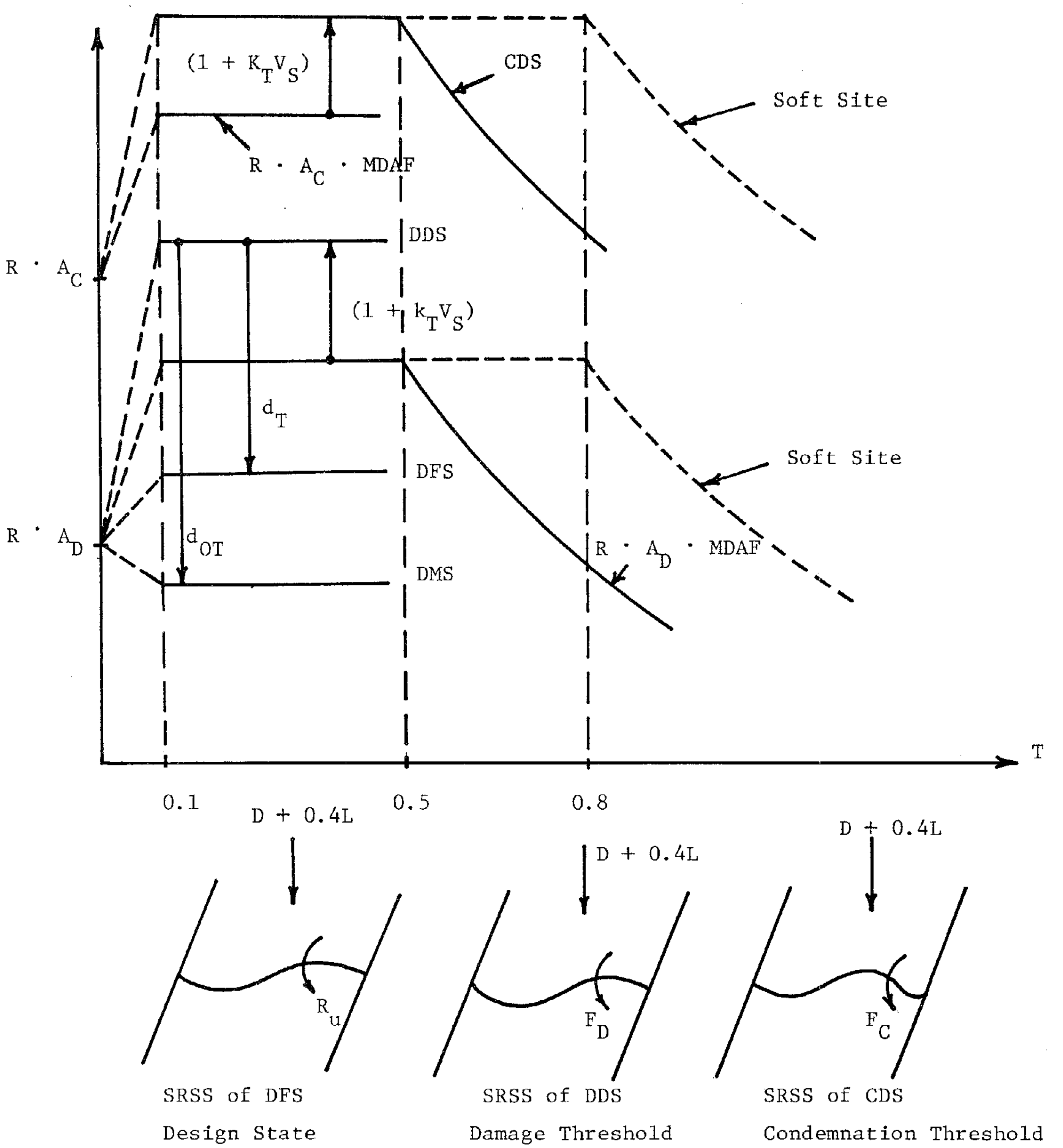
$$H' = \frac{R\ A_D}{d_T}(1 + k_T V_S)$$

$$H = \frac{R\ A_D}{d_T}(1 + k_T V_S)\ \mathrm{MDAF}$$

For design overturning moment, replace H with H_{OT} and d_T with d_{OT}.

For very soft sites, special site study needed.

FIGURE 10-1



DESIGN SPECTRA

FIGURE 10-2

level factor k_T are known. See Table 10-1. The deformation spectra for the damage and the condemnation level are formed by

$$DDS = R \cdot A_D \cdot (MDAF)\ (1 + k_T V_S) = d_T \cdot DFS \qquad \text{10-2 (4.3 repeated)}$$

$$CDS = R \cdot A_C \cdot (MDAF)\ (1 + k_T V_S) = \frac{A_C}{A_D} d_T \cdot DFS \qquad \text{10-3 (4.4 repeated)}$$

The SRSS response of the linear elastic structure model provides the deformations, (Figure 8-6)

Δ_D for DDS spectral input, and

Δ_C for CDS spectral input.

The same confidence factor k_T is used for both levels for simplicity. However it will be seen to have two purposes: In the CDS, k_T provides for the necessary reliable prediction of the inelastic structure deformations Δ_C at the condemnation level. In this case k_T allows for analytical errors due to the use of the elastic structure model for the computation of inelastic structure deformations. In the DDS and DFS, the k_T provides for a reliable high level of the DDS and the resulting DFS in order to account for the variable performance of the structure system type. In this case k_T gives a high design force value when the strength and deformation capacities of a system are relatively unknown or unreliable. (See Chapter IX).

The design force spectrum (DFS) is formed by dividing the DDS by the appropriate d_T factor for a given structure type. (Table 10-1). See Chapter VIII.

$$DFS = \frac{1}{d_T} \cdot DDS \qquad 10\text{-}4$$

Thus,

$$DFS = R \cdot A_D \cdot (MDAF) \frac{1}{d_T} (1 + k_T V_S) \qquad \begin{array}{c} 4\text{-}2 \\ \text{(repeated)} \end{array}$$

Similarly, the overturning moment spectrum is found by

$$DMS = RA_D \ (MDAF) \frac{1}{d_{OT}} (1 + k_T V_S) \qquad 10\text{-}5$$

In a strict sense the member design force spectrum DFS (and DMS) is not actually a spectrum since its ordinate values do not represent the response of a system to a definable earthquake ground motion. Its true meaning is as follows: when the members of a given structural system are designed (according to the stated design procedure) for the seismic forces due to the SRSS linear model response to the DFS (and DMS), then the resulting structure will have a reliable damage threshold at or beyond the deformations Δ_D as given by SRSS response to the DDS. Also, except in very rare cases, the local member inelastic deformation (ductility) demands will be within allowable limits at the condemnation threshold deformations Δ_C as given by the SRSS response to the CDS.

Story drifts in the structure at the damage threshold may be computed in terms of the SRSS response to the DDS. The resulting Δ_D values could be separated into the flexural and shear distortion components, since in most cases only the story shear distortion would relate to damage in nonstructural components. The resulting inter-story drift values could also be used to evaluate the P-Delta effects in the design procedure.

The structural members as designed for the DFS forces must be verified for their inelastic deformation demands at the condemnation threshold deformations Δ_C. Since the linear elastic structure model is used to obtain the Δ_C values as the SRSS response to the CDS, it is possible to use, in an approximate manner, elastic force ratios for the evaluation of ductility demands.

If Θ_{Des} is the member yield level deformation, and Θ_C is the member deformation due to the CDS, then the measure of the cyclic inelastic deformation or local "ductility" demand is

$$\mu_C = \frac{\Theta_C}{\Theta_{Des}} \qquad 10\text{-}6$$

The manner in which this deformation ratio can be converted into a force ratio is shown later in this chapter and in Appendix E which presents a detailed discussion of special design problems, such as P-Delta effects, drift, ductility and structural stability.

In summary, the design spectra should be used as follows:

- DFS should be used for the determination of seismic loads on members for strength design, except overturning effects on walls and foundations.

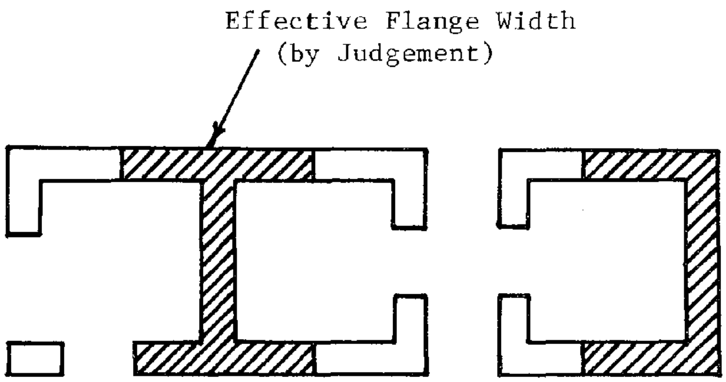
- DMS should be used for the determination of overturning moment for wall and foundation design.
- DDS should be used for the determination of structural drift corresponding to the damage level earthquake. This drift calculation will help in determining the adequacy of a design for non-structural damage control.
- CDS should be used for the determination of structural deformations corresponding to the condemnation level earthquake. This deformation calculation is used to check for the local ductility demand and stability of the structure.

X-3 Structure Modeling for Analysis

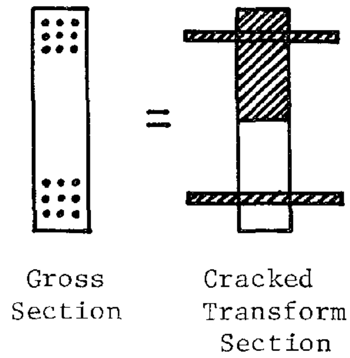
Modeling of the appropriate stiffness and constraint properties of each structural element and assemblage is one of the most important phases of the complete analysis for dynamic response and the related load-stress analysis.

The most elementary modeling approach is to employ gross (uncracked) section stiffness with rigid foundation constraints. Improvements to this elementary form may consist of: the use of cracked transformed sections; recognition of flooring, exterior cladding, and partition systems; and representation of known foundation flexibilities (see Figure 10-3 for typical modeling problems).

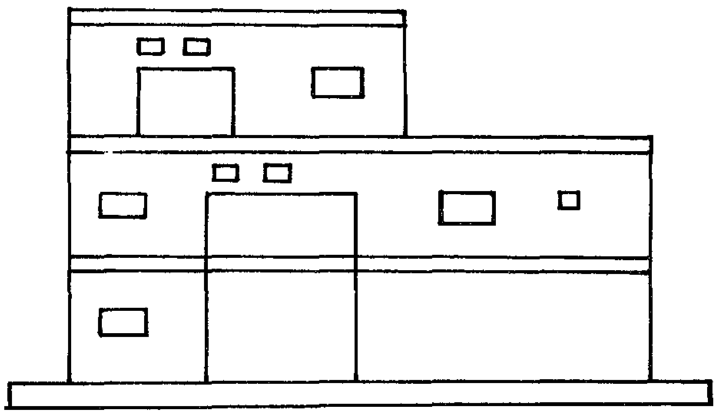
If at all possible, the owner and architect should be consulted for any proposed non-structural elements, revisions, or additions in order that these may be included in an analysis. Each major earthquake provides



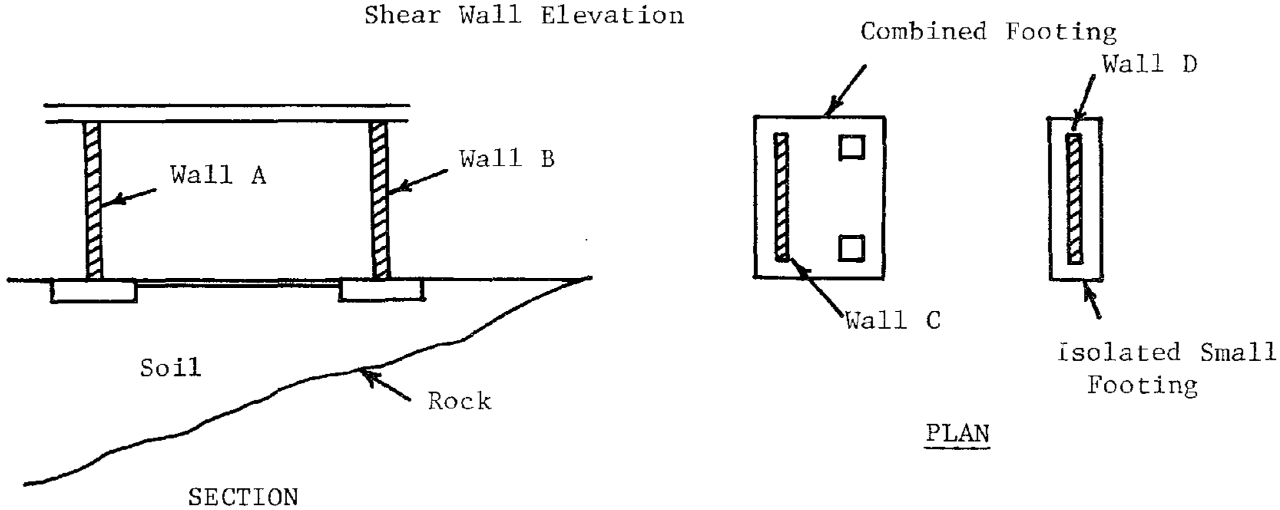
Elevator Tower Section



Heavily Reinforced
Wall Section



Shear Wall Elevation



Differences in Wall Footing Flexibilities

SOME MODELING PROBLEMS

FIGURE 10-3

cases of damage caused by the non-calculated stiffness effects of these elements (see Figure 10-4 for some non-structural problems). The designer should also see that wall details be provided to ensure that the wall can really behave as an integral unit. These include wall chord steel splices with adequate tie reinforcement, and well prepared construction joints with enough dowel reinforcing to prevent slippage. When overturning moment tension creates a foundation uplift condition, this should be recognized in modeling as a reduced stiffness in the foundation condition. Alternatively, a tie-down provision should be included in the foundation design.

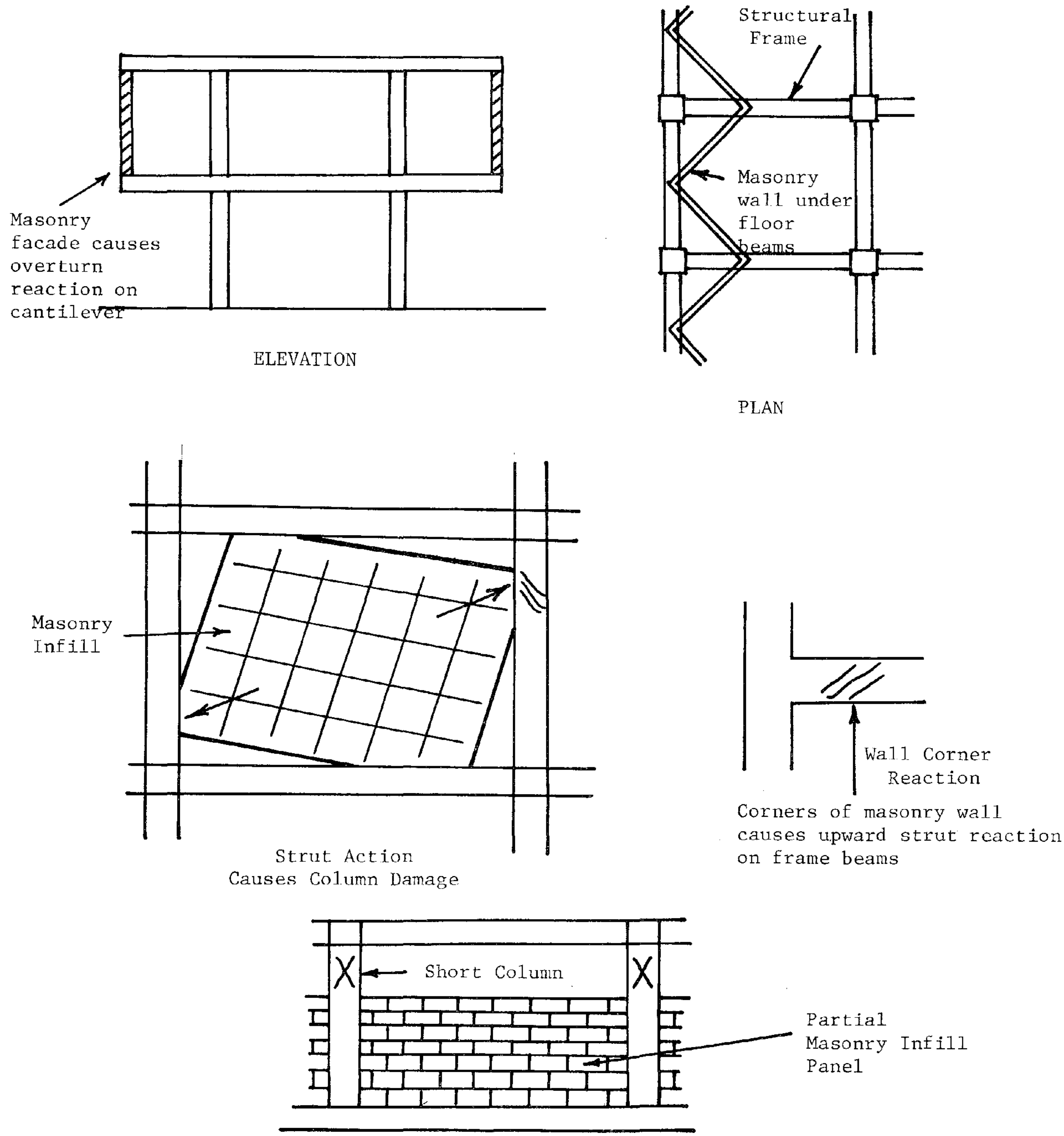
If non-structural flooring, partitions, and exterior cladding are not included in the model stiffness, then it is recommended that the calculated mode periods be decreased by a factor such as 10 percent.

In general, structural modeling for dynamic analysis should be carefully considered. Proper modeling can only be accomplished through experience and by extensive reviews of past cases.

Where uncertainties exist, upper and lower bound conditions should be investigated; for example, the upper and lower range of foundation constraint and flexibility may be used in two separate analyses.

X-4 Seismic Weights, Load Combinations and Load Factors

One basic principle that has guided the formulation of the proposed design procedure is that each step and parameter be rational. Specifically, there must be a simple rational explanation and reason for each representation of seismic input and the corresponding structural behavior. The



SOME NON STRUCTURAL ELEMENT EFFECTS

FIGURE 10-4

subject of load combinations and load factors provides a good example of this direct representation approach. Current code provisions will be stated for comparison.

- Seismic Structure Weight or Mass: At the time of the earthquake events corresponding to the PGA values of A_D or A_C a realistic, yet reasonably conservative, value must be assigned for the total structure weight or mass for the evaluation of inertia forces. Some amount of live load is to be expected and the judgement value of 40 percent is suggested. Therefore for dynamic analyses and for simplified base shear methods the weight or mass is dead load plus 40 percent live load (D + 0.4L). Present codes employ dead load only, except for warehouse structures.

- Load Combinations and Load Factors: Since the selected value of 40 percent live load is quite conservative for most structures in the sense that it is highly improbable that vertical live loads would exceed this value at the time of the earthquake, the load combination for the ultimate strength design R_u of members is dead load (D), 40 percent live load (.4L) and seismic forces E due to the SRSS response to the Design Force Spectrum (DFS)

$$R_u = D + 0.4L + E \qquad 10\text{-}7$$

In equation 10-7,

R_u = The required ultimate strength capacity for this specific case of loading. (Other cases may be for vertical load only such as (1.4D + 1.7L))

D = The member force (such as moment or shear) due to dead load.

L = The member force (such as moment or shear) due to the code specified value of live load.

E = The SRSS of the individual mode member force (such as moment or shear) due to the DFS.

While it appears, at first glance, that there are no load factors used in this ultimate strength load combination - these do exist. The purpose of load factors is to account for the chance of high possible loads and for differences between analysis and actual structure response. In the load combination of equation 10-7, the 0.4L is a reliable upper bound for vertical load uncertainties, and the value of E contains its load factor in the form of the confidence level factor $(1 + k_T V_S)$ of the DFS. It should be noted that each factor is applied directly to the source of load uncertainty. This can best be appreciated by a comparison with current code load combinations such as

$$(R_u)_{code} = 1.4\ (D + L + E_{code}) \qquad 10\text{-}8$$

where E_{code} is due to $V = KCW$

In this combination of equation 10-8, the safety or reliability of the member design for seismic resistance can vary according to the proportion of vertical to seismic load. For large D + L the section may be overdesigned, and for small D + L the section may be under-designed since 1.4 E_{code} is only about one half of reasonable damage level earthquake loads as represented by the DFS.

In order to account for the effects of vertical ground acceleration on the lateral force requirements, the following combination is used,

$$R_u = 0.8\ (D + E) \qquad 10\text{-}9$$

Here the most critical load condition, for overturning moment tension effects, occurs when there is only a small amount of live load. The 0.8D represents both the reduced dead and live load (due to vertical acceleration). The 0.8E reduces E corresponding to the small live load contained in the structure seismic mass, and also represents the smaller horizontal acceleration at the time of maximum vertical acceleration.

Preliminary computations have indicated that in moment resisting frames (and perhaps braced frames) the load combination of equation 10-7 may in some cases lead to axial column loads which are significantly smaller than those of the 1973 UBC. This problem needs to be pointed out and requires further study. To account for possible effects of vertical accelerations, it may be advisable to apply a load factor to D + 0.4L for such vertical elements.

X-5 Design Procedure Rules

In sections X-1 and X-2, instructions were given for the formulation of DFS, DDS, DMS and CDS. In this section a step by step procedure for the complete design sequence is given.

1. Given a use class of the structure (Table 5-5) and its location, the values of A_D and A_C can be determined from Iso-Contour Map or the Acceleration Zone Graph (Chapter II). The appropriate design spectra can be constructed with the above information together with the parameters MDAF, V_S, d_T, d_{OT} and k_T of a given structural type and soil condition (Table 10-1).

2. Formulate the linear elastic structure model and determine mode shapes and periods. Then, using the DFS developed in (1) above, obtain the SRSS force response E in the structural members.

3. Design members for load combinations on an ultimate strength basis for the following conditions.

   a) Load Factored Vertical Dead and Live Load;

      1.7 (D + L)

   b) DFS or DMS Force plus Vertical Dead and Live Load;

      (D + .4L) + E

   c) 0.8 (D + E) for vertical acceleration effects.

   In b) and c) above, the seismic load E is based on a (D + 0.4L) seismic weight of the structure.

4. Interstory drifts using the DDS are calculated as the SRSS of the individual modal drifts. These drifts shall not exceed 1% of the story height. This drift limitation is for damage control. (See Appendix E).

5. The member design procedure has produced known values for the individual member resistance values R_u, where

   $R_u > (D + 0.4L) + E; \quad R_u > 0.8(D + E); \quad R_u > 1.7(D + L)$ 10-10

   and commonly exceeds these load combinations because of the available section or sizing requirements as shown on the engineering plans for construction.

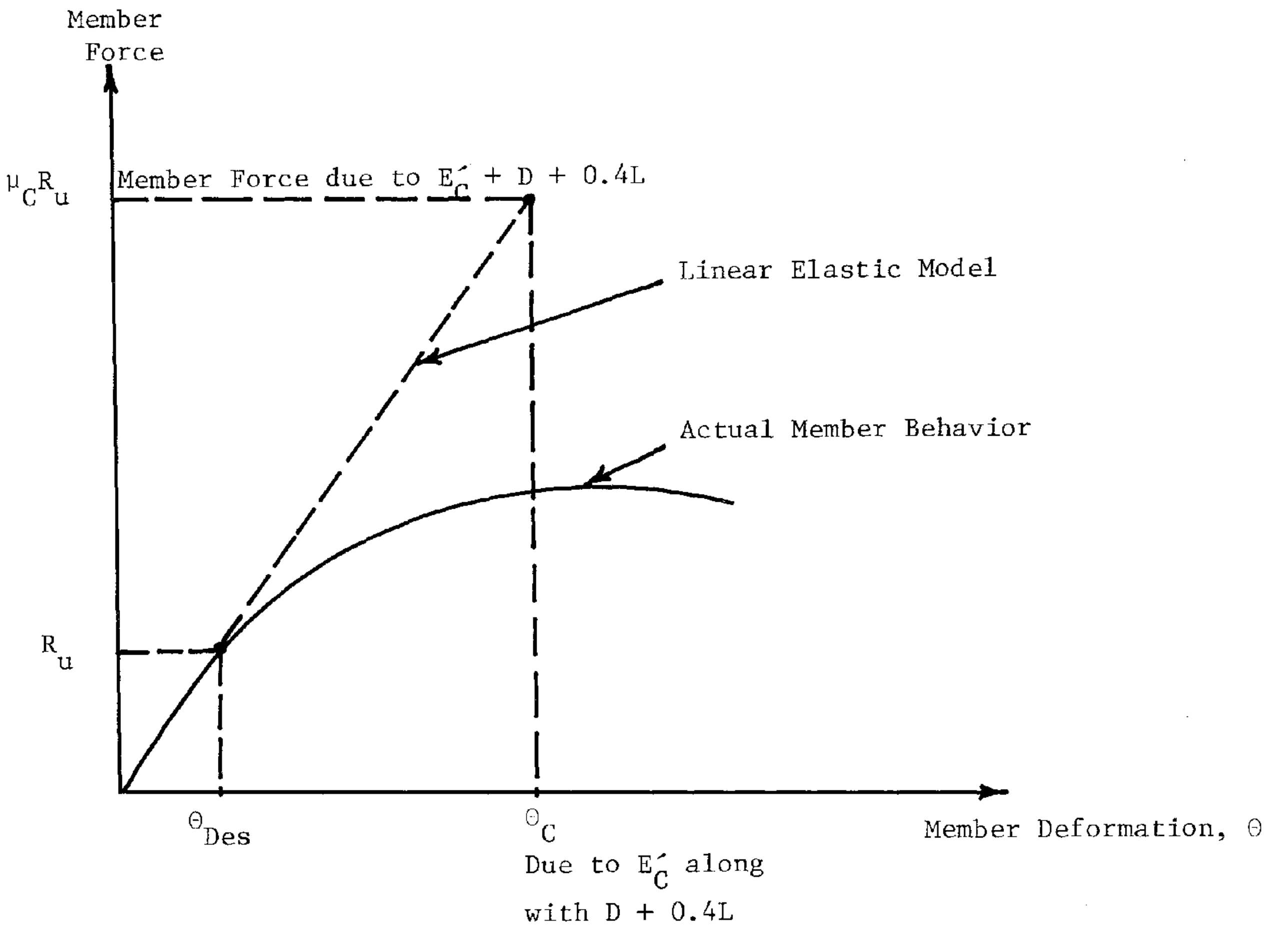
   Using the proportionality of forces to deformations in the elastic model response to the CDS, and defining the force in a member as E_C' due to the SRSS force response in the linear model due to the CDS, a measure of the local inelastic "ductility" demand in a member at the condemnation threshold is (see Figure 10-5).

$$\mu_C = \frac{(D + 0.4L + E_C')}{R_u} \qquad 10\text{-}11$$

$$\text{or} \quad \frac{0.8(D + E_C')}{R_u} \qquad 10\text{-}12$$

The computed values for μ_C are then to be compared with assigned allowable values. These allowable values have not yet been established at this reporting date, however, they could be of the order as follows:

Ductile Steel Beams	=	5	
Ductile Concrete Beams	=	4	
Columns in Non-Ductile Frames and X-Bracing Systems	=	1.5	
Concrete Shear Wall Flexure	=	2.	(In walls without ductile chords)
	=	4.	(In walls with ductile chords)
Concrete Shear Wall Shear	=	2.	(In walls and piers without ductile chords)
	=	3.	(In walls and piers with ductile chords)
Shear in Deep Concrete Spandrels	=	2.	



MEMBER FORCE - DEFORMATION BEHAVIOR

FIGURE 10-5

The availability of the μ_C values makes it possible to provide for extra detailing requirements in members with high μ_C values. For example, shear wall spandrels having $\mu_C > 2$ may require that all shear resistance be carried by shear reinforcement and none by the concrete. Alternatively, if the computed μ_C value is found to be less than 1 in shear wall flexure (this could occur in the upper stories of a shear wall structure), then the requirement for closely-spaced ties in ductile chords could be modified to larger spacing values. A more detailed discussion on ductility demands is presented in Appendix E.

The above procedure has been formulated for the case where the response spectrum method of analysis is used for the evaluation of seismic forces and deformations. It is intended that this procedure be used only for those special structures where importance and/or irregular configuration necessitate the increased theoretical accuracy of the complete response spectrum method. For the majority of reasonably regular structures, a simplified equivalent static load method, presented in Chapter XI, should be employed.

CHAPTER XI

SIMPLIFIED DESIGN METHOD

SCOPE

Having developed the response spectrum method of seismic analysis and design as a base line, a simplified equivalent static force approach is presented in this chapter. For the majority of regular structures, this approach is sufficient for seismic design considerations.

....

XI-1 Existing Methods of Analysis

Within the state-of-art of seismic analysis, the following methods are available. They are listed in decreasing order of analytical complexities.

1. Time history analysis. This type of dynamic analysis is conducted for an elastic or inelastic model of the structure. A proper modeling of the soil-structure interaction is included.
2. Response spectrum approach as outlined in this report.
3. Equivalent static load method with empirically derived mode shape and period.
4. Constant factor method where no structure period evaluation is required.

Method 1 above should only be used for highly important and or uniquely irregular structures. Irregularity applies to both the characteristics of configuration and to the potential for the unpredictable inelastic performance.

The response spectrum method developed so far in this report should be used as an alternate to the equivalent static method for cases where the importance of the structure and/or its structural and mass irregularities merit a more accurate prediction of the dynamic response. However, for a majority of the structures in use class 1, 2 and 3, a simplified design approach based on equivalent static load may be used. It should be pointed out that the equivalent static load method is an approximation of the response spectrum method. It should only be used for cases where this approximation is acceptable. As an example, for a building with setback or large torsional vibratory characteristics, the response spectrum method rather than the equivalent static load method should be used. A critical look at this decision parameter is needed.

For most of the low buildings with 1, 2 and possibly 3 stories, a constant factor method is quite sufficient.

XI-2 Justification for Simplified Design Method

For reasonably regular buildings, the proposed response spectrum design method provides a structure with the specified reliability of performance for the damage and condemnation levels of earthquake excitation. However, it is anticipated that a simplified "equivalent static load" procedure for these buildings would be most useful in design practice.

Naturally, the "equivalent static load" would be reasonably higher than the SRSS results of the DFS analyses so as to provide a conservative upper-bound envelope of mode shapes, modal periods, and the SRSS of the mode response load on each element. However, the practical advantages of the simplified "equivalent static" method are:

- Preliminary designs or sizing of the structure members can be performed without knowledge of final design stiffness values.
- Some projects with low design budgets can profit more from a better detailing job on the plans than from the expense of the response spectrum modal analysis.
- Some structures, particularly shear wall buildings, are rather difficult (arbitrary) to model as a dynamic stiffness - mass system. In these cases the resulting mode shapes, periods, and SRSS Response may vary widely, depending on the modeling decisions employed. Therefore,it may be just as good to use the conservative (high) equivalent static method.
- An "Equivalent Static Force" method may be crude and approximate, but in the design office it has one very distinct and necessary advantage, namely, that the statical equilibrium of each portion and element of the structure can be verified for the given lateral "equivalent forces". In the formal response spectrum analysis, the output is in the form of SRSS modal response for each elemental load or deformation and statics can not be applied. For example,

the SRSS value of a story shear cannot be obtained by the sum of the SRSS values of the story mass forces above the particular story.

Probably most designers would agree that it is well worth a little increase in lateral load in order to have the ability to check the final story shears, torsions, and overturning moments for statical equilibrium and for structure resistance capacity.

Speaking in favor of a properly formulated simplified "equivalent static force" analysis, with reasonable design details and with enough enforcement and inspection to assure that the structure is built as specified, this analysis can provide a structural design which is reliably resistant to both past recorded and future predicted earthquake motions.

In the review of the reports of failure of engineered buildings during the past major earthquakes of Anchorage, Alaska; Caracas, Venezuela; San Fernando, California; and Managua, Nicaragua, the proposed response spectrum design procedure or its "static force" equivalent would have corrected the design deficiencies of many of the failed structures. For example,

- In Alaska, strength design levels were far below those corresponding to a reasonable PGA value of A_D for the region, shear wall spandrel shears were not calculated in the "static" force analysis and equilibrium was not verified for each structure element or portion.
- In Caracas, again low design force levels were employed together with a non-recognition (unknown at the time of

design) of site soil column response interaction with the structure. Also principal failures were due to collapse of non ductile concrete framing - now outlawed in the U.S.A. and replaced by ductile moment resisting frame provisions.

- In San Fernando, again low design levels together with non ductile concrete frames, unprotected by shear walls, were principal causes of failure.
- In Managua, large building damage was due to possibly low design levels and non ductile concrete framing, but primarily due to a neglect of the plan torsion induced by non-calculated concrete service towers located eccentrically in frame buildings.
- In all past earthquakes, insufficient detailing and disregard of ductility requirements was a major cause of damage and failure.

Therefore, while the more detailed analyses involving time histories, refined structure models, and spectral analyses, are helpful in improving a given design and can give a better feeling of security to the designer and owner, probably the best insurance against future damage and collapse is by the universal application and enforcement of the simple "static load" design procedure; where this procedure includes:

- design force levels consistent with the regional seismicity (proper choice of the A_D value).
- recognition of soil-site response magnification.
- complete statical force analysis which carries lateral force shears, moments, and torsions down to the foundation.
- structural element detailing and connections necessary to resist the inelastic deformations of the condemnation (A_C) level earthquake.

The most essential element of the equivalent static force method is the force specification; this must include

(1) An empirical structure period equation based on structure system type, material, and configuration.

(2) An upper confidence limit shape of the DFS with some allowance for multi-mode effects. This can be accomplished by using a spectral shape beyond 0.5 secs for medium to hard soil and 0.8 secs for soft soil in the form of $\frac{1}{\sqrt{T}}$ instead of $\frac{1}{T}$.

(3) A rule (preferably linear) which provides information about the deformation shape of the structure.

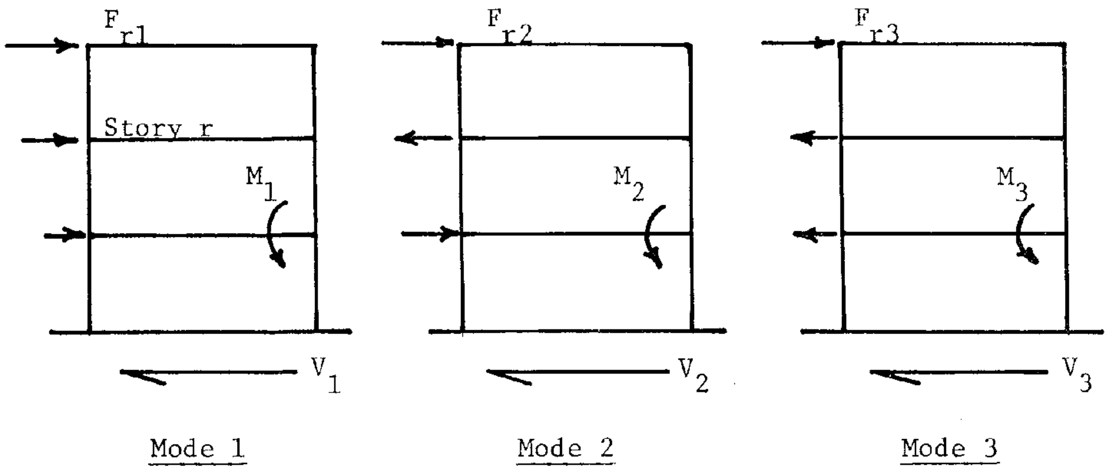
(4) A simple procedure (such as a constant multiple of design level deformations) for estimating structure drift and the damage DDS level, and deformation and related ductility demands at the condemnation CDS level.

The proposed response spectrum method is of course most essential in providing the theoretical basis for the evaluation of simplified static load levels and force distribution on the structure. However, it is recommended that first priority be given to the implementation of the "equivalent static force" method in order to produce the largest number of reliably safe new structures.

XI-3 Equivalent Static Force Method

The response spectrum method with the DFS input provides the following results. (See Figure 11-1). Each mode has lateral forces F_{rn} which

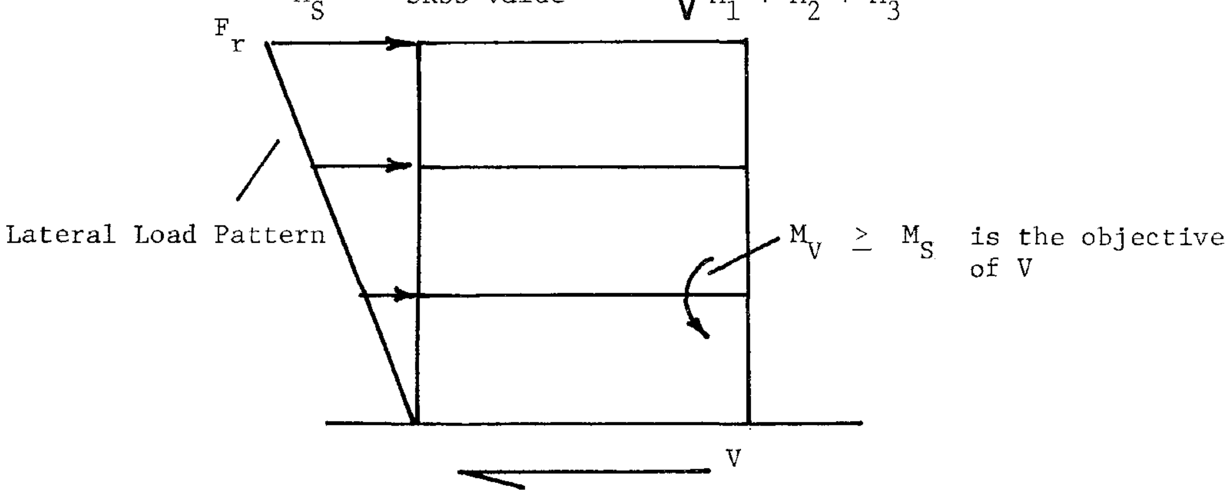
EXAMPLE FOR A 3-MODE SYSTEM:



M_n = Member Load Effect in Mode n

Member Seismic Design Load = M_S

M_S = SRSS value = $\sqrt{M_1^2 + M_2^2 + M_3^2}$



M_V = Member Seismic Design Load due to the Equivalent Static Force System.

RELATION OF RESPONSE SPECTRUM METHOD TO EQUIVALENT STATIC FORCE

FIGURE 11-1

cause a given load effect (such as moment, shear, or axial load) on a given member section. The member seismic design load (M_S) is the SRSS value of these individual mode effects.

An equivalent static force distribution based on a base shear V and a linear structure deformation shape must provide a reasonable upper bound (M_V) for the response spectrum load results M_S. In order to best explain the method of achieving the upper bound approximation of the response spectrum results, the basis of the equivalent static force method will be presented and then this will be followed by a discussion of the various factors as they relate to the response spectrum method.

Equivalent Static Force Method

Seismic loads for ultimate strength design are to be calculated from the following base shear (See Figure 11-2)

$$V = A\ D\ B\ M_N \qquad 11\text{-}1$$

where

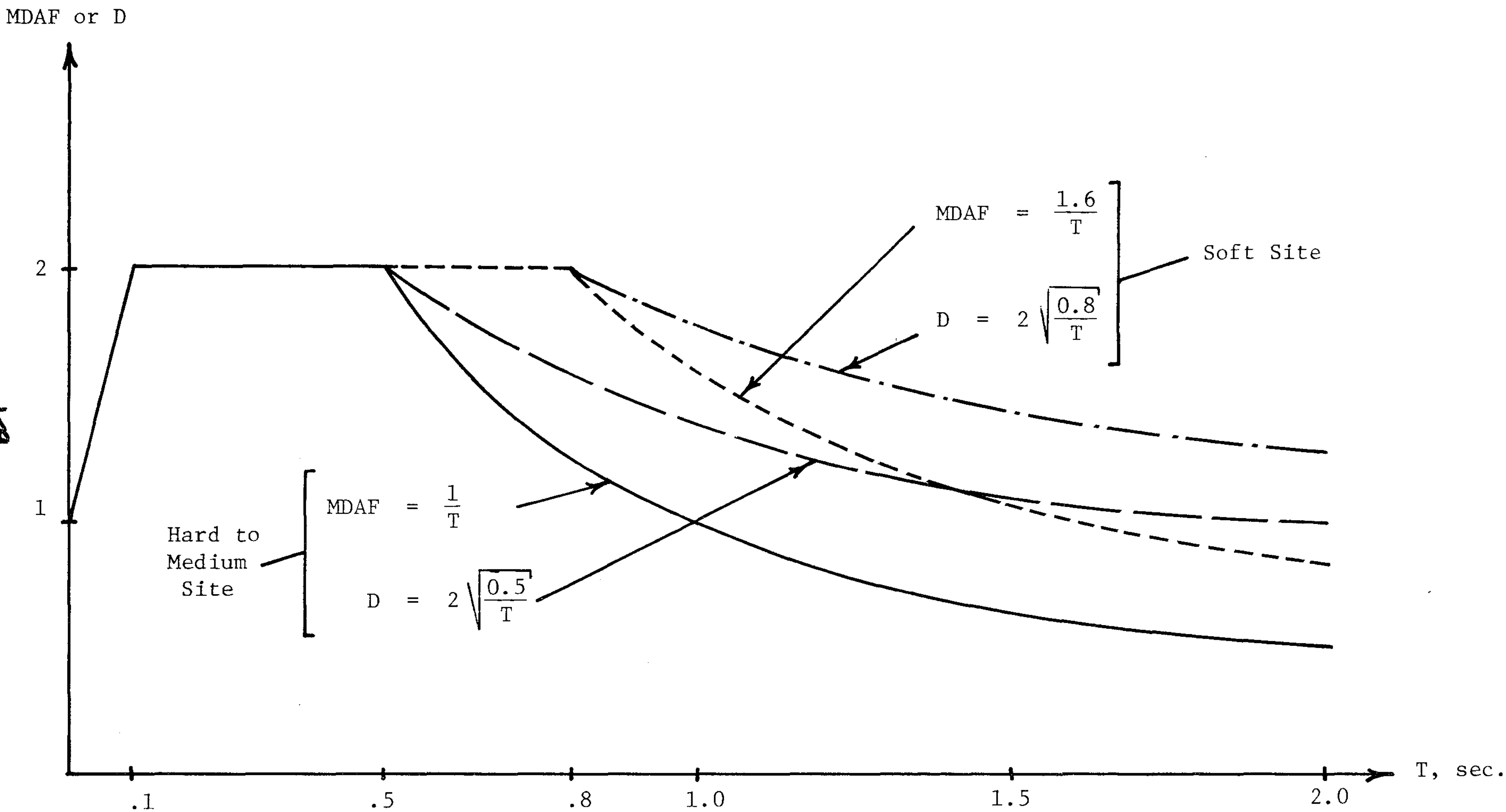
A = The PGA value in g units from the iso-contour map at the structure site. This value is the same as the A_D value obtained for a given use group.

D = Dynamic amplification factor given as follows (similar to the MDAF of Chapter VI).

For medium to hard soil site conditions-

$$D = 2 \quad \text{for} \quad T \leq 0.5 \text{ secs.}$$

$$= 2\sqrt{\frac{0.5}{T}} \quad \text{for} \quad T \geq 0.5 \text{ secs.}$$



SHAPE OF MDAF AND D FACTORS

FIGURE 11-2

For soft soil site conditions,

$$D = 2 \qquad \text{for } T \leq 0.8 \text{ secs.}$$

$$= 2\sqrt{\frac{0.8}{T}} \qquad \text{for } T \geq 0.8 \text{ secs.}$$

T = Structural fundamental period as given by the 1973 Uniform Building Code.

B = Structural system behavior factor (see Chapter X).

$$B = R \frac{1}{d_T}(1 + k_T V_S) \qquad 11\text{-}2$$

where d_T, R, k_T and V_S are discussed in previous chapters. Example numerical values are given in Chapter X.

M_N = Structural mass = W_N/g.

$$W_N = W_D + 0.4W_L \qquad 11\text{-}3$$

W_D is the dead weight of the structure, partitions, fixtures and other permanent attachments.

W_L is the code specified live load weight

- The base shear obtained by equation 11-1 should be distributed throughout the height of the structure according to 1973 UBC.
- The load combinations for ultimate strength design should be the same as discussed in Chapter X.
- The overturning moment reduction factor should be in the ratio of d_T to d_{OT} for each specific structural type. (Refer to Table 10-1).

- Damage Level Drift should be based on d_T times the calculated Base Shear Drift.
- Local Ductility Demands at the Condemnation Level should be evaluated as given Chapter X, but where the E_c' value is given by

$$\frac{A_C}{A_D} \cdot d_T \cdot E \qquad 11\text{-}4$$

where E is the member seismic load due to the base shear equation 11-1.

In the summary volume Technical Report #12B of the John A. Blume Earthquake Engineering Center, an example is given to demonstrate the use of this method.

XI-4 Discussion of Equivalent Static Force Method

The procedure used to develop the base shear equation 11-1 is as follows:

Beginning with a general form of

$$V = (\text{DFS}) \cdot (M_{\text{eff.}}) \cdot (\text{Safety Margin}) \qquad 11\text{-}5$$

where

$$\text{DFS} = R \cdot A_D \cdot (\text{MDAF}) \frac{1}{d_T} (1 + k_T V_S)$$

$M_{\text{eff.}}$ = Effective mass of the multi-mode structure (see Ref. 10). The safety margin consists of two components:

(1) a factor equal to $\sqrt{2T}$ or $\sqrt{1 \cdot 25T}$ that converts the (MDAF) to approximate multi-mode response spectrum in the range where multi-mode effects are believed to be important (T > 0.5 sec., or T > 0.8 sec. resp).

(2) a factor equal to

$$M_N / M_{eff} > 1$$

which provides for the approximation of SRSS effects by the single equivalent static force system. It has the general desirable property of increasing with the number of stories in the structure.

The Base Shear equation for medium to hard soil conditions and $T > 0.5$ is therefore

$$V = R \cdot A_D \cdot (MDAF) \frac{1}{d_T} (1 + k_T V_S) \cdot \sqrt{2T} \; M_{eff} \cdot \frac{M_N}{M_{eff}} \qquad 11\text{-}6$$

$$= A \cdot D \cdot B \cdot M_N$$

where

$$A = A_D$$

$$D = MDAF \sqrt{2T}$$

$$B = R \cdot \frac{1}{d_T} \cdot (1 + k_T V_S)$$

$$M_N = \text{Structure mass}$$

For $T \leq 0.5$ secs and medium to hard soil conditions.

$$V = RA_D (MDAF) \frac{1}{d_T} (1 + k_T V_S) \; M_{eff} \frac{M_N}{M_{eff}} \qquad 11\text{-}7$$

$$= ADBM_N$$

where, as before,

$$A = A_D$$

$$D = MDAF$$

$$B = R \frac{1}{d_T} \cdot (1 + k_T V_S)$$

$$M_N = \text{Structure mass}$$

Similar expressions for soft soil conditions would be

$$V = RA_D(\text{MDAF}) \frac{1}{d_T} (HK_T V_S) \sqrt{1.25T}\, M_{eff} \frac{M_N}{M_{eff}} \qquad 11\text{-}8$$

if $T \geq 0.8$ secs

$$= RA_D(\text{MDAF}) \frac{1}{d_T} (HK_T V_S)\, M_{eff} \frac{M_N}{M_{eff}} \qquad 11\text{-}9$$

if $T \leq 0.8$ secs

Thus,

$$V = A\ D\ B\ M_N$$

Again,

$$D = \text{MDAF} \sqrt{1.25T} \quad \text{for} \quad T \geq 0.8 \text{ secs}$$

$$= \text{MDAF} \quad \text{for} \quad T \leq 0.8 \text{ secs}$$

Note that the safety margin in the spectrum shape is introduced only in the region beyond 0.5 sec for medium to hard soil and 0.8 sec for soft soil. As mentioned earlier, this is where the multi-mode participation effects are important.

The methods of estimating the structure period T and distributing the base shear to each story as given by the 1973 Uniform Building Code are judged to be sufficient for the proposed equivalent force procedure. Some modifications in their use may be necessary to represent the structure types, materials, and construction practices of Nicaragua. This co-ordination is particularly necessary for the selection of a constant factor multiplier (method 4 in section XI-1) for one and two story structures. Any factor (such as 0.2g) must be consistent with the short period region of the response spectrum, the material working stresses, and the type of construction employed for these low-rise buildings.

CHAPTER XII

DISCUSSION OF THE PROPOSED DESIGN METHOD

SCOPE

This chapter discusses the particular aspects of structure type grading and the ductility demand analysis. An attempt is made to point out that a better understanding of the seismic behavior of structures is achieved by the application of the proposed procedures.

....

XII-1 The Descriptive Quality of Response Spectrum Analysis

The response spectra (DFS, DMS, DDS, CDS) and the SRSS method of computing response helps the designer to better understand the effects of earthquake ground motion on his building. This can be appreciated by reviewing the equivalent static load and working stress design method given in the existing codes,

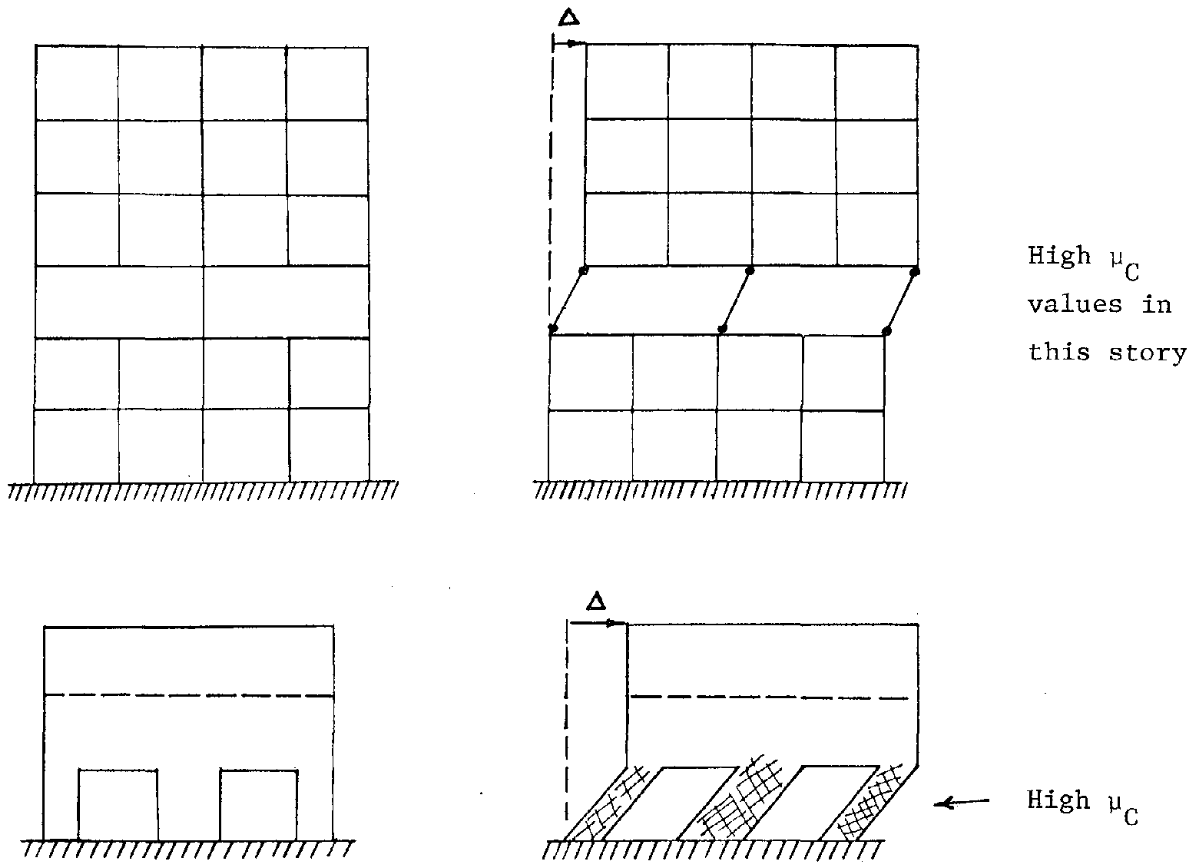
- A base shear V = KCW, having no direct relation to any identifiable earthquake spectrum, is distributed as lateral story forces.
- The lateral forces create member loads which, together with vertical dead and live loads, must be resisted with stresses no larger than 1 1/3 times the allowable material working stress.

This has been the complete design method; and its use by engineers, who were not experienced in strong motion earthquake resistant design procedures, has sometimes resulted in unsatisfactory designs. During actual strong motion earthquakes, some of these structures exhibited unpredicted destructive dynamic response such as soft-story distortions and torsional wracking. (See Figure 12-1). Their structural members, connections, and details lacked the toughness necessary to survive without excessive damage or collapse. This unsatisfactory performance is mainly due to a lack of awareness of a basic concept which is not made evident to the designer by the code equivalent static force - working stress method. The concept is that during any large earthquake a substantial amount of cyclic, non-elastic deformation capability is required in the structure beyond the design value (at 1 1/3 working stress). Therefore, in any proposed method, the more this method can be made to consider realistically forecasted earthquake deformation effects, the less likely it will be that a designer will commit the major errors of the past.

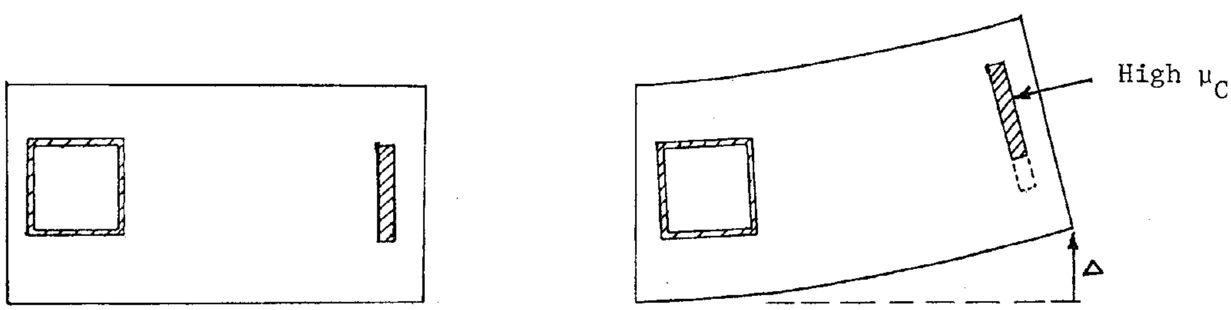
In the design methods (response spectra, and equivalent static force) as proposed in this report, the designer is made well aware of the force resistance and stiffness requirements for the damage level earthquake, and of the deformation and ductility demands of the condemnation level earthquake. Also, a critical grading of the structural system is required - such that the designer is made aware of the qualities and/or deficiencies of the structure. These aspects are discussed in the next section.

XII-2 Importance of the Quality Grading of Structure Types Together with a Deformation Demand Analysis

One of the major weak points of present seismic code methods, (either 1973 UBC, 1974 SEAOC, or 1976 UBC) is the lack of an effective



STORY DRIFT DUE TO
RESISTANCE DISCONTINUITIES IN ELEVATION



$$\mu_C = \frac{D + 0.4L + E_C'}{R_u}$$

TORSIONAL DRIFT DUE TO
RESISTANCE DISCONTINUITIES IN PLAN

FIGURE 12-1

discrimination between good and poor configurations of a given lateral force system. The characteristics of a structural configuration may be classified into two general groups according to: Degree of Redundancy or Back-up Systems and the Degree of Discontinuities in Resistance.

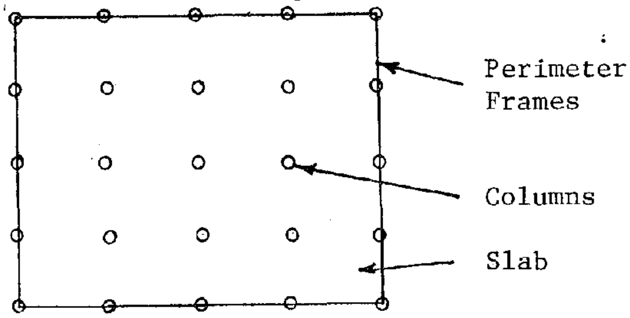
Deficiencies in Degree of Redundancy

In present codes, a given K factor type has the same seismic design load value without respect to the redundancy of the system. For example, referring to Figure 12-2, the perimeter frame structure (two frames) and the multiple frame structure both have the same $K = 0.67$ base shear factor. Similarly, the two tower wall system with low redundancy has the same $K = 1.33$ factor as the highly redundant box wall system. The grading system of A, B, or C in the proposed method offers the means of representing the degree of redundancy or "back-up" in a given structure type. High redundancy is assigned an A grade, with its low confidence level factor k_T and low design forces. Low redundancy is assigned a C grade with the penalty of higher design forces.

Deficiencies in the Distribution of Resistance Within a System

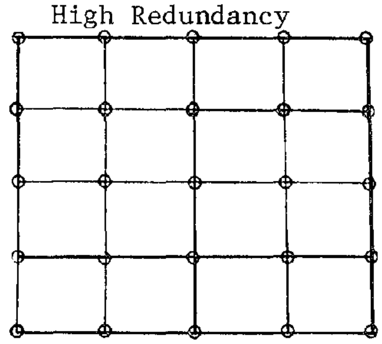
While the location and distribution of differences of structure stiffness (either in elevation or in plan) can be represented in the linear elastic structural model, the existence of unbalanced ductility demands in the structure (either in elevation or in plan) can be detected only after the member strengths are established by design and the condemnation deformation (CDS) analysis is performed. Except for a special requirement for columns (vertical load carrying capacity at about 4 times design deformation), present codes do not require any deformation analyses

Low Redundancy



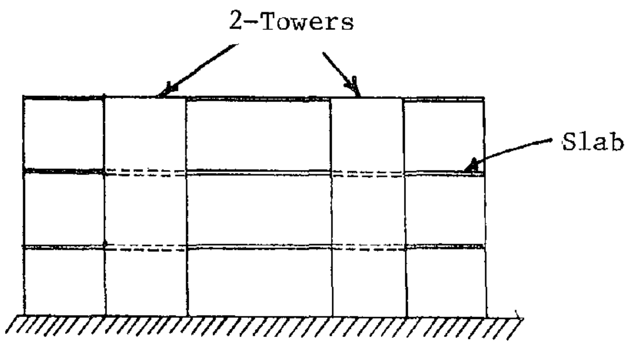
2 - Frames

High Redundancy



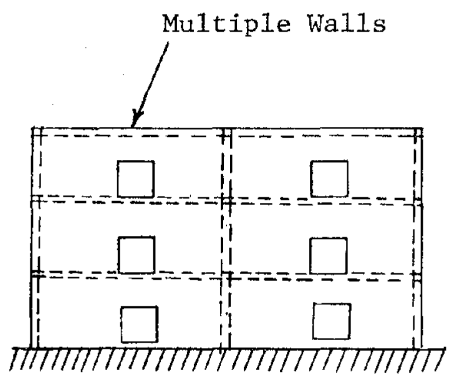
Multiple Frames

PLAN OF K = 0.67 SYSTEMS



Low Redundancy

2 - Tower System



High Redundancy

Box System

ELEVATION OF K = 1.33 SYSTEMS

SYSTEM REDUNDANCY COMPARISON

FIGURE 12-2

at the major earthquake level (condemnation level). The importance of determining the amounts of local ductility demands and the detection of possible concentration of these demands in any one story of the building elevation or any one frame or wall in the plan, lies in the fact that this deformation information is an indicator of either a good or poor seismic resistance system. Referring to Figure 12-1, the story drift and torsional plan drift effects due to unbalanced concentrations of ductility demands are shown. Many failures or near-failures in past earthquakes could have been avoided if the designer would have been aware of these unbalanced resistance conditions in his structure. In the proposed method the ductility demand, as indicated by μ_C, needs to be estimated. (See Chapter X and Appendix E).

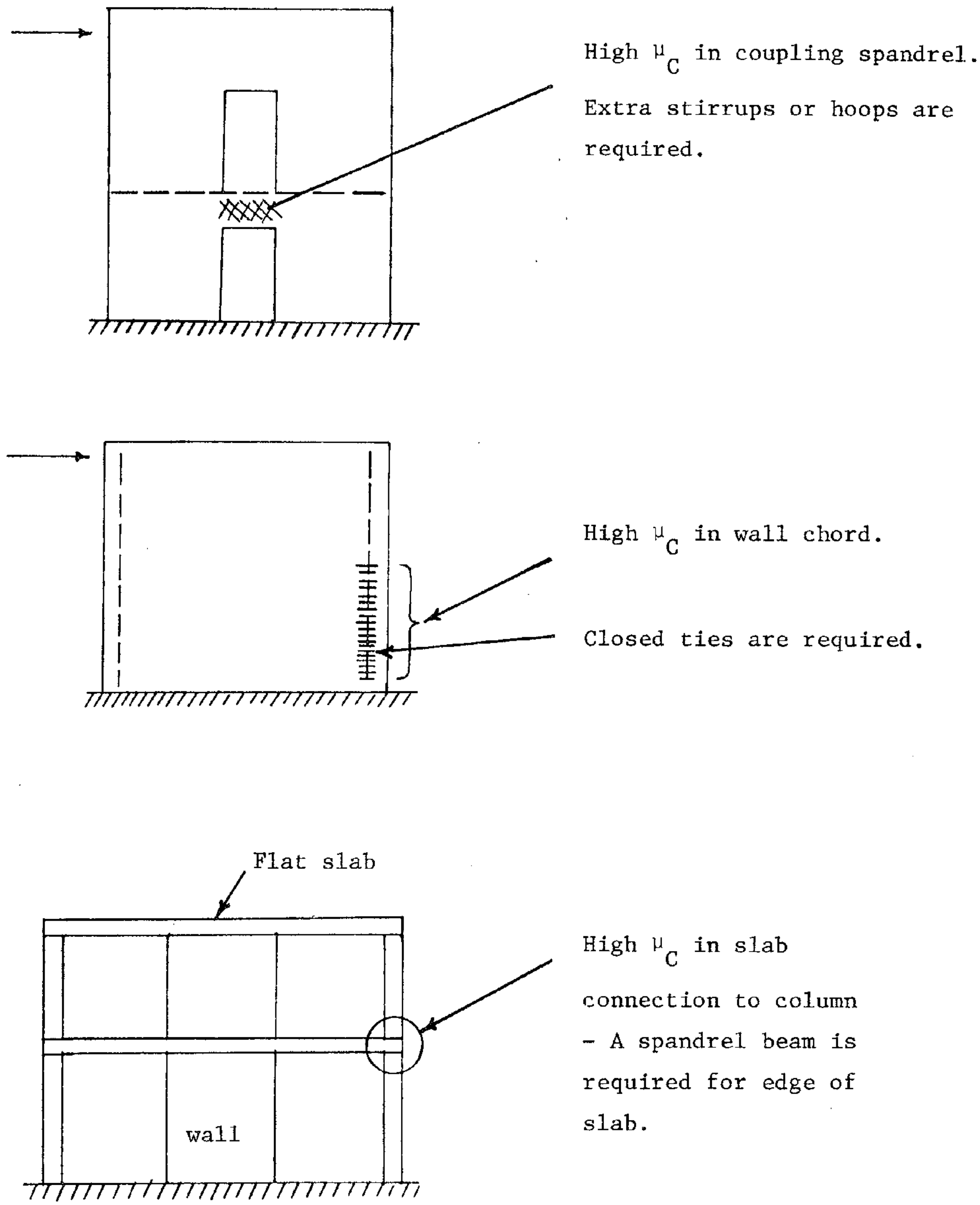
In addition to this capability to detect inelastic drift problems in a structure, the CDS analysis also provides a warning of intensified local damage conditions. Figure 12-3 shows some particular conditions where a high μ_C indicates early damage or local destruction - these locations require extra detailing in order to preserve the member or joint.

XII-3 A Discussion of Two Methods of Assigning the Design Force Spectrum

Given the design objectives of damage control for moderate earthquakes and condemnation protection (within allowable limits of ductility demand) for major earthquakes, there exist two alternative methods of assigning the member Design Force Spectrum: these may be termed as the DFS and the CFS.

DFS Method

The DFS is the method employed in the proposed design procedure. It basically assumes that a relatively small amount of inelastic defor-



LOCAL DUCTILITY DEMAND PROBLEMS

FIGURE 12-3

mation ($d_T \cdot \Delta_{Des}$) is admissable in members with the highest stress ratio at the damage threshold. When members are designed for the DFS then the damage control objective is achieved at a structure deformation given by the DDS. At this deformation the structure is still essentially linear elastic although some inelastic action occurs in some members; and therefore multi-mode response (SRSS) may be assumed to be valid for the computation of this deformation.

In order to verify that local ductility limits are not exceeded by the major or condemnation threshold earthquake, it is necessary to employ the assumption that the structure deformations are given by the linear structure response to the CDS spectrum. While this assumption is not theoretically valid because of the significant amount of inelastic action in the entire structure, reasonably conservative values can be achieved by use of a suitable confidence level for the CDS spectrum; also, conservative lower-bound values may be used for the allowable local ductility limits for given materials. Note that only two analyses need be made: one essentially elastic analysis at the damage threshold - and one hopefully conservative elastic approximation of inelastic deformations at the condemnation threshold.

CFS Method

This method employs the concept that the structure has an allowable ductility factor μ, which is related to the system type, and that the design yield level spectrum or CFS may be obtained from the condemnation threshold spectrum CDS by modifications in terms of μ. These modifications such as $\frac{1}{\mu}$ or $\frac{1}{\sqrt{2\mu-1}}$ in appropriate spectral frequency bands are empirical and are actually applicable only for single-degree-of-freedom elastic-perfectly plastic systems.

This method assumes that the design forces can be found by SRSS linear model response to the CFS spectrum. The CFS is not a real spectrum, however it gives force levels at which inelastic behavior is initiated in the structure.

The main disadvantages of this method are the need to assume both a general μ value and the modification method corresponding to the formation of the CFS spectrum. Since the CFS is an inelastic force spectrum for a structure having substantial inelastic deformation, it is difficult to visualize how the elastic structure modes, periods, and participation factors may be used to predict the response to the DFS. Also, it should not be tacitly assumed that the local member ductility demands are within allowable values just because a reasonable μ value has been employed for the total structure.

Because of these shortcomings of the CFS method, the DFS method is preferred for this work.

CHAPTER XIII

CONCLUSION

SCOPE

The results of this complete study are in the form of three major catagories: seismic hazard zoning and the related damage forecast; seismic load criteria; and the structural design procedure. These are reviewed for their direct applicability to Nicaragua Planning and Design Practice.

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XIII-1 Seismic Risk Zoning

With the Iso-Contour Map and Acceleration Zone Graphs for principal population centers it is possible to determine the PGA values of earthquake events having a given risk of exceedance during a structure life period. Design earthquakes can thereby be selected such that the risk of occurrence is consistant with the use priority of a proposed structure at a given site.

It is shown that for similar construction practices, the damage potential for a region should be directly related to the seismic hazard for the region. However, for insurance risk evaluation, the distribution of population and seismic hazard should be convolved. This aspect was discussed in part I of this study (reference 1) and concluded in Chapter III of this report.

XIII-2 Seismic Load Criteria

A statistical average response spectrum in the form of the MDAF can be scaled by the PGA value to represent the spectrum for any given future earthquake. With the known damping, deformation behavior, and reliability of a structural system the following design spectra are formed:

DFS - for seismic design forces

DMS - for seismic design overturning moments

DDS - for the damage threshold deformation

CDS - for the condemnation threshold deformation

Of these spectra, the DFS serves to illustrate the principal advantage of what may be termed as "local adaptability" which is inherent in this proposed design method.

$$DFS = R \cdot A_D \cdot (MDAF) \frac{1}{d_T} (1 + K_T V_S) \qquad \text{4-2 (repeated)}$$

A_D represents the PGA of the seismic event consistent with the selected damage risk for the structure use group. (Chapter V)

R converts forecasted seismological instrument time history input to real structure input. (Chapter VII)

MDAF represents the best estimate of the response spectrum for the future seismic event represented by the PGA value of A_D. The damping conforms to the lateral force resisting system. Soil Column Effects can also be represented in the MDAF shape. (Chapter VI)

d_T is the capability of the given lateral force resisting system to resist damage beyond the member design level. (Chapter VIII)

$(1 + k_T V_S)$ gives the opportunity to allow for the experience, reliability, and quality control associated with a lateral force system, its analysis, and method of construction. (Chapter IX)

When specific seismic design recommendations are formulated for adoption within a building regulation, the above DFS format allows the input of all of the important local factors and conditions of a given city, region, or country. If a simplified "static load" or base shear factor method is required, then a conservative multi-mode version of the DFS can be employed to provide the load level.

XIII-3 Structural Design Procedure

With the given design spectra (DFS, DMS, DDS, CDS), the SRSS method of modal superposition provides structure response. Empirical structure period equations and force distributions can be employed within the format of a simplified "Equivalent static force" method in order to provide an upper-bound approximation of this response. Load factors and load combinations are proposed for ultimate strength design such that the structure can reliably provide damage protection and condemnation prevention at the selected earthquake risk levels. A required deformation analysis at the condemnation earthquake level serves to enforce the need for ductile connections and details at the locations of high computed values of inelastic deformation demands.

In conclusion, the seismic hazard mapping of Nicaragua is in a final form based on all available data. If new methods are developed and or new data becomes available, then an upgrading system is available to reflect these improvements. The proposed seismic design procedure is a form which now requires the direct input of criteria from Nicaraguan Planners and Engineers. Risk levels for structure use groups must be finalized by planners. Engineers must adapt ultimate design equations, allowable stress levels, methods of analyses, system type grading rules, and allowable ductility values, for applicability to Nicaraguan materials, construction practice, and enforcement procedures. The John A. Blume Earthquake Engineering Center will assist in any way necessary to provide an effective complete seismic design regulation for Nicaragua.

While the attention in this study has been directed towards a design regulation for new construction, it is extremely important also that a major effort be devoted to the strengthening of existing facilities. This is especially critical for cities such as Leon and Granada, since it has been observed in past earthquakes that a major contribution to number of fatalities and property damage result from the failure of older structures. It is hoped that officials in these cities will get sufficient information from this study to evaluate the adequacy of existing design procedures and existing structures.

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APPENDIX A

A STATISTICAL ANALYSIS OF RESPONSE SPECTRA*

*This work was done by Mr. David Tan, a Graduate Student in Civil Engineering

A Statistical Analysis of Response Spectra

As part of the seismic risk analysis project for Managua, probability based pseudo-absolute acceleration (S_a) response spectra are to be determined. These will serve the purpose of providing structural engineers with the seismic load values for which their structures will be designed. This report gives a summary of the work completed so far, and outlines that which is to follow.

Data Base Selection. S_a spectra are derived from accelerograms, of which only a limited number have been recorded in Managua. The more useful of these accelerograms have been digitized by the U. S. Geological Survey and their response spectra computed (Virgilio Perez, "Time-dependent Spectral Analysis of Four Managua Earthquake Records," Managua, Nicaragua Earthquake of December 23, 1972 Earthquake Engineering Research Institute Conference Proceedings, Volume 1, November, 1973). The 8 spectra representing the two directions of recorded horizontal motions were chosen to form part of the data base, and are listed in Table 1 as records 25 to 32.

Though some amount of information is contained in these 8 records, it is desirable to obtain a larger sample size in order to achieve a sounder basis for prediction. Since spectrum shapes are determined to a large degree by the geology of the recording site, it is reasonable to assume that a future earthquake will not produce an S_a spectrum with a shape vastly different from those of the eight records. However, some minor variation in shape may be expected, and to take this possibility into account, the eight record data base was augmented by additional records.

To carry out this augmentation, the following procedure was followed. The shapes of the velocity response spectra (of the eight Managua records) which were computed by Perez were studied carefully. Then, a search was made through velocity response spectra graphs of United States earthquakes for those that resembled the shapes of Managua spectra. These U. S. velocity response spectra shapes were contained in "Analysis of Strong Motion Earthquake Accelerograms, Volume III - Response Spectra," Earthquake Engineering Research Laboratory, California Institute of Technology. Although tripartite logarithmic spectra plots were also available in both the Perez and CIT publications, velocity response spectra were used for convenience as the most effective method of choosing similar shapes. However, statistical analysis was performed on S_a in comparison. Twenty-four U. S. records were chosen in this manner, and are listed in Table 1 as records 1 to 24. Thus, the data base as augmented consisted of 32 records.

It may seem that some degree of arbitrariness was involved in the forming of the data base. It is admitted that the data base chosen may be incomplete or inaccurate in its representation of future expected response spectra because it consists of a core of only eight Managua records, and 24 supplementary records that resemble the eight. However, with the present state of available information, one can do no better. Structures will have to be built and more will be gained from the use of available though possibly incomplete information than by their rejection. The incorporation of 24 additional records is believed to result in a better data base, because of the following reasoning: In a future earthquake, its S_a spectrum may be expected to resemble in a general way the shapes of the eight Managua records. However, minor variations will probably occur. These variations may not be adequately represented in the eight records.

Therefore, why not search for actual earthquake records with spectra that resemble the spectra shapes of the eight records? The inclusion of these supplementary records into the data base will result in one that is more representative of spectra shapes from future earthquakes.

The data base selected here represents prior information, in the language of Bayesian decision analysis. This is thus considered tentative and subject to modification in the light of new information that may be made available by future earthquakes.

Data Base Statistics. The data base having been selected, the next step consisted of determining its statistics. First, pseudo acceleration response spectra values were obtained for each record, in periods from 0.05 to 1.0 second in intervals of 0.05 second, from 1.1 to 2.0 seconds in intervals of 0.1 second, and from 2.2 to 3.0 seconds in intervals of 0.2 second, all for damping ratios of 0.00, 0.02, 0.05, and 0.10. For records 1 to 24, this was done by multiplying displacement response spectra values (listed in the previously cited CIT report) by the respective (circular frequency)^2 values. For records 25 to 32, S_a spectra values for periods 0.10 second and higher have been previously computed and were generously supplied by V. Perez of the U. S. Geological Survey. For increments of period equal to .05 second, a response spectra computer program (developed at CIT) was run with accelerogram data supplied by C. F. Knudson of the U. S. Geological Survey, the displacement response spectra value extracted, and the S_a value computed.

The dynamic amplification factors (DAF) corresponding to these S_a values were computed next. Each S_a value was divided by the peak ground acceleration (pga) of the accelerogram from which it was derived; the resulting quotient is the DAF value.

Simple statistics (mean, standard deviation, and coefficient of variation) of the DAF values were determined, considering the DAF's from all 32 records, then only the 24 U. S. records, and only the eight Managua records. The latter two sets of results were computed for comparison purposes. The results are presented in graphs in the succeeding pages.

Outline of Future Work. Succeeding work shall consist of the following:

(1) Probability distributions shall be fitted to the DAF for each period and damping. These shall be combined with probability distributions of pga derived in Part I of the Seismic Risk for Nicaragua report to arrive at probability distributions for S_a.

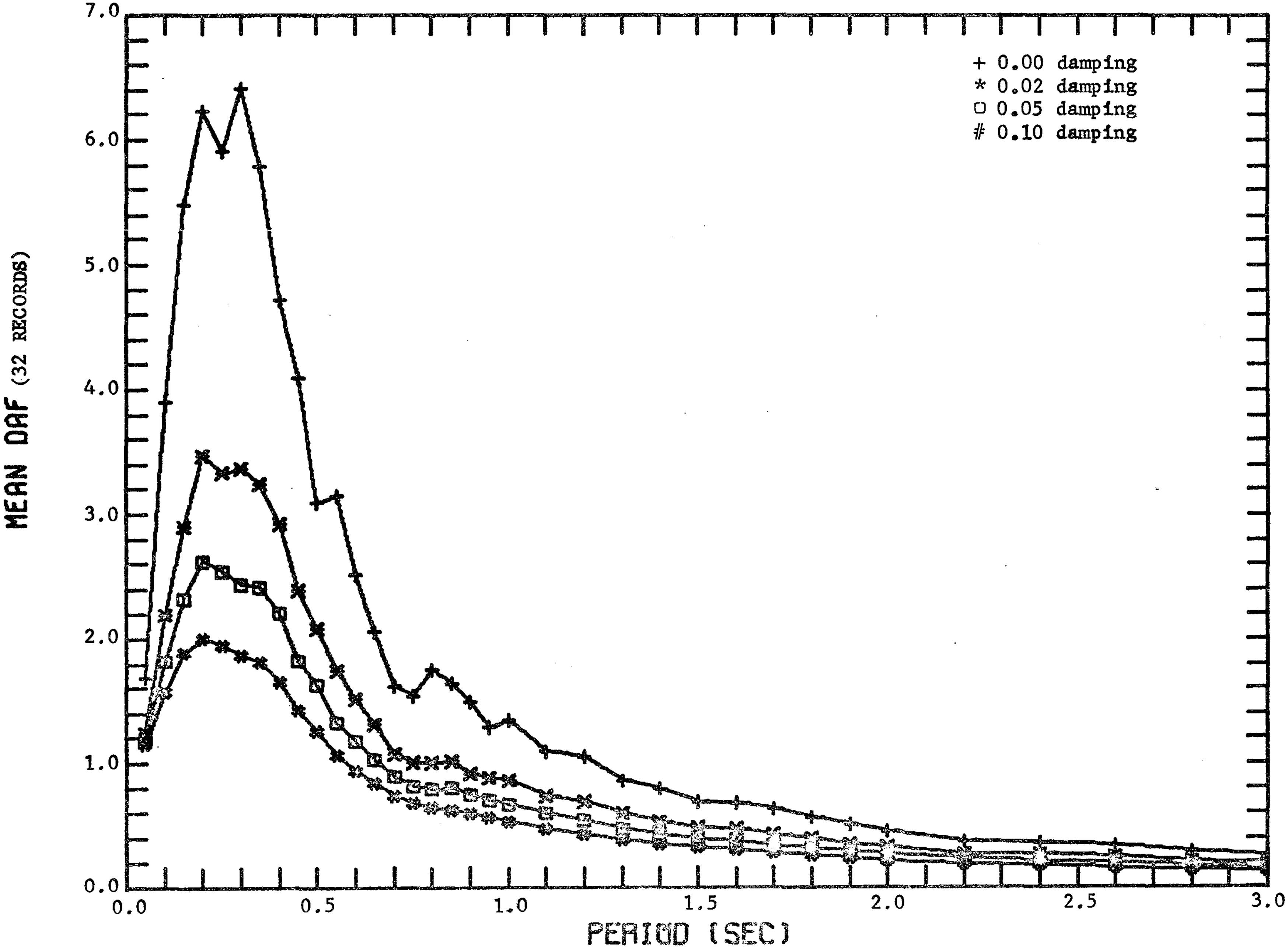
(2) Alternatively, recommended design shapes for different coefficients of variation shall be developed.

(3) Finally, recommended design shapes for different sites in Nicaragua shall be developed.

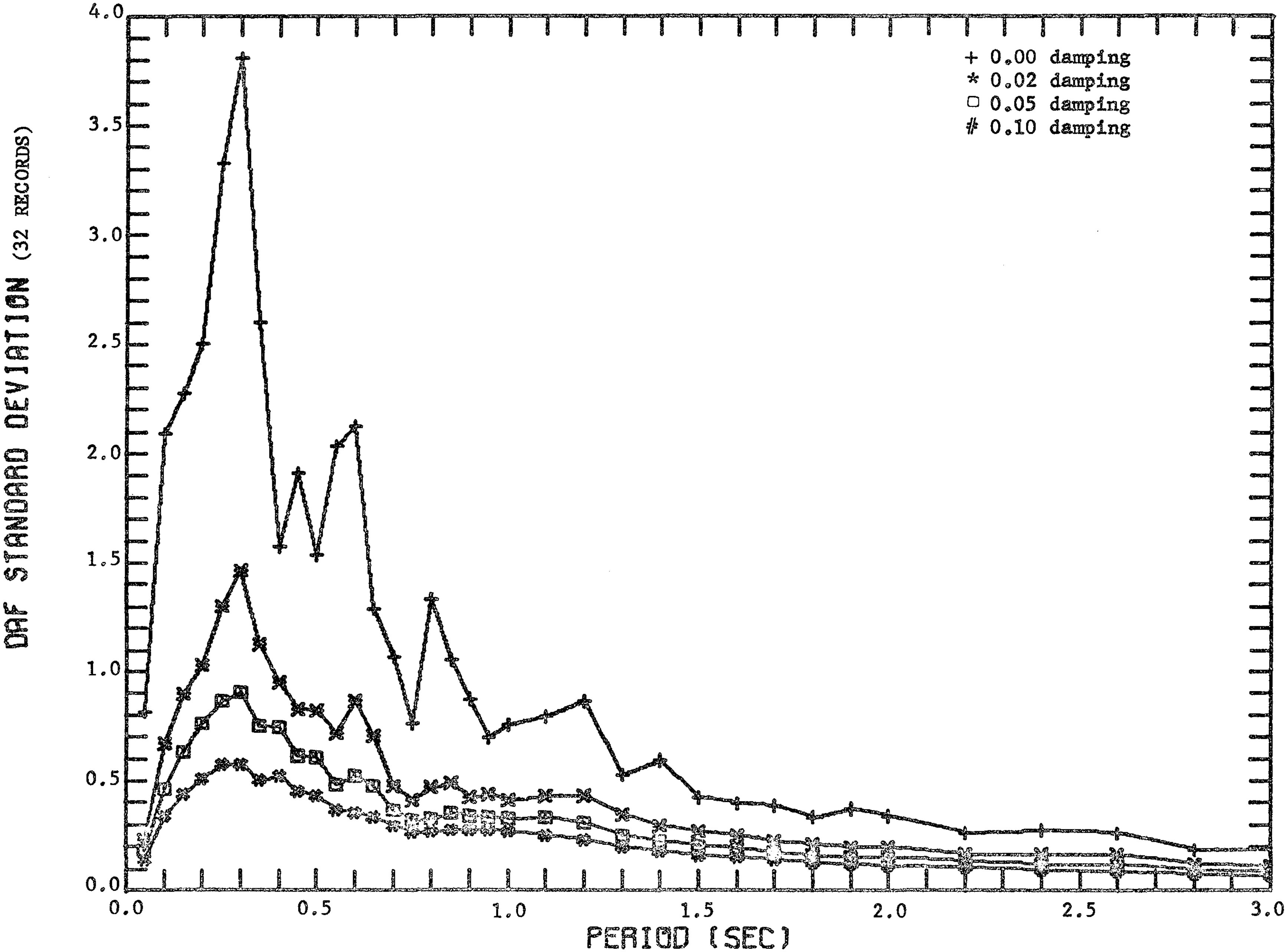
Table I

Data Base

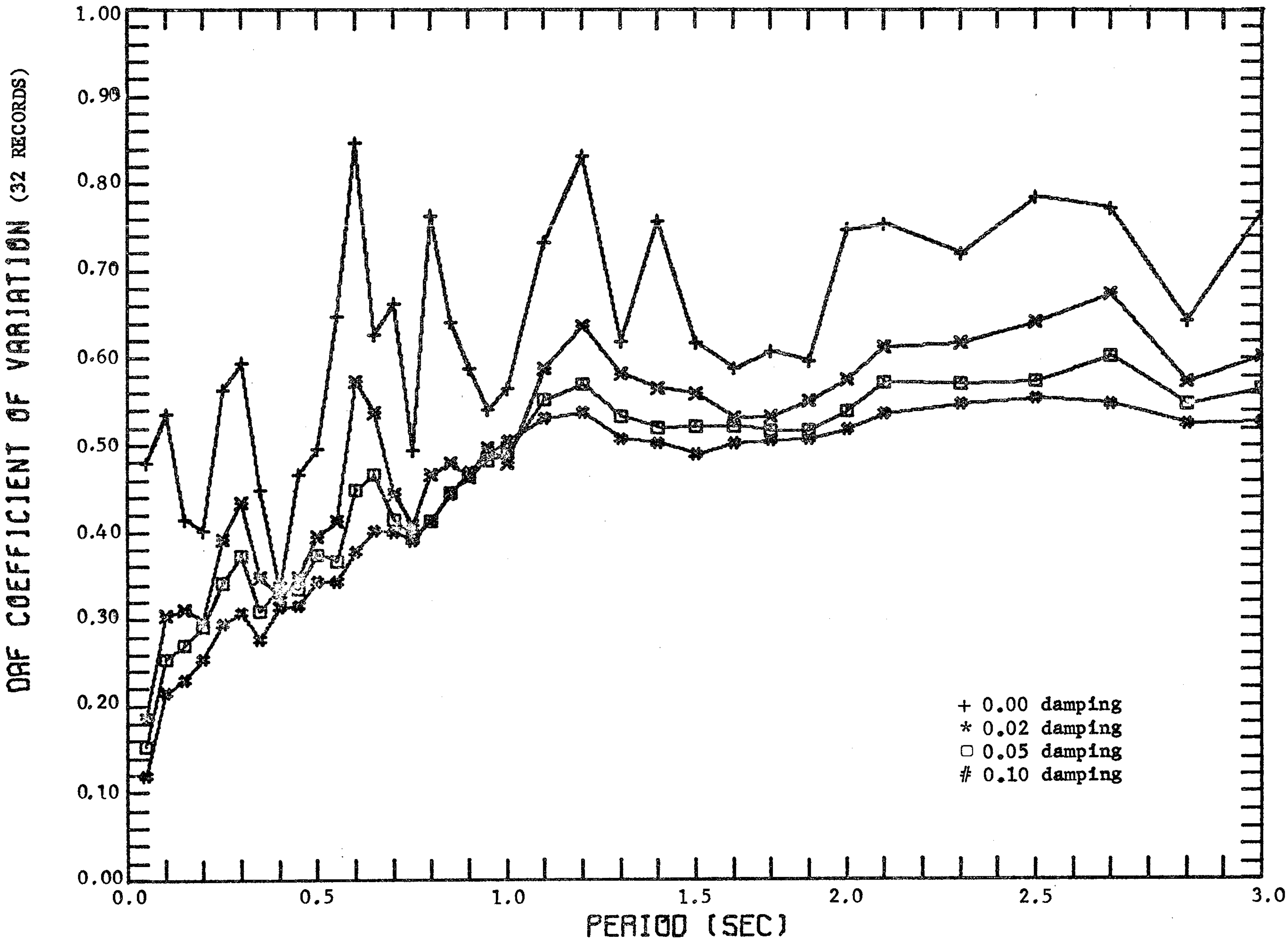
Record No.	Earthquake & Date	Recording Station	Component
1	Lower Calif., 12/30/34	El Centro, Imperial Valley	N00E
2	"	"	N90E
3	Helena, Montana, 10/31/35	Helena, Montana Carroll College	N00E
4	"	"	N90E
5	Western Washington, 4/13/49	Olympia, Wash. Hwy. Test Lab	S04E
6	"	"	S86W
7	Wheeler Ridge, Cal., 1/12/54	Taft Lincoln School Tunnel	N21E
8	"	"	S69E
9	Parkfield, Calif., 6/27/66	Cholame, Shandon, Array No. 5	N05W
10	"	"	N85E
11	"	Cholame, Shandon, Array No. 8	N50E
12	"	"	N40W
13	Borrego Mountain, 4/8/68	San Onofre SCE Power Plant	N33E
14	"	"	N57W
15	San Fernando, 2/9/71	Pacoima Dam, California	S14W
16	"	"	N76W
17	"	Edison Co., Colton, Calif.	S00W
18	"	"	N90E
19	"	Pumping Plant, Pearblossom, Cal.	N00E
20	"	"	N90W
21	"	Lake Hughes, Array Stn. 12, Cal.	N21E
22	"	"	N69W
23	"	Carbon Canyon Dam, Calif.	S50E
24	"	"	S40W
25	Managua, Nicaragua, 12/23/72, 06:29:42.5 GCT	ESSO Refinery	SOUTH
26	"	"	EAST
27	Managua, Nicaragua, 12/23/72, 07:19:40.0 GCT	"	SOUTH
28	"	"	EAST
29	Managua, Nicaragua, 1/4/68, 10:03:56.5 GCT	Banco Central de Nicaragua	N84.5W
30	"	"	S05.5W
31	Managua, Nicaragua, 3/31/73, 20:13 GMT	Universidad Nacional Autonoma de Nicaragua	N-S
32	"	"	E-W



+ 0.00 damping
* 0.02 damping
□ 0.05 damping
0.10 damping
MEAN DAF (32 RECORDS)
PERIOD (SEC)
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
0.5
1.5
2.5

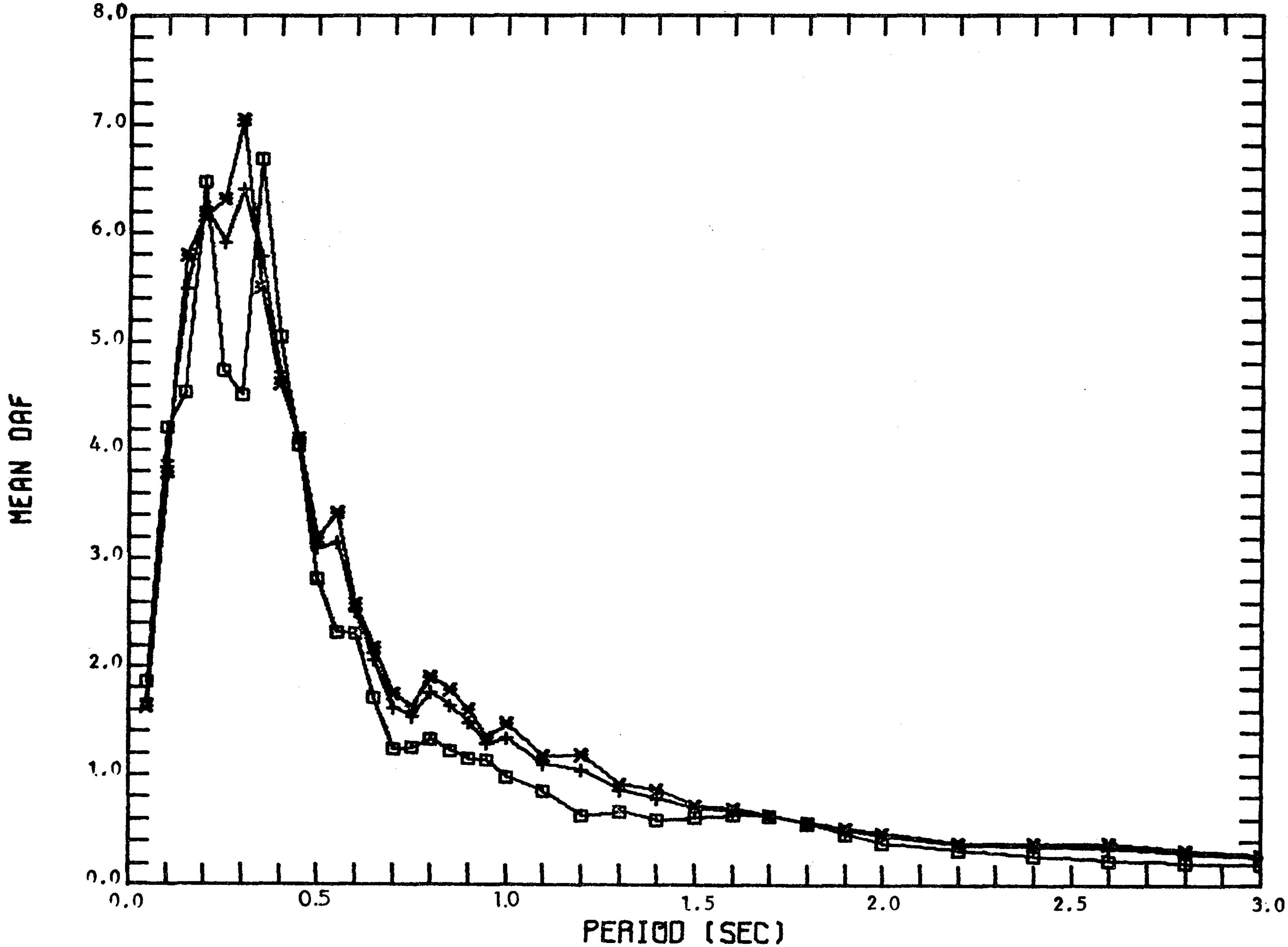


+ 0.00 damping
* 0.02 damping
□ 0.05 damping
0.10 damping
DAF STANDARD DEVIATION (32 RECORDS)
PERIOD (SEC)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

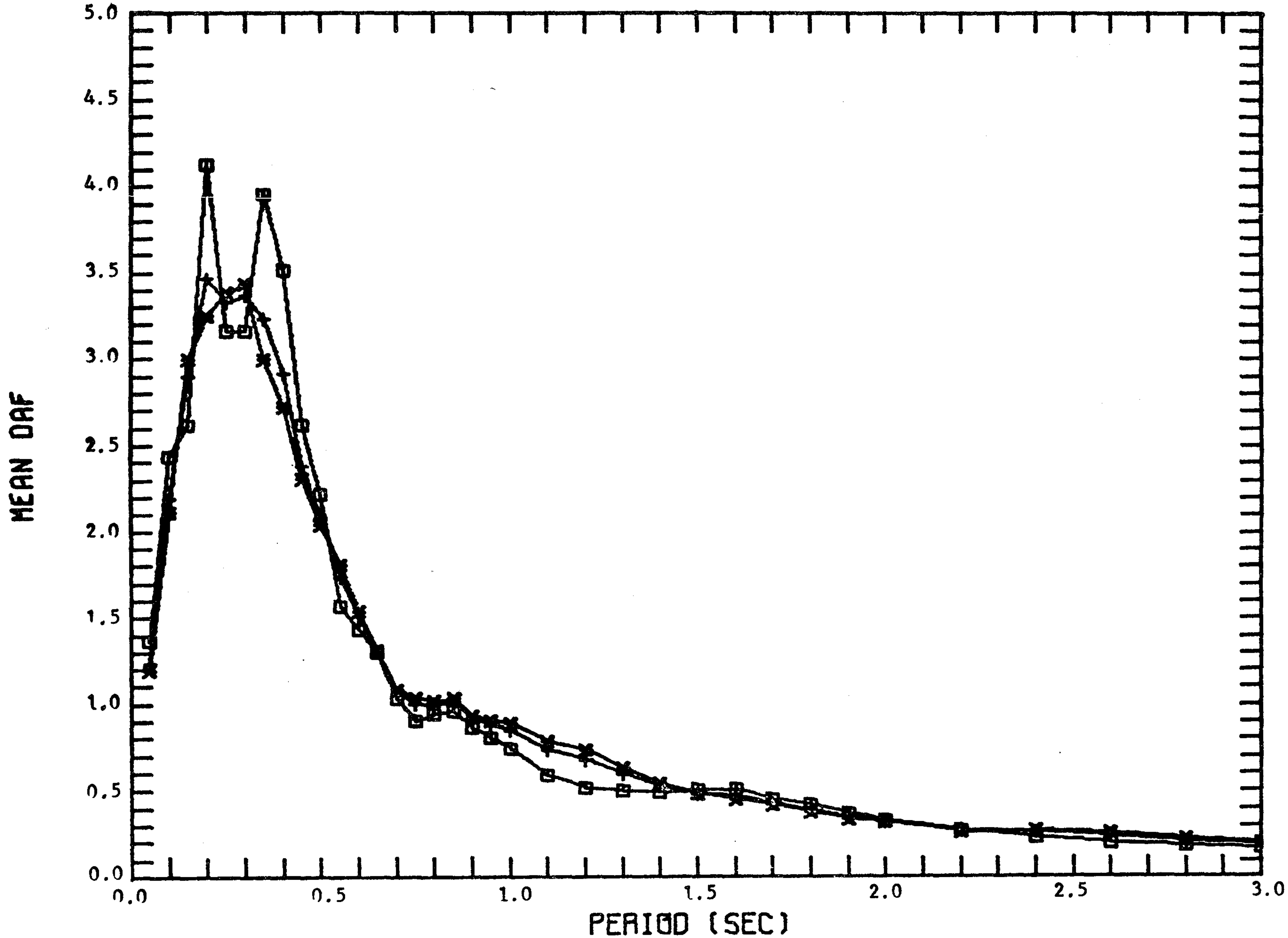


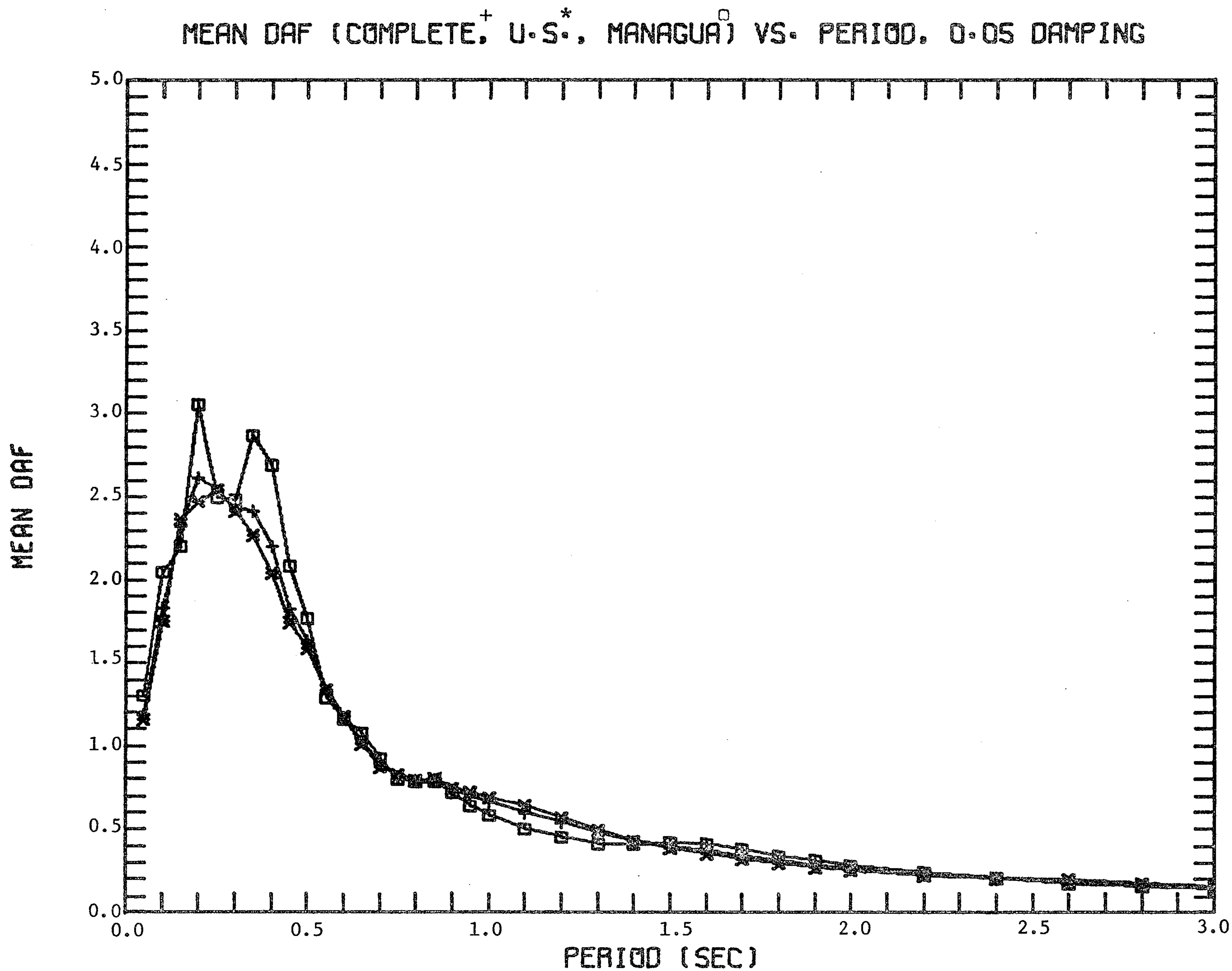
DAF COEFFICIENT OF VARIATION (32 RECORDS)
PERIOD (SEC)
+ 0.00 damping
* 0.02 damping
□ 0.05 damping
0.10 damping

MEAN DAF (COMPLETE^{+}, U.S.*, MANAGUA$^{\square}$) VS. PERIOD, ZERO DAMPING



MEAN DAF (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.02 DAMPING





MEAN DAF (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.05 DAMPING
MEAN DAF
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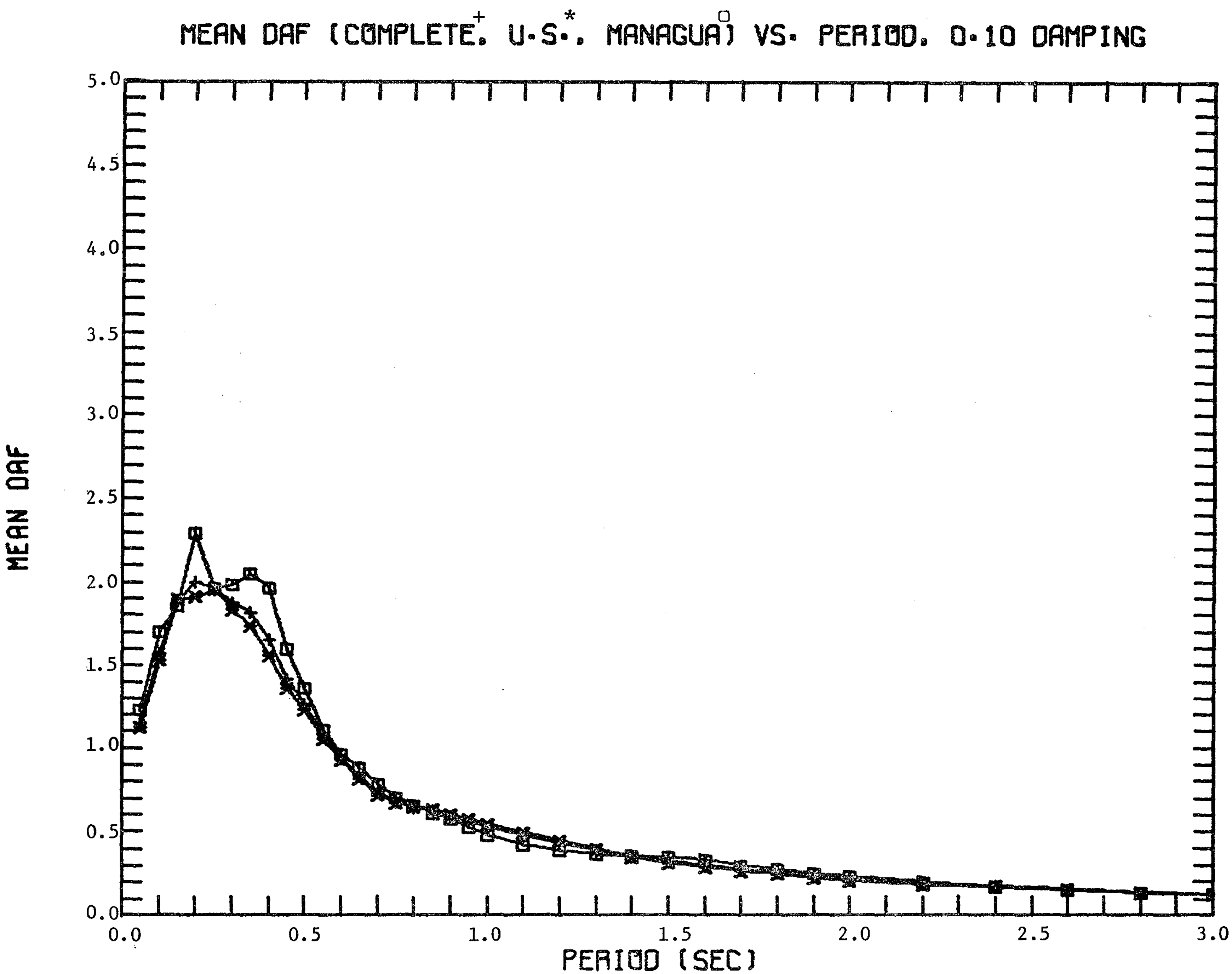
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
PERIOD (SEC)


MEAN DAF (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.10 DAMPING
MEAN DAF
PERIOD (SEC)
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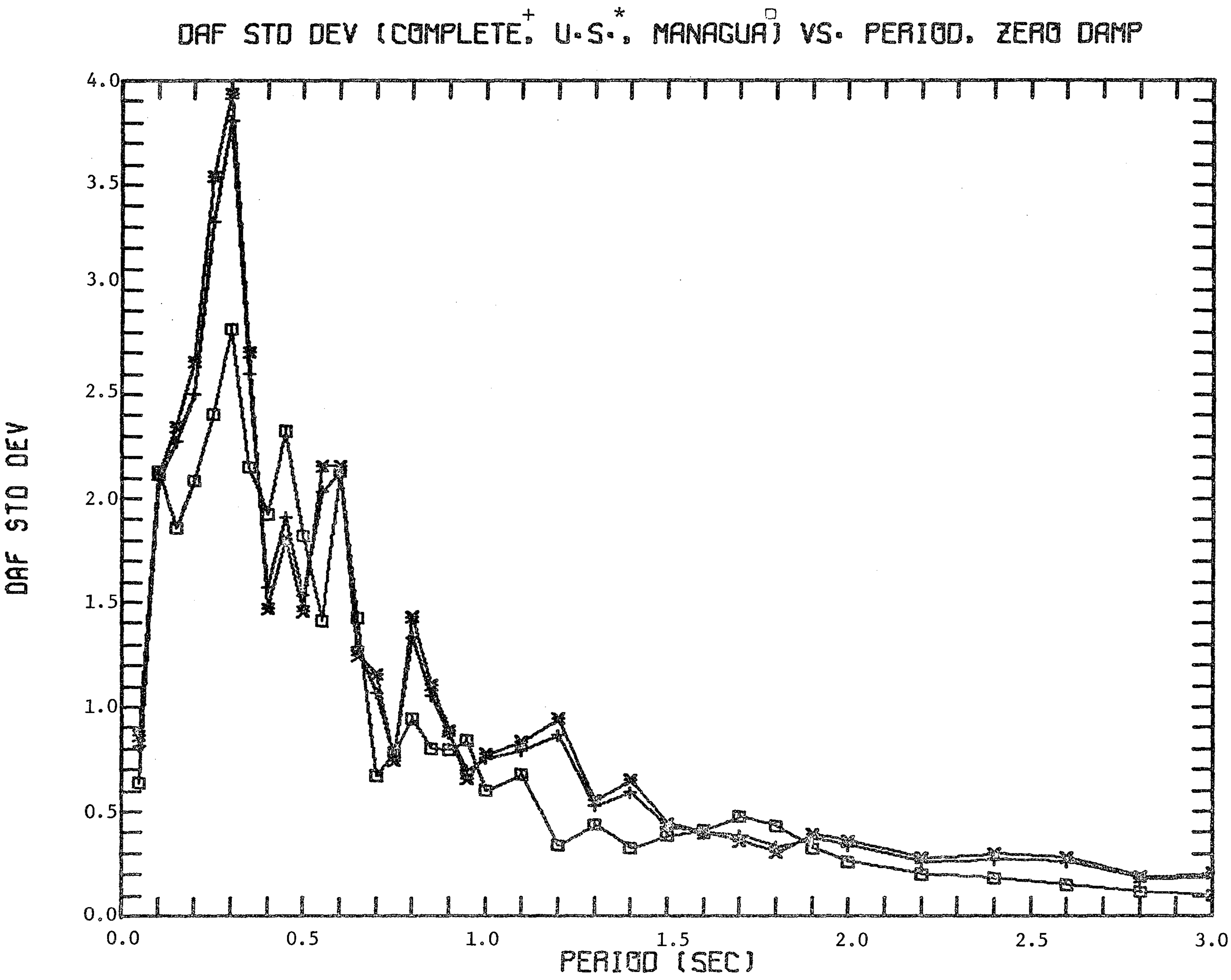
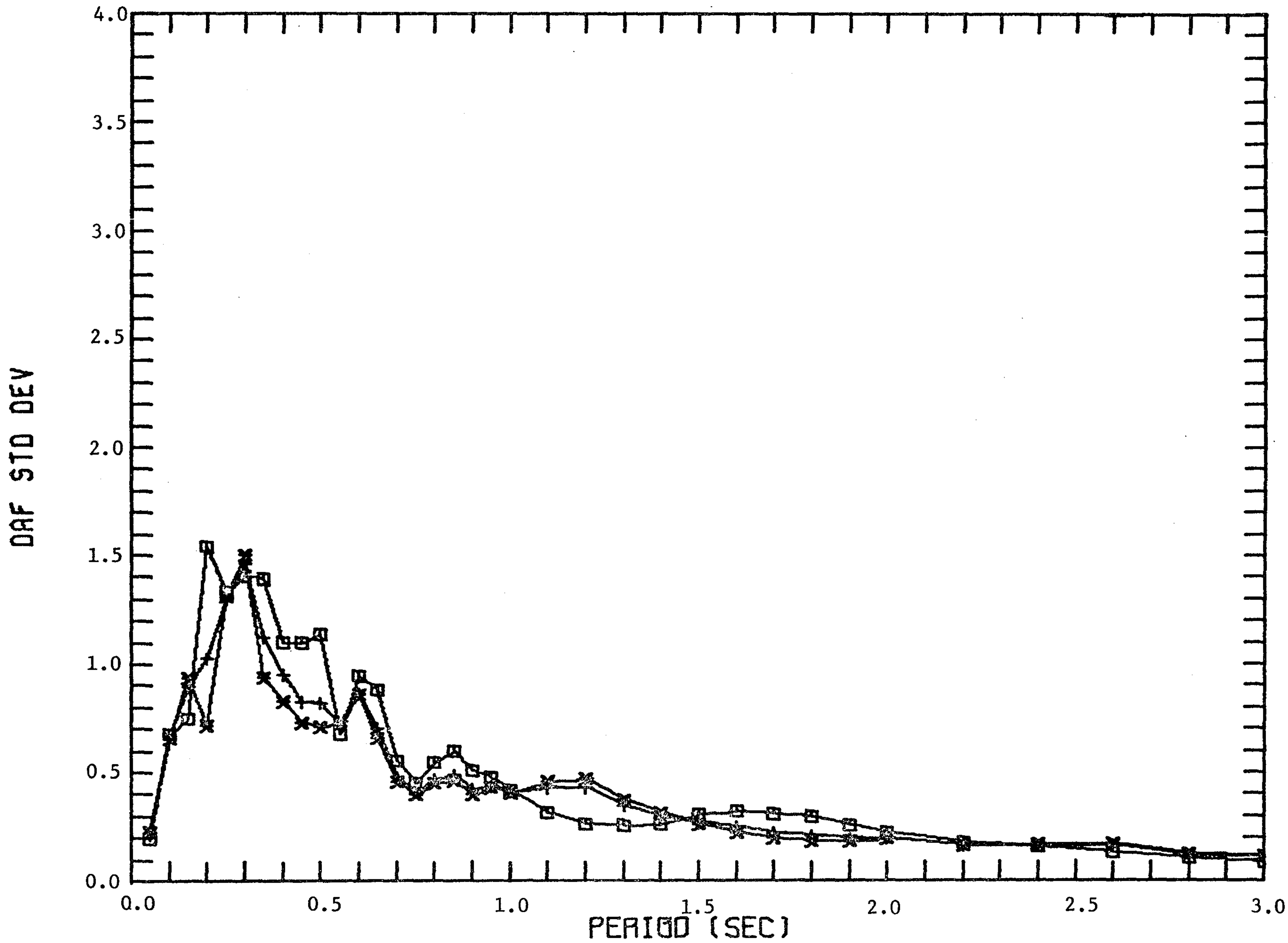
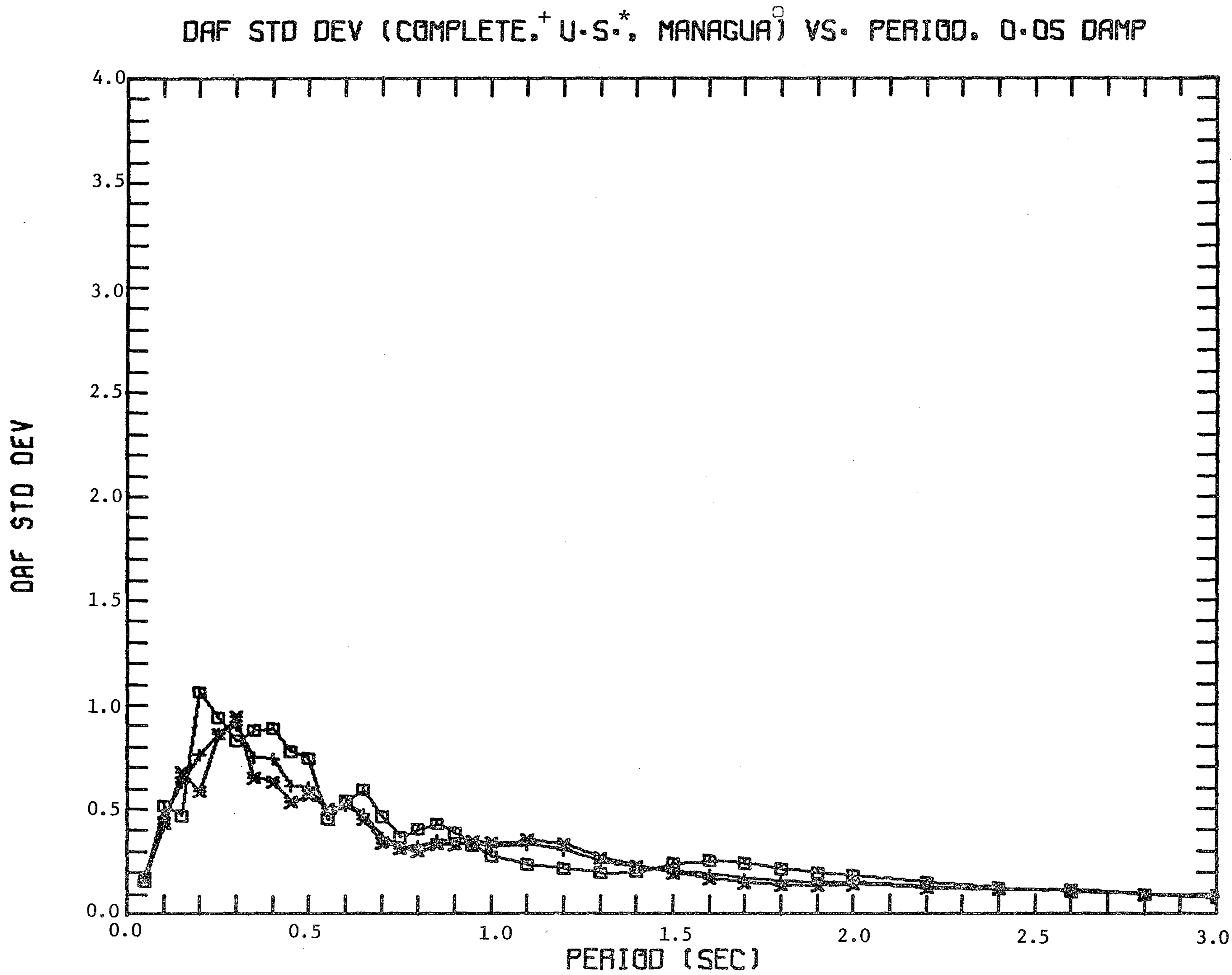
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0


DAF STD DEV (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, ZERO DAMP
DAF STD DEV
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.0
0.5
1.0
1.5
2.0
2.5
3.0
PERIOD (SEC)

DAF STD DEV (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.02 DAMP




DAF STD DEV (COMPLETE,+ U.S.*, MANAGUA□) VS. PERIOD, 0.05 DAMP
DAF STD DEV
PERIOD (SEC)
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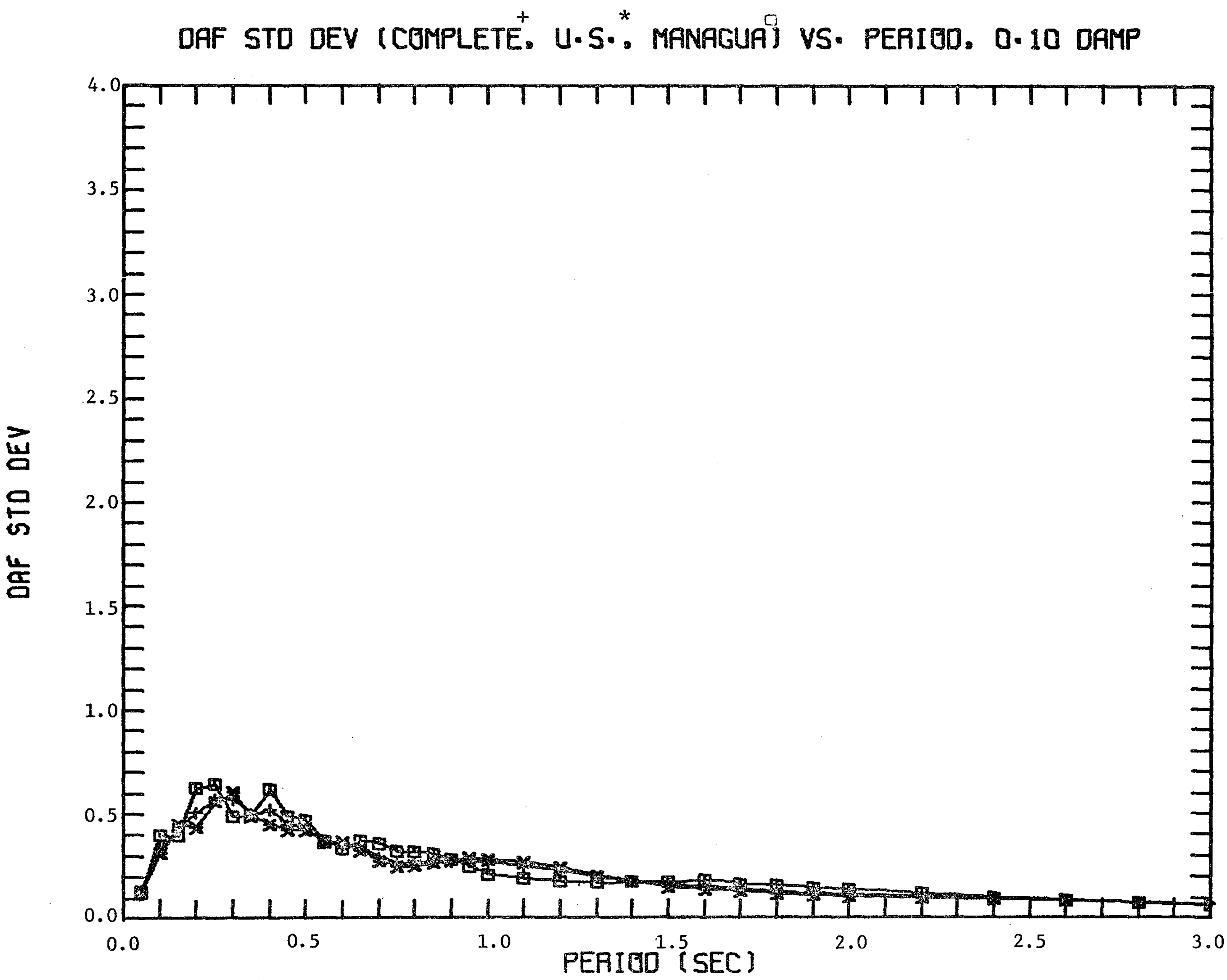
1.0
1.5
2.0
2.5
3.0
3.5
4.0



DAF STD DEV (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.10 DAMP
DAF STD DEV
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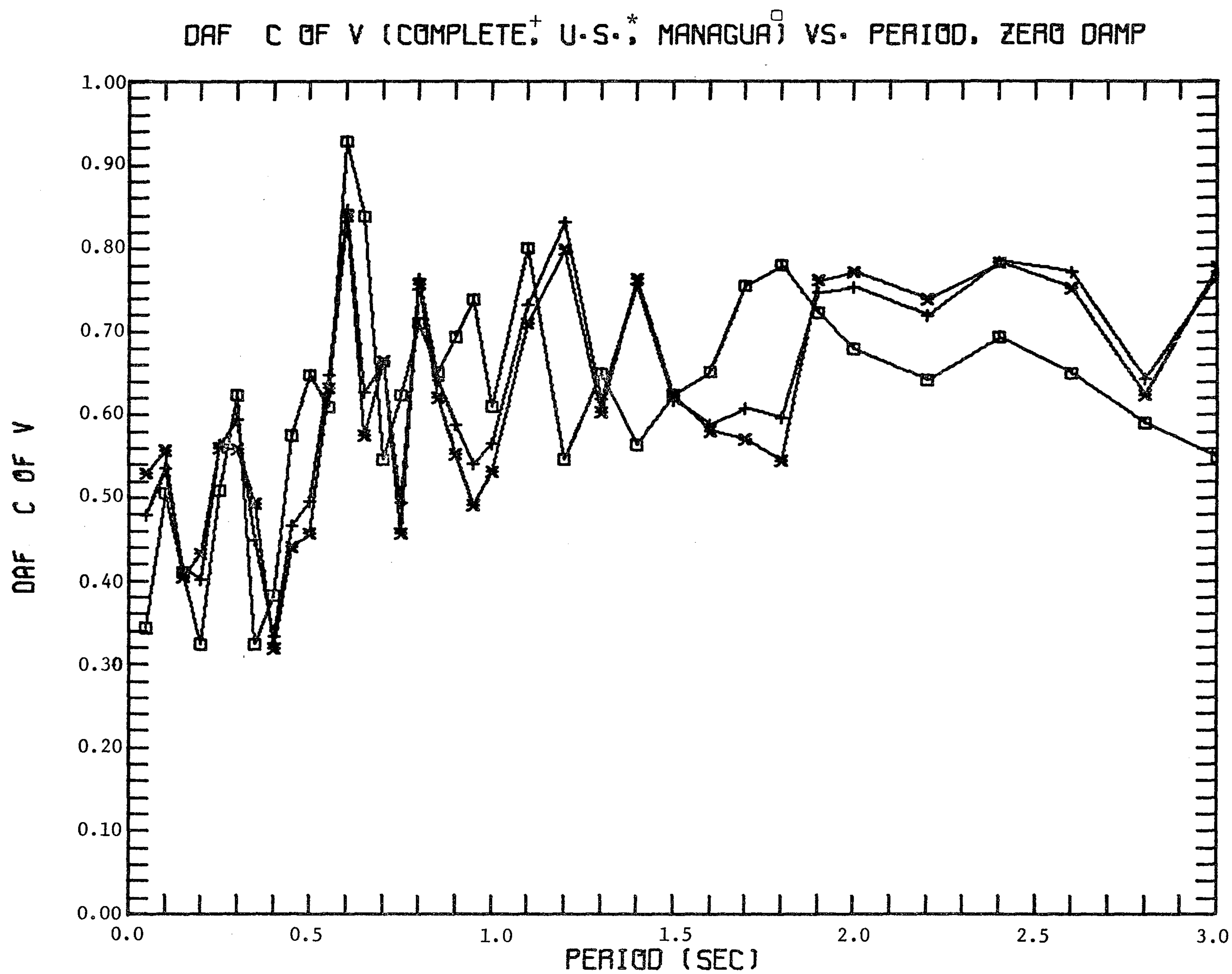
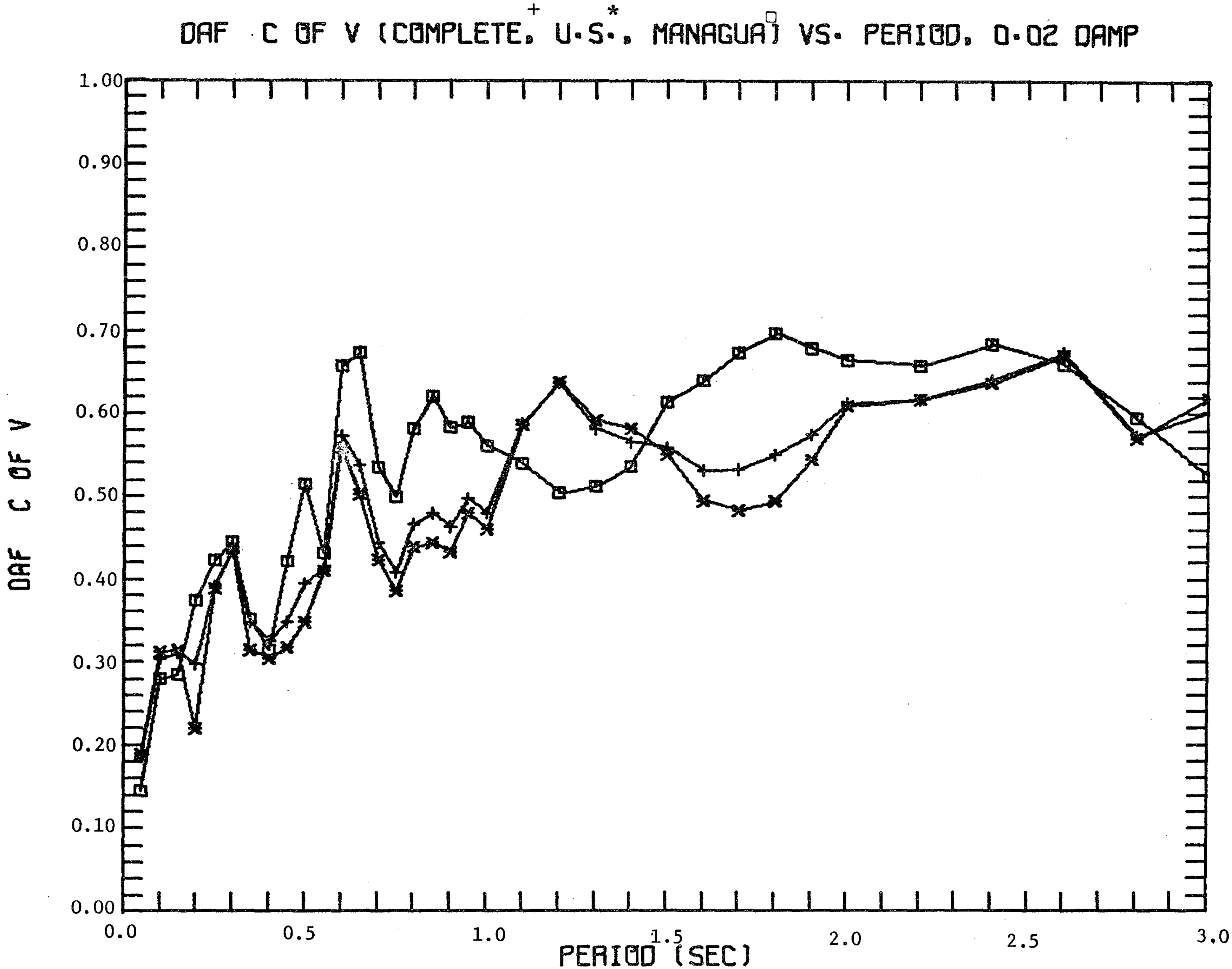
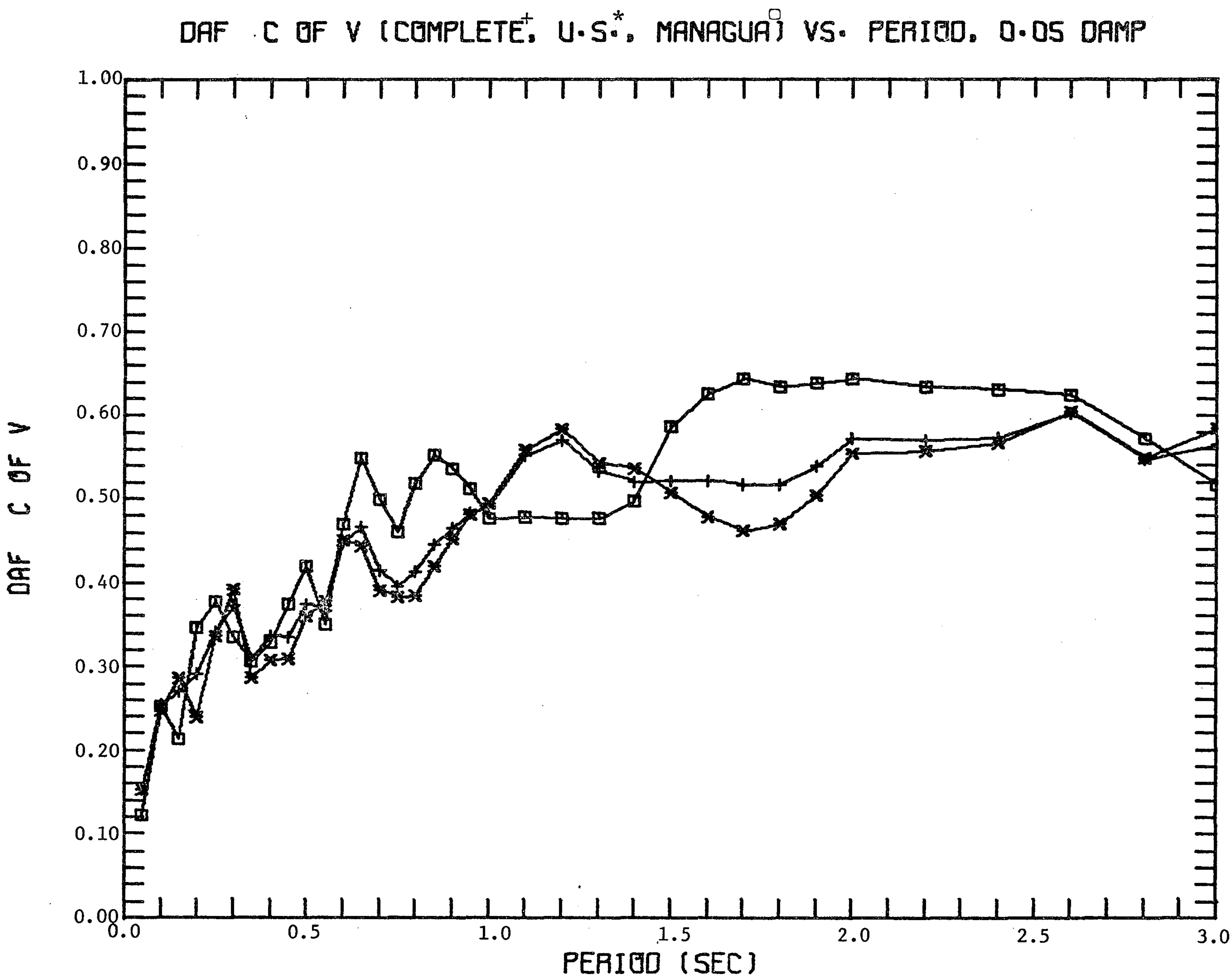
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.0
0.5
1.0
1.5
2.0
2.5
3.0
PERIOD (SEC)

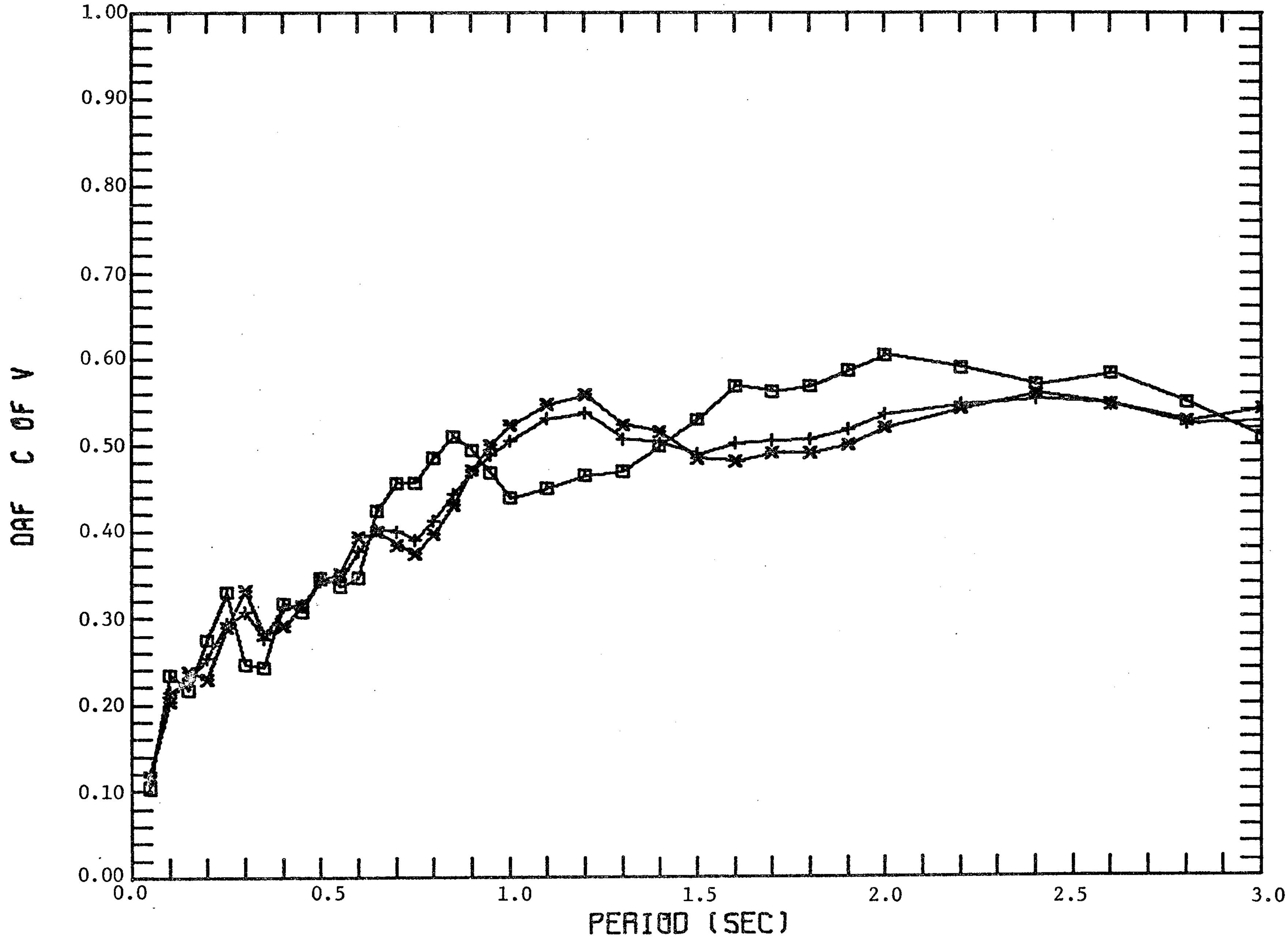

DAF C OF V (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, ZERO DAMP
DAF C OF V
PERIOD (SEC)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0


DAF C OF V (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.02 DAMP
DAF C OF V
PERIOD (SEC)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0

A19


DAF C OF V (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.05 DAMP
DAF C OF V
PERIOD (SEC)
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0

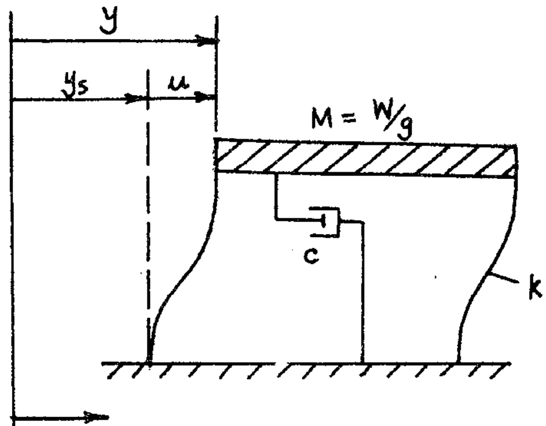
DAF C OF V (COMPLETE+, U.S.*, MANAGUA□) VS. PERIOD, 0.10 DAMP



APPENDIX B

BASICS OF ELASTIC DYNAMIC ANALYSIS

I. Single Degree of Freedom Response

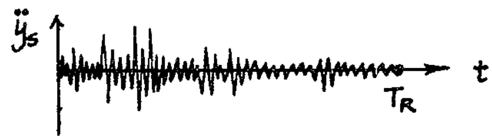


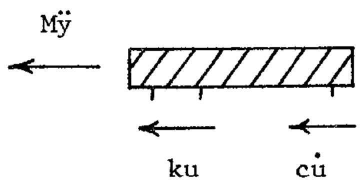
$u = y - y_s$

t in seconds
u in inches
Force in kips

$\ddot{y}_s = \ddot{y}_{so} f_a(t)$ = Earthquake Ground Acceleration Record

A. Dynamic Free Body





$\ddot{y} = \ddot{u} + \ddot{y}_s$

$M\ddot{y} + c\dot{u} + ku = 0$

Gives $\ddot{u} + \omega^2 u + 2\beta\dot{u} = -\ddot{y}_{so} f_a(t)$

$\omega = \sqrt{\frac{k}{M}}$ = natural circular frequency, rad/sec

$\beta = \frac{c}{2M}$; critical damping = $c_{cr} = 2M\omega$

common structural value of $\beta = 0.1\omega$ which corresponds to 10% of c_{cr}.

Period $T = \frac{1}{f} = \frac{2\pi}{\omega}$, seconds

$f = \omega/2\pi$, cycles per sec

B. Time Domain Analysis

Given Earthquake Ground Acceleration Record $\ddot{y}_{so} f_a(t) = \ddot{y}_s$

Differential Equation Solution is by Duhamel Integral Superposition of Impulses.

$$u(t) = \frac{-\ddot{y}_{so}}{\omega^2} \cdot \omega\int_o^t f_a(\tau)e^{-\beta(t-\tau)} \sin \omega(t-\tau)d$$

Gives displacement u during entire earthquake.

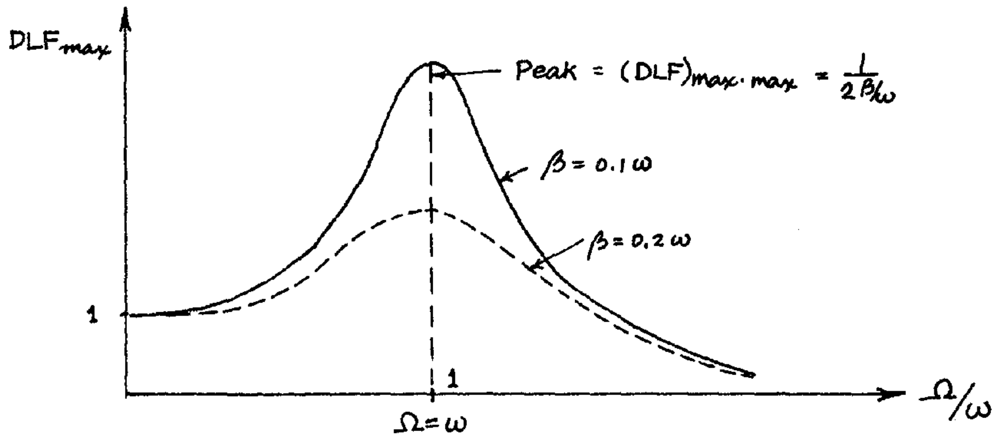
Note that the relative displacement spectrum value S_d at ω if the value of $u(t)_{max}$ found during $0 \leq t \leq T_R$ duration of the earthquake.

C. The Frequency Response Function: For the special case of $\ddot{y}_s = (1)\sin\Omega t$, here $\ddot{y}_{so} = 1$, $f_a(t) = \sin\Omega t$, for a long T_R duration.

Ω = Excitation frequency in rad/sec.

For any Ω, the response maximum is $u(t)_{max} = \frac{\ddot{y}_{so}}{\omega 2} (DLF)_{max}$ where

$$\ddot{y}_{so} = 1,\ (DLF)_{max} = \frac{1}{[(1 - \Omega^2/\omega^2)^2 + 4(\beta\Omega/\omega^2)^2]^{1/2}}$$ = Freq. Response Function.



For this special case of harmonic (sinusoidal) input, the output u(t) is also harmonic, and the peak response value $u_{max-max}$ occurs when $\Omega = \omega$.

$$u_{max-max} = \frac{1}{\omega^2}\left[\frac{1}{[4(\beta/\omega)^2]^{1/2}}\right] = \frac{1}{\omega^2} \cdot \frac{1}{2\beta/\omega}$$

From this very idealized special case we can see the effect of a possible predominant amplitude sine wave in the earthquake record.

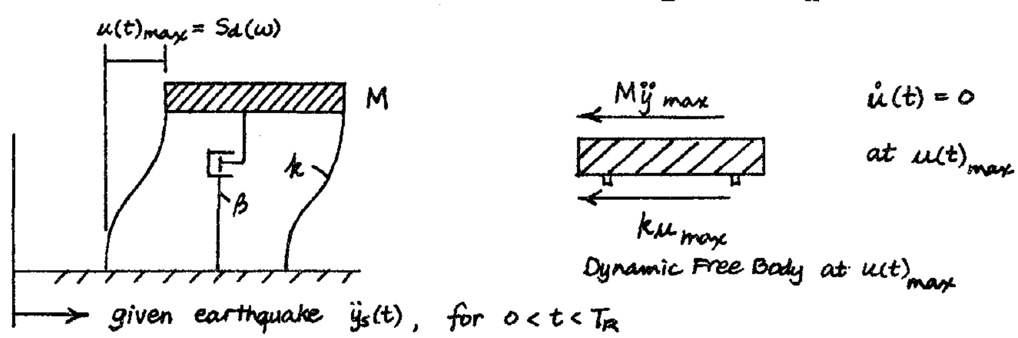
If we could model the earthquake as

$$\ddot{y}_s = \sum_{i=1}^{m} A_i \sin[\Omega_i t + \phi_i] \qquad \phi = \text{phase angle}$$

Then u(t) would be high if say $A_{i=3}$ were to be large and $\Omega_{i=3}$ was near to the system natural frequency value of ω.

D. Response Spectrum Analysis

Given a specifc earthquake acceleration record, and given β, and a family of single Degree of Freedom (S.D.F.) systems with a range of natural frequencies of ω_1 to ω_n, the relative displacement response spectrum is defined as $S_d(\omega) = u(t)_{max}$, for system frequency values in the range of $\omega_1 < \omega < \omega_n$.



Since the velocity $\dot{u}(t) = 0$ at $u(t)_{max}$, the differential equation at $u(t)_{max}$ is

$$M\ddot{y}_{max} + ku_{max} = 0, \text{ where } \ddot{y} = \ddot{y}_s + u,$$

giving

$$\ddot{y}_{max} = -\frac{k}{M} u_{max} = -\omega^2 u_{max}$$

thus, the absolute maximum acceleration, which is the definition of the absolute acceleration spectrum, S_a

$$S_a = \ddot{y}_{max} = -\omega^2 u_{max} = \omega^2 S_d \quad \text{(neglect sign).}$$

The "spectral velocity" S_v is a close approximation of both u_{max} and y_{max}, and is computed as that value of $y = S_v$ that has a kinetic energy equal to the system strain energy at S_d

$$\frac{1}{2}\frac{W}{g} S_v^2 = \frac{1}{2} k \, S_d^2, \qquad \frac{k}{M} = \frac{k}{W/g} = \omega^2$$

$$S_v^2 = \omega^2 S_d^2$$

$$S_v = \omega S_d \text{ approximates } \dot{u}_{max} \text{ and } \dot{y}_{max}.$$

Note also since $kS_d = MS_a = \frac{W}{g} S_a$

$$\frac{1}{2}\frac{W}{g} S_v^2 = \frac{1}{2} \cdot \frac{1}{k}\left(\frac{W}{g} S_a\right)^2$$

$$S_v^2 = \left(\frac{W/g}{k}\right) S_a^2 = \frac{S_a^2}{\omega^2}$$

$$S_v = \frac{S_a}{\omega}$$

Summary:

$$S_a = \omega^2 \cdot S_d = \omega S_v$$

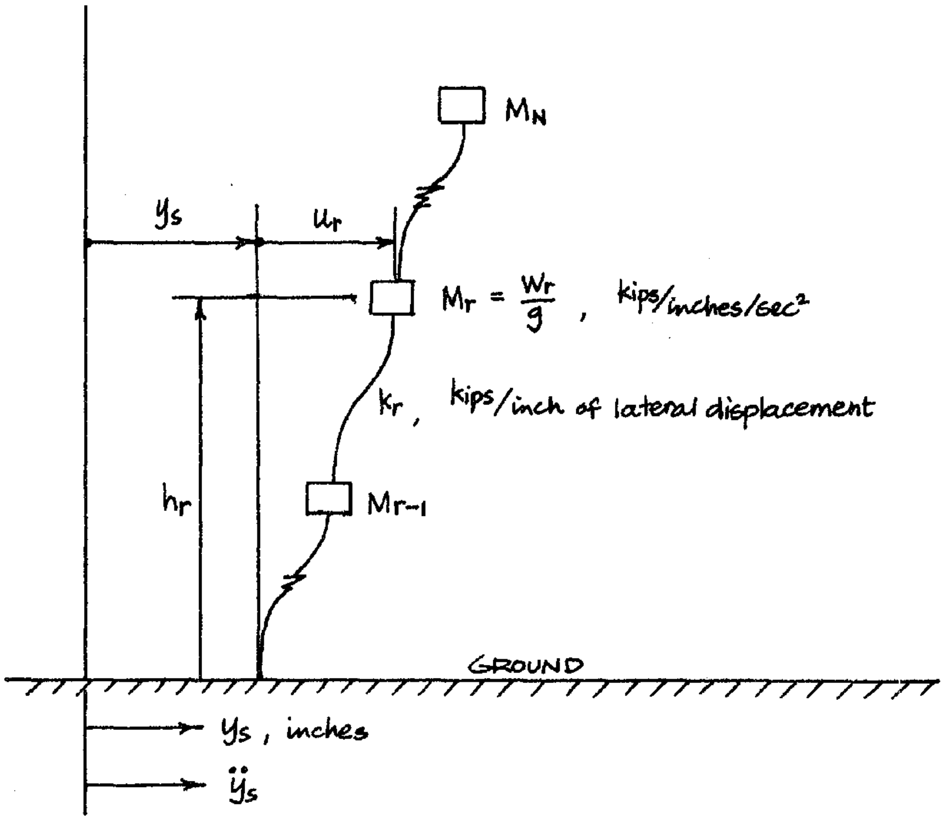
$$S_v = \omega \cdot S_d = {}^{S_a}/\omega$$

$$S_d = {}^{S_v}/\omega = {}^{S_a}/\omega^2$$

II. Multi-Degree-of-Freedom Response

Reference: Biggs, Introduction to Structural Dynamics, McGraw-Hill. (Sections 3.7 and 6.2)

Shear Building Model



Given the results of an elastic modal analysis of the dynamic undamped free vibration (where the results are also valid for light damping) of an N-floor shear building, for each of the m = 1, 2, ..., N modes of vibration configuration:

ω_m = natural modal frequency in rads/sec

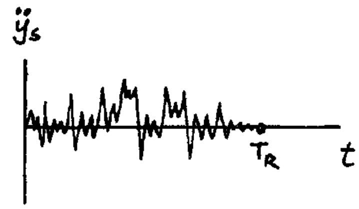
ϕ_{rm} = characteristic shape coordinate at floor mass "r" for mode "m", r = 1, 2, ..., N.

β_m = damping in mode "m", about 0.05 to $0.10\omega_m$

For any mode "m", the measure of its participation in the total response is given by the "Participation Factor"

$$\Gamma_m = \frac{\sum_{r=1}^{N} M_r \phi_{rm}}{\sum_{r=1}^{N} M_r \phi_{rm}^2}, \quad \text{for } m = 1, 2, \ldots, N$$

A. <u>Time Domain Analysis</u>



Given a specific earthquake acceleration record

$\ddot{y}_s = \ddot{y}_{so} f_a(t)$ for $0 \leq t \leq T_r$

The relative displacement response is at any t, at floor Mass "r",

$$u_r(t) = \sum_{\text{mode } m=1}^{N} \Gamma_m \cdot u_m^o(t) \cdot \phi_{rm}$$

where

$$u_m^o(t) = \frac{-\ddot{y}_{so}}{\omega_m^2} \cdot \omega_n \int_o^t f_a(\tau) e^{-\beta_m(t-\tau)} \cdot \sin[\omega_m(t-\tau)]d\tau$$

which is the response of a single degree of freedom system with natural frequency ω_m, and damping β_m.

Therefore, we see that $u_r(t)$ is the superposition of single-degree-of-freedom responses as modified by the Γ_m, ϕ_{rm} values.

If we examine the m^{th} mode term of $u_r(t)$

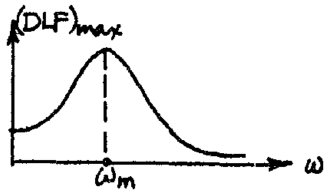
$$u_{rm}(t) = \Gamma_m \cdot u_m^o(t) \cdot \phi_{rm},$$

Γ_m amplifies the S.D.F. response u_m^o according to the modal participation of the m^{th} mode.

ϕ_{rm} adapts the $u_m^o(t)$ value to the shape of the m^{th} mode at the r^{th} floor position.

and

$u_m^o(t)$, as it is characterized by its frequency response function $(DLF)_{max}$ centered or peaked at ω_m,



absorbs and contributes response due to excitation sinusoids in $\ddot{y}_s$ which are near to its natural frequency ω_m.

B. <u>Response Spectrum Analysis</u>

If we examine the m^{th} component u_{rm} of

$$u_r = \sum_{m=1}^{N} u_{rm}$$

$$u_{rm}(t) = \Gamma_m \cdot u_m^o(t) \cdot \phi_{rm}$$

then the relative displacement spectrum component is

$$S_{drm}(\omega_m) = \Gamma_m \cdot S_d(\omega_m) \cdot \phi_m$$

where $S_d(\omega_m)$ is the single-degree-of-freedom spectrum value at $_m$. Also, since Γ_m and ϕ_m are constants for given r and m, the velocity spectrum component is

$$S_{vrm} = \omega_m S_{drm} = \omega_m \cdot \Gamma_m \cdot S_d(\omega_m) \cdot \phi_{rm}$$

and the acceleration spectrum component is

$$S_{arm} = \omega_m^2 S_{drm} = \omega_m^2 \cdot \Gamma_m \cdot S_d(\omega_m) \cdot \phi_{rm}$$

However, since all of the modes m=1 to N are not all in phase, and therefore <u>do not</u> reach maximum values S_{drm}, S_{vrm}, S_{arm} at the same time for all modes, and also all modal frequencies ω_m are different; we should not super-impose the individual mode spectrum values to find the total spectrum value at the r^{th} story mass. That is, the maximum value of u_r is not given by $\sum_{m=1}^{N} S_{drm}$.

It is better, therefore, to estimate or approximate u_{rmax} by a square-root-of-sum of squared values (SRSS).

$$u_{rmax} \simeq \sqrt{\sum_{m=1}^{N} \Gamma_m^2 \cdot S_d^2(\omega_m) \cdot \phi_{rm}^2}$$

and similarly

$$\dot{y}_{rmax} \simeq \sqrt{\sum_{m=1}^{N} \Gamma_m^2 \cdot S_v^2(\omega_m) \cdot \phi_m^2}$$

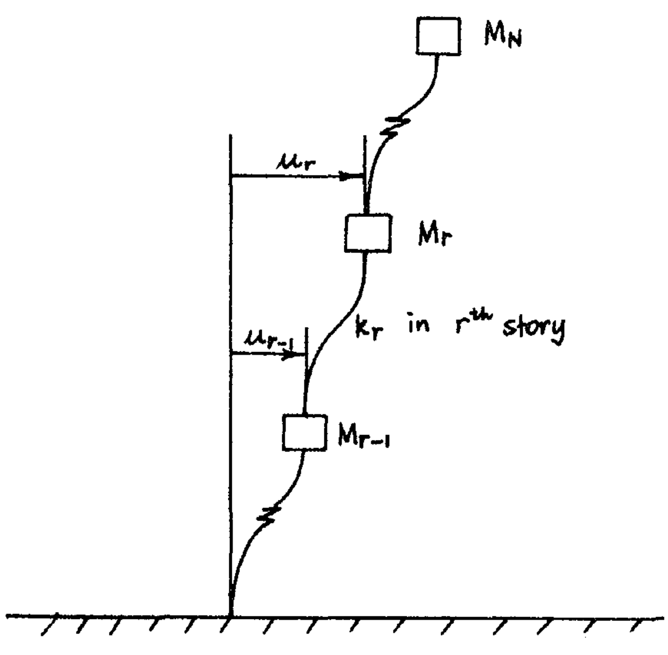
where

$$S_v(\omega_m) = \omega_m S_d(\omega_m)$$

$$\ddot{y}_{rmax} \simeq \sqrt{\sum_{m=1}^{N} \Gamma_m^2 \quad S_a^2(\omega_m) \quad \phi_{rm}^2}$$

where $S_a(\omega_m) = \omega_m^2 S_d(\omega_m)$, and recall that $S_d(\omega_m)$ is spectrum value at ω_m.

C. Column Shear in r^{th} Story



Since maximum accelerations $\ddot{y}_{rmax}$ do not occur simultaneously, the column shear V_r in the r^{th} story (between M_{r-1} and M_r) is estimated by

$$V_r \simeq (u_r - u_{r-1})_{max} \cdot k_r$$

where

$$u_r = \Sigma\Gamma_m \, u_m^o(t) \, \phi_{rm}$$

$$u_{r-1} = \Sigma\Gamma_m \, u_m^o(t) \, \phi_{r-1,m}$$

$$(u_r - u_{r-1}) = \Sigma\Gamma_m \, u_m^o(t) \quad [\phi_{rm} - \phi_{r-1,m}]$$

$$(u_r - u_{r-1})_{max} = \sqrt{\sum_{m=1}^{N} \Gamma_m^2 \cdot S_d^2(\omega_m) \cdot [\phi_{rm} - \phi_{r-1,m}]^2}$$

APPENDIX C

PLANNING MATRIX

This planning matrix is taken from Table 1 of Reference 12.

PLANNING MATRIX

GUIDE TO MINIMIZE THE RISKS OF SURFACE FAULTS

USES	KNOWN ACTIVE FAULTS		PROBABLE ACTIVE FAULTS	DOUBTFUL AREAS*	NO EVIDENCE
	MAJOR FAULT RED	MINOR FAULT ORANGE	MAJOR TRACE BLUE	MINOR TRACE GREEN	WHITE
1. Hospitals, electric power stations, water plants and pumping stations, fire departments, medicine and drug centers, overpass roadways and buildings with more than eight stories which height is at least 1.5 times larger than minimum plan dimension	Exclude	Exclude	Exclude	N	N
1A Underground public utilities, fire mains, main sewer lines, electric conduits	Special Design	Special Design	Special Design	N	N
2. Schools, large hotels, churches, government centers, museums, theaters, auditoriums, ammunition storage	Exclude	Exclude	N	N	N
3. Housing developments, multi-family apartment houses, small hotels, office buildings, commercial buildings (all structures in this category less than three stories high)	Exclude	Exclude	N	N or Standard A	N or Standard A
4. Open markets, one-family homes, industrial buildings, parking buildings, repair shops, inhabited warehouses	Exclude	Standard A	N	Standard B	Standard B
5. Uninhabited warehouses, animal shelters, car shelters, parking lots, wood-frame houses, special construction with light roofs not for permanent habitation, light structures for bus terminals or pick-up points	Standard B	Standard B	Standard B	Standard B	Standard B

N: Local Fault Study Needed.

Standard A: Structures designed to resist the maximum surface fault displacement, tilting, or warping. Foundations are designed as a single unit.

Standard B: Comply with Building Code.

***Doubtful Areas: Faults may be located somewhere within the indicated area.**

APPENDIX D

DIFFERENCES WHICH AFFECT ANY COMPARISON BETWEEN NICARAGUA AND SEAOC OR UBC SEISMIC LOAD CRITERIA

DIFFERENCES WHICH AFFECT ANY COMPARISON BETWEEN NICARAGUA AND SEAOC OR UBC SEISMIC LOAD CRITERIA

Subscripts
(N = Nicaragua, S = SEAOC)

A consistent comparison of load criteria is difficult to generalize since each structure has its own particular load characteristics. However, in order to provide an approximate evaluation of the relative effects of the proposed criteria, the following study is given:

1) Seismic Weight

$W_N = W_D + 0.4W_L$

$W_S = W_D$

2) Equivalent Mass from Spectral Analysis - for Base Shear Comparison

$V_N = W_{equiv} S_a$

$W_{equiv} = 0.7W_N$ for 10-12 stories and above

$= 0.9W_N$ for short structures

$V_S = W_D$ (UKCS)

3) Load Factors for Ultimate Strength

$R_N = D + 0.4L + E_N$

$R_S = 1.4 (D + L + E_S)$

Extra SEAOC Factors

$R_S = 1.4 (D + L) + 2E_S$

$R_S = 1.4 (D + L) + 1.25 (1.4) E_S$
for X-Bracing

THE LOAD FACTOR EFFECT

The 1974 SEAOC recommendations prescribe load combinations for ultimate strength design as

$$RS = 1.4\ (D + L + E_S)$$

(here the subscript S is for SEAOC), where E_S is the seismic load due to base shear KCSW.

The Nicaragua Design Rules give

$$R_N = D + 0.4L + E_N$$

(the subscript N is for Nicaragua), where E_N is the seismic load due to the DTSF.

The effect of this different method of load combinations is to be studied for the case of Live Load L = 0.5D (D = Dead Load), and where E_S may be either 2D, 4D, or 6D.

E_S	=	2 D	4 D	6 D
$1.4(D + .5D + E_S)$	=	4.9D	7.7D	10.5D
Nica Vertical $D + .4L$	=	1.2D	1.2D	1.2D
Seismic Capacity	=	3.7D	6.5D	9.3D
SEAOC Demand $1.4E_S$	=	2.8D	5.6D	8.4D

Ratio of Capacity-to-Demand if Nica load factors were used for vertical load effects

$$\frac{3.7}{2.8} = 1.3 \qquad \frac{6.5}{5.6} = 1.2 \qquad \frac{9.3}{8.4} = 1.1$$

These ratios show that the SEAOC seismic load levels could be increased by about 1.1 to 1.2 if the Nica factors were used, and there would be no change in the resulting member strength requirements. Therefore, in order to compare the Nica and SEAOC seismic load levels on the basis of the Nica load combinations, either the SEAOC value should be increased by 1.1, or the Nica seismic load be decreased by $\frac{1}{1.1} = .9$.

Managua - Class B Structures

Equivalent Plateau Comparison at 0.86H*

Type		$.86H^*$ Shear	$.86H_{OT}$ Flexure	1974 SEA UKCS Shear U = 2	1974 SEA UKCS Flexure U = 1.4	1973 UBC UKC Shear U = 2.8	1973 UBC UKC Flexure U = 1.4
	A	.14	.14				
.67	B	.17	.17	---	.13	---	.09
	C	.20	.20				
	A	.20	.17				
0.80	B	.24	.20	.22	.16	.22	.11
	C	.27	.23				
	A	.25	.17				
1.00	B	.29	.20	.28	.20	.28	.14
	C	.34	.22				
	A	.34	.17				
1.33	B	.39	.20	.37	.26	.37	.19
	C	.45	.45				

For Class "A: Essential Facilities

Multiply H and H_{OT} by $\frac{0.45}{0.35} = 1.29$

and compare with I = 1.5 times the 1974 SEA Value

or with 3 times the 1973 UBC Value,

where this latter criterion represents the California State Hospital Requirements (the 3K Factor).

*Assuming $W_L = 0.5W_D$ then $W_N = 1.2W_D$

Using $W_N = 1.2W_D$

$W_{equiv} = 0.8W_N = 0.8(1.2)W_D$

Load Factor Advantage Effect = 0.9

Comparison Base Shear Coefficient = 0.8(1.2)(0.9)H = 0.86H

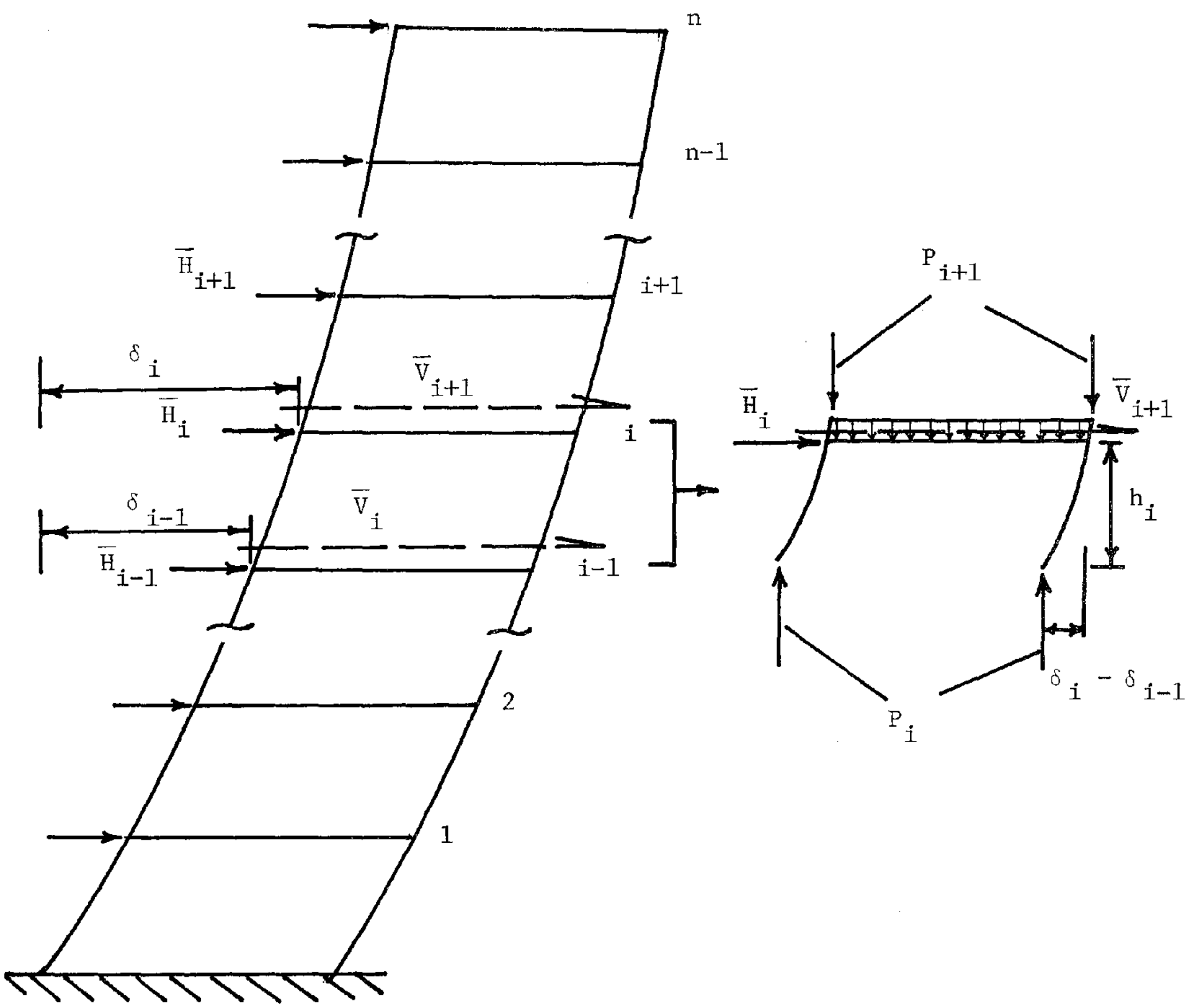
P-Delta Effect:

The strength design of the lateral load resisting elements of a structure is based on providing protection against excessive damage under an earthquake with probability of exceedance, P_D, during the economic life of the structure (damage threshold earthquake). The design criterion used for damage control is that under the actions of gravity loads and the loads caused by the design force spectrum (DFS), the internal forces in all elements of the lateral load resisting system shall be smaller than or in the worst case equal to the strength capacity of the elements. This renders it necessary, theoretically, to include all actions, primary and secondary, in the response calculations.

The one secondary effect that may be of importance under lateral loads, is the P-Delta effect. Most analysis procedures neglect this effect, and in most cases rightfully so, since it is negligibly small for most types of structural systems in the elastic range. Still, a simple method of estimating the P-Delta effect should be available to the designer to aid in the decision whether or not to include the additional member forces caused by this effect.

A good estimate of the additional member forces produced by the P-Delta effect can be obtained through replacing the moments due to P-Delta by equivalent story loads. This method is illustrated in Figure E-1. The equivalent story loads can be computed as

$$\bar{H}_i = \frac{P_i(\delta_i - \delta_{i-1})}{h_i} - \bar{V}_{i+1}$$



P-DELTA EFFECT

FIGURE E-1

where

P_i is the sum of the axial column loads below level i

δ_i is the lateral deflection at level i as computed from first order analysis

δ_{i-1} is the lateral deflection at level i-1 as computed from first order analysis

h_i is the height of story i

$$\bar{V}_{i+1} = \sum_{j=i+1}^{n} \bar{H}_j$$

The equivalent story shears representing the effects of P-Delta are then

$$\bar{V}_i = \sum_{j=i}^{n} \bar{H}_j = \frac{P_i(\delta_i - \delta_{i-1})}{h_i}$$

These story shears, $\bar{V}_i$, can now be compared to the story shears, V_i, produced by the design force spectrum (DFS), and their relative importance can be evaluated. When $\bar{V}_i$ values exceed a certain percentage of V_i, say 5 percent, the P-Delta effect should be included in the strength design of the structure. This can be done in an approximate manner through replacing V_i by $V_i + \bar{V}_i$, if it is intended to redesign the structure for an increased stiffness that will lead to the previously computed deflections under the increased lateral forces $V_i + \bar{V}_i$. If it is not intended to increase the stiffness of the structure, theory requires that V_i shall be replaced by

$$V_i \frac{1}{1 - \frac{\bar{V}_i}{\bar{V}_i}}$$

The above method is based on the approximation that the relative story displacement, Δ_i, including the P-Delta effect, is given by

$$\Delta_i = \frac{\delta_i - \delta_{i-1}}{1 - \frac{P_i(\delta_i - \delta_{i-1})}{V_i h_i}} = \frac{\delta_i - \delta_{i-1}}{1 - \frac{\overline{V}_i}{V_i}}$$

This equation also gives an approximation for the magnitude of the elastic critical axial load at the i-th story, i.e.

$$P_{cr,i} = \frac{V_i h_i}{\delta_i - \delta_{i-1}} = P_i \frac{V_i}{\overline{V}_i}$$

Hence, the ratio $V_i/\overline{V}_i$ represents the factor of safety against elastic frame instability at each story. For a desired factor of safety against frame instability, the ratio $V_i/\overline{V}_i$ indicates the adequacy of the design.

Alternative methods for evaluating elastic frame stability for unbraced and braced frames are presented in Reference II.

The P-Delta effect, as it relates to stability of the structure under the condemnation threshold earthquake, will be discussed later in this chapter.

Drift Control:

Damage control has to be concerned with structural as well as nonstructural damage. Protection against excessive structural damage is provided through specifying relatively small d_T values for the damage threshold earthquake. Implicitly, a certain amount of inelastic deformation is accepted under moderate earthquakes (damage threshold earthquake) since it is expected that the actual lateral deflections will be d_T times those computed from the design force spectrum. In accordance with the design philosophy, the damage associated with these inelastic deformations is believed to be repairable without major costs.

As far as nonstructural damage is concerned, the same criterion must hold true: the damage must be repairable without major cost. This will necessitate the specification of detailing criteria for "nonstructural" elements and limitations on lateral deflections.

"Nonstructural" elements in this context should include all elements that are not part of the lateral and vertical load resisting system, such as certain types of elevator shafts, staircases, floor systems, interior walls and partitions, exterior claddings, architectural elements, etc. Detailing requirements should be specified for all such elements such that damage does not become excessive under the below discussed drift limitations. Obviously, particular emphasis has to be placed on vital elements that need to remain functional after an earthquake, such as elevator shafts and staircases. Also, elements of life lines in structures, such as electricity and water supply may need special consideration.

The allowable story drift under the damage threshold earthquake will strongly depend on the above detailing requirements. Allowable story drift is generally expressed in terms of the story drift index, δ/h where δ is the relative lateral deflection between adjacent stories.

A better measure of nonstructural damage could be achieved through replacing the story drift index δ/h by a story shear distortion index since nonstructural damage is caused primarily through interstory shear distortions and not flexural deformations. However, the additional effort of computing a story shear distortion index may not be justifiable, since structures for which drift considerations become important are usually structures that deform primarily in shear type deformations. To follow generally accepted practice, the story drift index is therefore retained as a basic measure of nonstructural damage.

It is suggested to limit the story drift index under the damage deformation spectrum (DDS) to 0.01. Nonstructural elements can be detailed adequately to resist excessive damage at this drift index. As far as design for stiffness is concerned, the elastic story deflections under the DFS loads need then to be kept below $(0.01/d_T)h$ for each story.

It has to be pointed out that, if this stiffness criterion is adopted, the design of many moment resisting steel frames and some braced steel frames will be controlled by drift considerations and not strength.

One more point regarding drift needs to be emphasized. It is required that all elements of the vertical load carrying system (including those which are not part of the lateral load carrying system) must maintain their vertical load carrying capacity under the lateral deflections caused by the condemnation threshold earthquake. It is necessary, therefore, to assure this load carrying capacity under lateral deflections that are $(A_C/A_D)\ d_T$ times as large as those computed from the DFS. Important elements, for which this criterion has to be verified are, for example, columns in flat slab structures and other columns that are not designed in a ductile manner.

Protection Against Condemnation and Collapse

Explicit criteria for protection against condemnation and also collapse of the structure have not been formulated as yet. Condemnation is defined as the state of nonrepairable damage in vital structural elements that necessitates the replacement of the structure. Clearly, also at the condemnation state, a margin of safety against collapse must be provided. It is necessary, therefore, to formulate a set of design criteria which provides a desired margin of safety against collapse when the structure is subjected to a severe earthquake of the low probability of exceedance, P_C, during its economic life (condemnation threshold earthquake).

The causes of collapse in a structure can be as follows:

1. Improper detailing of connections that may lead to partial or complete failure at critical points and does not allow a redistribution of internal forces to other lateral load resisting elements. Such failure can in general be avoided by designing the less ductile components of connections (welds and bolts in steel structures, shear strength of beam-column joints in reinforced concrete structures, etc.) for the capacity of the elements being connected.

2. Insufficient diaphragm action of the floor diaphragms connecting components of the lateral load resisting system. These diaphragms should allow a distribution of story shears to components capable of resisting lateral loads. This requires a proper strength design of such floor diaphragms and, in particular, a careful design for shear transfer from the diaphragms to the vertical elements resisting lateral loads.

3. Instability of individual elements. Axially loaded members that are vital to the integrity of the lateral and vertical load resisting system (primarily columns in frames) should be designed such that buckling of these members is prevented under the largest possible loads that may be expected. Due regard shall be given to overturning effects and possible effects of vertical accelerations. Considering the present state of knowledge on plastic hinging in columns, it is strongly recommended to design columns such that plastic hinges are prevented whenever possible. This leads to the design criteria that at beam-column joints the reduced moment capacity of the columns (under the presence of the largest possible axial load) framings into the joint should be larger than the moment capacity of the beams framing into the joint.

4. Insufficient ductility of structural elements. This is discussed in detail below.

5. Instability of the structural system due to dynamic action. The prime cause of such dynamic instability is the P-Delta effect as discussed below.

Ductility

Ductility is defined as the ability of structural members to deform inelastically without an appreciable loss in strength. The parameter most widely used to describe numerically the demand on ductility is the ductility ratio

$$\mu = \frac{\text{maximum deformation}}{\text{deformation at yield}}$$

This ductility ratio needs to be treated with great caution since it varies widely with the deformation parameter selected. The ratio may be applied to strain, curvature, rotation, shear distortion, deflection, etc., and it is strongly dependent on geometric configuration. It is confusing, therefore, to use the above definition as the basic parameter for evaluating available and required ductility, particularly, since an elastic design and analysis procedure has been selected. Realistic required ductility ratios can only be obtained through a series of dynamic inelastic analyses of the actual structure subjected to acceleration histories that resemble the condemnation threshold earthquake. Clearly, this is in most cases unfeasible for design office work. Also, such dynamic analyses will not necessarily provide the answer needed by the designer, since at the present time no definite correlation exists between ductility ratio demands and required section detailing.

Of primary importance in a design process is a rough estimate of an overall inelastic deformation (ductility) demand for the structure and its elements as well as a method that isolates those elements for which the inelastic deformation demand is probably large and which therefore need special attention in detailings.

As can be deduced from the design philosophy, the overall ductility demand for the structure could be estimated as

$$\bar{\mu} = \frac{CDS}{DFS} = \frac{A_C}{A_D} d_T$$

However, this $\bar{\mu}$ is nothing but an indication of structure ductility demands. Taking the suggested values of d_T from Table 10-1 and the range of A_C/A_D values from Chapter V, it is evident the $\bar{\mu}$ will be rather large for most types of structural systems.

To provide safety against condemnation and failure in systems with large $\bar{\mu}$, it is necessary for the code writing body to formulate a stringent set of design criteria for detailing which assures the attainment of the required ductilities. The recommendations provided in the SEAOC Blue Book appear to be an acceptable example for such design criteria.

For the design for ductility of individual structural elements the following criteria are suggested: First, the design of all elements of the lateral load resisting system shall strictly adhere to the detailing requirements formulated in the previous paragraph and, second, the elements for which the ductility requirements appear to be excessive should be isolated and additional detailing requirements should be considered.

The isolation of such critical elements can be achieved, in an approximate manner, through the computation of an overstress ration, r, at the condemnation level. This ratio is defined as

$$r = \frac{\text{elastic strength demand due to gravity loads plus CDS}}{\text{strength capacity of element}}$$

This ratio is, to some degree, an indication of the ductility demand in elements. However, it is not an absolute measure since it is based on an elastic CDS response and does not include the effects of redistribution of internal forces which is always present in structures subjected to inelastic deformations.

This overstress ratio, called μ_C in the main body of the report, has been selected as the basic measure of ductility demand primarily because it can be computed without much additional effort. If the elastic strength demand, E, due to DFS is known, then the elastic strength demand, E'_C, due to CDS can be computed as

$$E'_C = \frac{CTSD}{DTSF} E = \frac{A_C}{A_D} d_T E$$

as long as all design computations are based on elastic analysis, a more elaborate and time consuming computation of actual ductility ratios is of little value since it will not produce more realistic estimates of ductility demands for structures that respond inelastically at a load level much smaller than that given by CDS.

Acceptable values for r (μ_C) need to be rationally formulated for all types of structural elements, based on satisfactory performance within the previously mentioned set of design criteria for detailing. Some suggested values for different types of structural elements are presented in Chapter X. If these suggestions are followed, it is evident that for $K = 0.67$ buildings the computed r values will in general be smaller than the allowable ones, since the presence of gravity loads will largely affect the required strength capacity of the members. Hence, no additional ductility considerations are required. This is in agreement with accepted practice, since it is well established that properly designed members of ductile moment resisting frames are capable of sustaining large inelastic deformations wihtout loss in strength. The major problems in such frames are sufficient stiffness for drift control and, perhaps, instability problems as will be discussed in the next section.

In systems that include shear walls, deep spandrel beams, or bracing elements, the computed r may exceed the allowable one in critical elements. For such elements additional design criteria should be specified to assure sufficient ductility. Such additional design criteria could be of the following nature:

- In deep spandrel beams and coupling beams of shear walls the shear resistance of the concrete may have to be neglected and the full shear is to be resisted by reinforcement.
- Special shear reinforcement may be specified for elements subjected to high shear (spandrels, coupling beams, piers), such as diagonal X reinforcement.
- Piers and vertical load bearing shear walls may have to include vertical boundary elements.

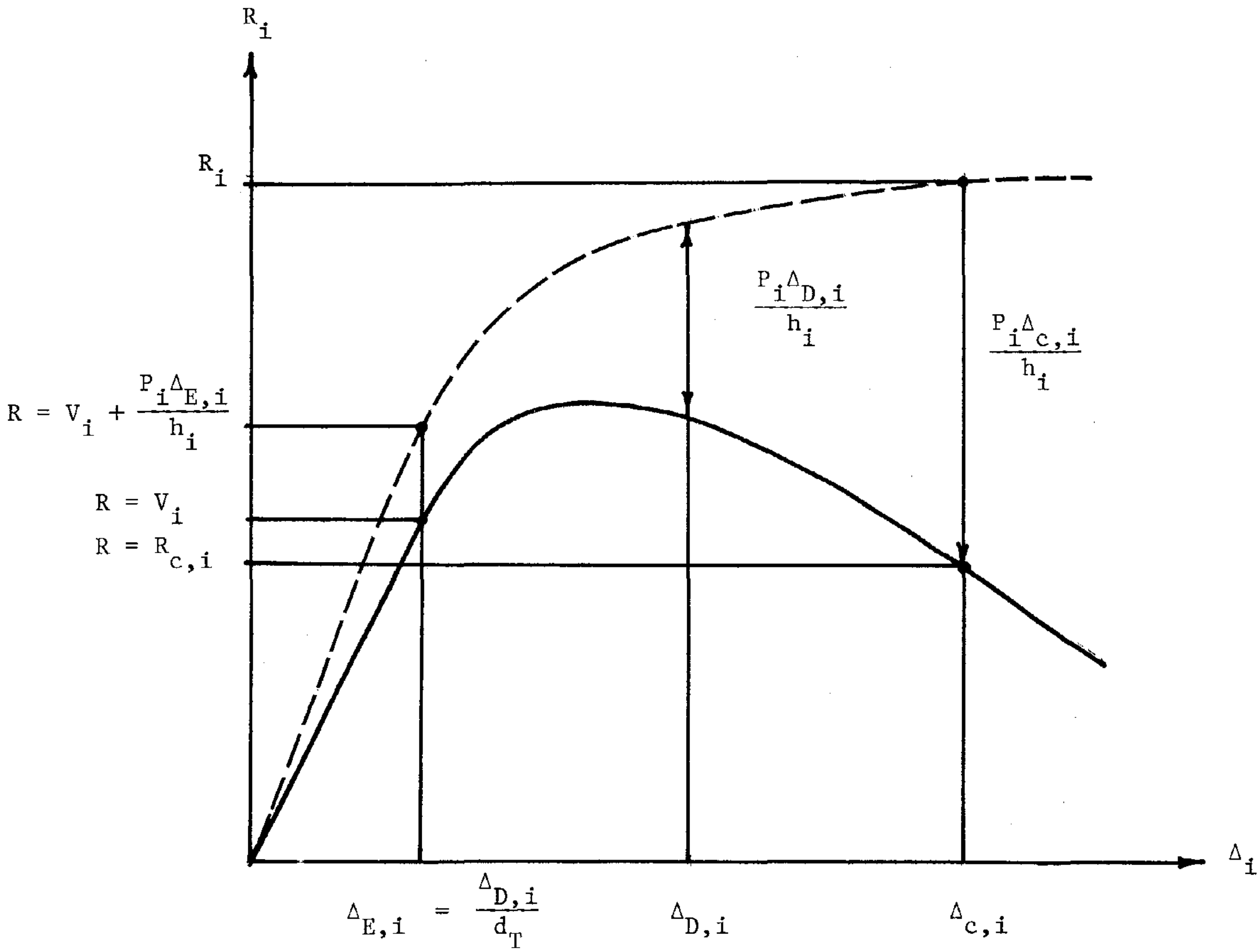
- Shear walls with boundary elements may require special details for chord confinement and splicing of tension reinforcement.

Dynamic Instability

The possibility of earthquakes that may cause lateral deflections in the structure at or beyond the specified condemnation threshold level renders it necessary - at least for flexible structures - to pay attention to dynamic instability considerations. The basic design criterion is that the structure at this level of deformation safety must maintain its vertical load carrying capacity. Safety means that the structure must exhibit, in every story, positive restoring force characteristics when subjected to vertical loads and lateral dynamic excitations.

The restoring force characteristics for a story are illustrated in Figure E-2. There, R_i represents the restoring force of story i at the relative story displacement, $\Delta_{c,i}$, when the effects of axial loads (P-Delta) are neglected. The displacement $\Delta_{c,i}$ is computed through multiplying the elastic relative story displacement at the DFS level by the scaling factor $(A_C/A_D)\ d_T$. R_i can be computed by rational means and is in general equal to the ultimate shear capacity of the story.

The P-Delta effect, given by $P_i \Delta_{c,i}/h_i$, may be significant at this level and may reduce the available restoring force capacity, $R_{c,i}$, to a dangerously small value. Also, the energy absorption capacity, represented by the shaded area, may be reduced to a value smaller than can be justified within the proposed design philosophy.



RESTORING FORCE CHARACTERISTICS FOR STORY i

FIGURE E-2

It is suggested, therefore, to compute the available restoring force capacity at the condemnation threshold level

$$R_{c,i} = R_i - \frac{P_i \Delta_{c,i}}{h_i}$$

and place minimum requirements on $R_{c,i}$, for instance $(R_{c,i})_{min} = V_i$.

Alternatively, simplified design criteria could be developed that will permit the incorporation of the above requirement in the initial strength design procedure. This could be achieved by specifying that, in addition to the conventional elastic strength design criteria, the ultimate shear capacity in each story, R_i, should be at least equal to the specified $(R_{c,i})_{min}$ plus $P_i \Delta_{c,i}/h$.

The design criteria discussed in this appendix are, in general, of approximate nature. This is done for the following two reasons: (1) the criteria should be directed towards providing safety against catastrophic failures but should, at the same time, be of simple form such that they can be utilized by the designer without rendering the design too complicated or costly; and (2) it has to be recognized that earthquakes can not be predicted confidently in regard to peak ground acceleration, frequency content and duration, and, hence, more refined design criteria not necessarily lead to a safer design. Besides fulfilling minimum requirements on strength, stiffness and stability, the designer should not be burdened with additional cumbersome criteria, but instead should invest his time and energy in evaluating the dynamic peculiarities of his structure and, above all, in proper detailing of elements and structural connections to assure sufficient ductility.

APPENDIX F

STATISTICAL ANALYSIS OF ACCELERATION PEAKS. (32 ACCELEROGRAPHS).

The complete time history of each of the 32 accelerograms considered in this project is reviewed and the peaks are located. The RMS acceleration and the ratio of PGA and RMS acceleration are computed for each of these records. Based on these calculated values, one can see that the ratios of PGA and RMS acceleration lie within the range of 2 to 15, with a mean value of 7.47. From the plot of PGA versus RMS acceleration, it is clear that the two parameters are, in general, linearly proportioned to each other, with the exception of a few records. This shows that the PGA of a record does influence the RMS acceleration of that record. Furthermore, it is evident from the histograms plotted for each record that most of the peaks lie within the range of 10 to 20 percent of the PGA of that record and the shape of the distribution of peaks for each record looks almost alike.

From the above statistical analysis one can conclude that it is justifiable to reduce the spectral shape obtained from using the PGA values to a certain percentage to take into consideration the distribution of peaks in each of the records considered. To be on the conservative side, we recommend reduction factor of 0.9. This implies that the effective peak is taken to be 90 percent of the PGA. As can be seen from the following results, hardly 1 percent of the peaks in any of the records exceeds the recommended level.

Record No.	Earthquake and Date	PGA (g)	RMS Accel.(g)	PGA/RMS
1	El Centro, Calif.,12/30/34	0.1600	0.0203	7.88
2	"	0.1828	0.0218	8.39
3	Helena, Montana, 10/31/35	0.1464	0.0099	14.83
4	"	0.1454	0.0125	11.67
5	Western Washington,4/13/49	0.1649	0.0246	6.72
6	"	0.2802	0.0298	9.40
7	Wheeler Ridge, Ca.,1/12/54	0.0652	0.0064	10.25
8	"	0.0682	0.0066	10.25
9	Parkfield, Calif., 6/27/66	0.3549	0.0316	11.22
10	"	0.4344	0.0361	12.03
11	"	0.2374	0.0286	8.28
12	"	0.2751	0.0315	8.72
13	Borrego Mountain, 4/8/68	0.0408	0.0072	5.63
14	"	0.0464	0.0072	6.41
15	San Fernando, Ca., 2/9/71	1.1715	0.1193	9.82
16	"	1.0765	0.1137	9.47
17	"	0.0382	0.0133	2.87
18	"	0.0306	0.0088	3.46
19	"	0.0933	0.0192	4.86
20	"	0.1230	0.0253	4.85
21	"	0.3532	0.0411	8.59
22	"	0.2836	0.0384	7.38
23	"	0.0687	0.0122	5.63
24	"	0.0686	0.0134	5.12
25	Managua, Nicaragua, 12/23/72	0.3289	0.0548	6.00
26	"	0.3806	0.0490	7.77
27	"	0.3326	0.0495	6.72
28	"	0.2887	0.0432	6.68
29	Managua, Nicaragua, 1/4/68	0.1250	0.0243	5.15
30	"	0.0968	0.0217	4.45
31	Managua, Nicaragua, 3/31/73	0.2508	0.0588	4.26
32	"	0.5916	0.1376	4.30

Mean PGA/RMS
= 7.47

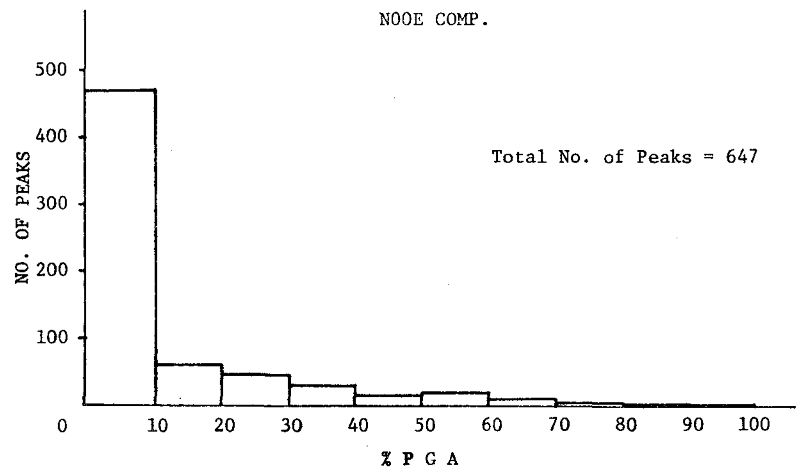
NUMBER OF PEAKS

Record No.	0%	10%	20%	30%	40%	50%	60%	70%	80%	90%	100%
1	466	59	45	30	15	17	9	3	2	1	
2	499	72	45	28	19	5	9	2	3	1	
3	483	24	4	6	1	3	1	3	0	1	
4	643	18	7	6	1	2	1	4	1	2	
5	347	90	52	41	18	19	12	4	3	3	
6	413	81	42	40	11	6	2	0	0	1	
7	99	44	24	8	8	3	0	0	1	2	
8	222	44	20	13	5	5	4	1	0	1	
9	302	40	12	8	3	0	0	1	0	1	
10	264	21	9	6	7	1	0	0	0	1	
11	168	43	29	4	5	3	1	0	0	1	
12	200	44	13	10	1	0	2	1	0	1	
13	154	111	47	20	8	9	2	3	0	1	
14	130	85	52	16	13	6	4	1	0	1	
15	496	34	23	12	6	2	1	2	0	1	
16	491	45	24	9	8	3	2	1	0	2	
17	13	12	10	12	9	11	7	7	3	1	
18	11	24	20	17	19	5	5	2	1	3	
19	183	81	37	27	15	21	9	5	0	2	
20	163	71	55	27	16	14	11	5	0	2	
21	346	56	24	8	5	2	4	0	0	2	
22	370	68	29	7	5	1	4	4	2	2	
23	222	87	33	25	9	10	8	5	1	1	
24	194	78	38	29	18	14	8	5	4	2	
25	319	23	22	11	6	11	7	2	6	4	
26	328	53	19	19	9	7	4	2	0	1	
27	307	18	9	5	2	5	0	1	0	1	
28	238	39	9	7	7	3	1	0	0	1	
29	124	19	8	10	3	3	1	0	0	3	
30	132	21	13	3	2	3	4	2	0	3	
31	114	21	9	7	3	1	1	3	1	3	
32	120	9	7	9	1	1	1	0	1	1	

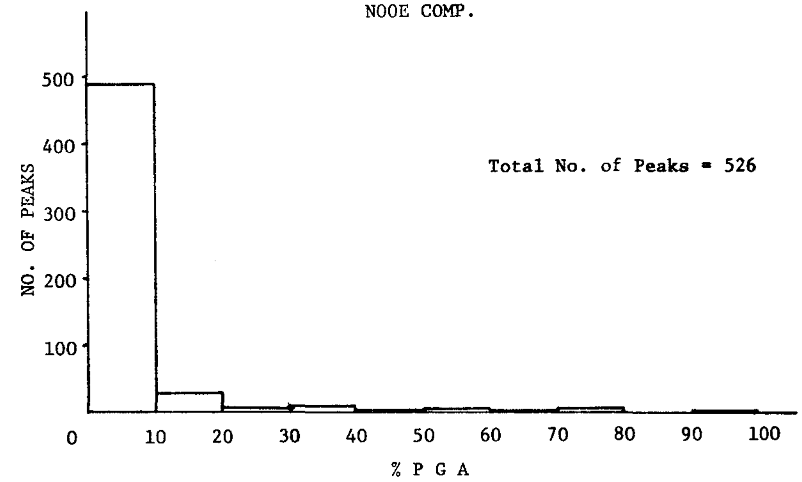
NO. OF PEAKS ABOVE PERCENTAGE PGA

Record No.	0%	10%	20%	30%	40%	50%	60%	70%	80%	90%	100%
1	647	181	122	77	47	32	15	6	3	1	1
2	683	184	112	67	39	20	15	6	4	1	1
3	526	43	19	15	9	8	5	4	1	1	1
4	685	42	24	17	11	10	8	7	3	2	1
5	589	242	152	100	59	41	22	10	6	3	1
6	596	183	102	60	20	9	3	1	1	1	1
7	289	90	46	22	14	6	3	3	3	2	1
8	315	93	49	29	16	11	6	2	1	1	1
9	367	65	25	13	5	2	2	2	1	1	1
10	309	45	24	15	9	2	1	1	1	1	1
11	254	86	43	14	10	5	2	1	1	1	1
12	272	72	28	15	5	4	4	2	1	1	1
13	355	201	90	43	23	15	6	4	1	1	1
14	308	178	93	41	25	12	6	2	1	1	1
15	577	81	47	24	12	6	4	3	1	1	1
16	585	94	49	25	16	8	5	3	2	2	1
17	85	72	60	50	38	29	18	11	4	1	1
18	107	96	72	52	35	16	11	6	4	3	1
19	380	197	116	79	52	37	16	7	2	2	1
20	364	201	130	75	48	32	18	7	2	2	1
21	447	101	45	21	13	8	6	2	2	2	1
22	492	122	54	25	18	13	12	8	4	2	1
23	421	199	112	79	54	25	15	7	2	1	1
24	390	196	118	80	51	33	19	11	6	2	1
25	411	92	69	47	36	30	19	12	10	4	1
26	442	114	61	42	23	14	7	3	1	1	1
27	348	41	23	14	9	7	2	2	1	1	1
28	303	65	26	15	12	5	2	1	1	1	1
29	171	47	28	20	10	7	4	3	3	3	1
30	183	51	30	17	14	12	9	5	3	3	1
31	163	49	28	19	12	9	8	7	4	3	1
32	150	30	21	14	5	4	3	2	2	1	1

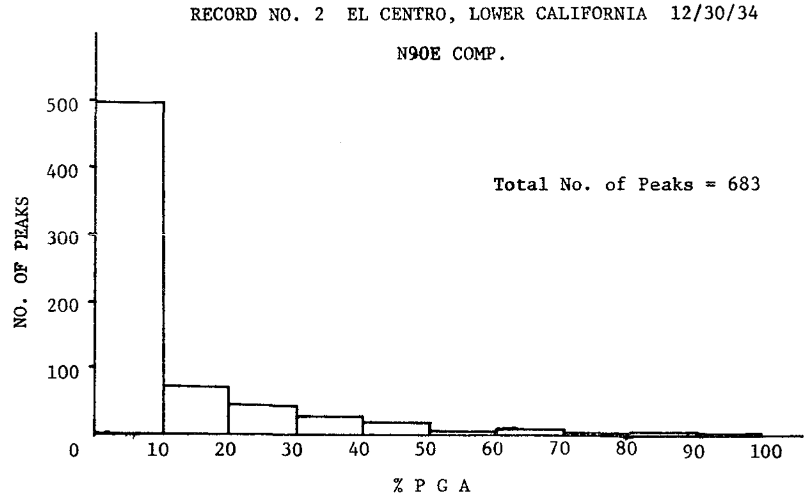
RECORD NO. 1 EL CENTRO, LOWER CALIFORNIA 12/30/34
NOOE COMP.



RECORD NO. 3 HELENA, MONTANA 10/31/35
NOOE COMP.



RECORD NO. 2 EL CENTRO, LOWER CALIFORNIA 12/30/34
N90E COMP.



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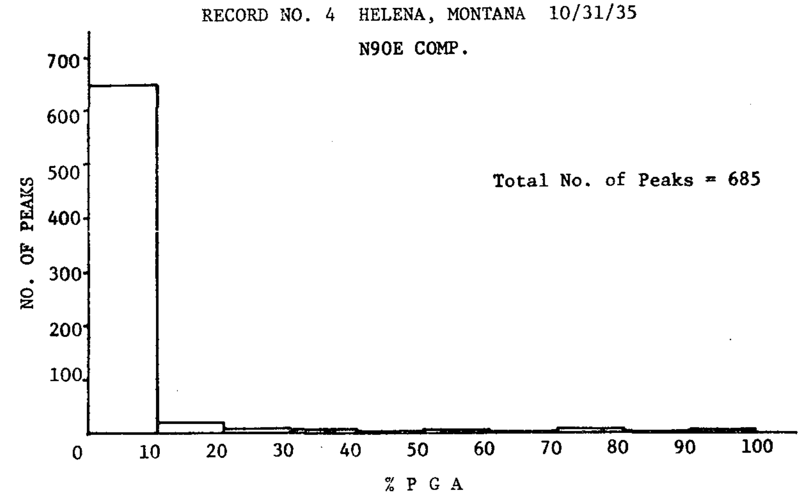
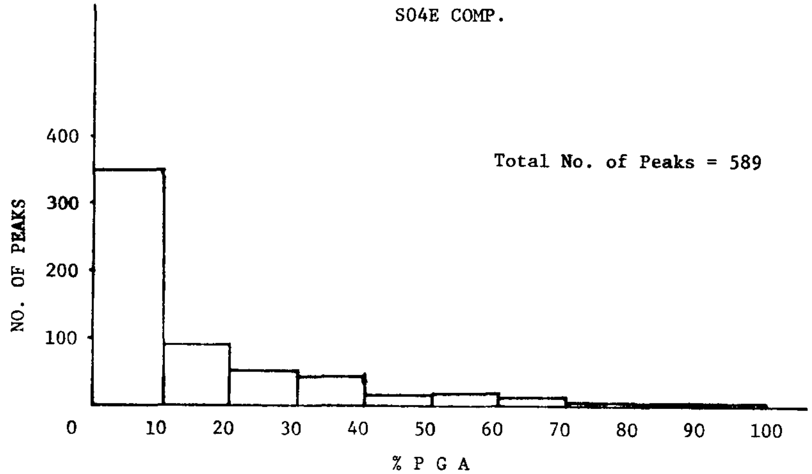
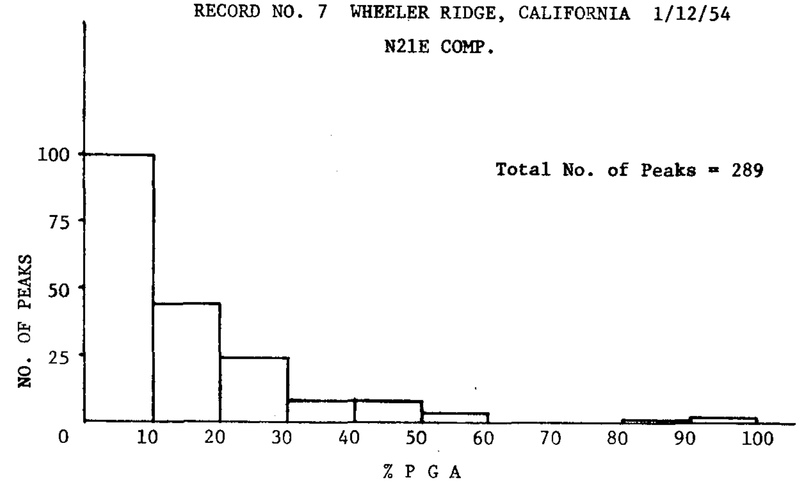


RECORD NO. 5 WESTERN WASHINGTON 4/13/49
S04E COMP.



RECORD NO. 7 WHEELER RIDGE, CALIFORNIA 1/12/54
N21E COMP.



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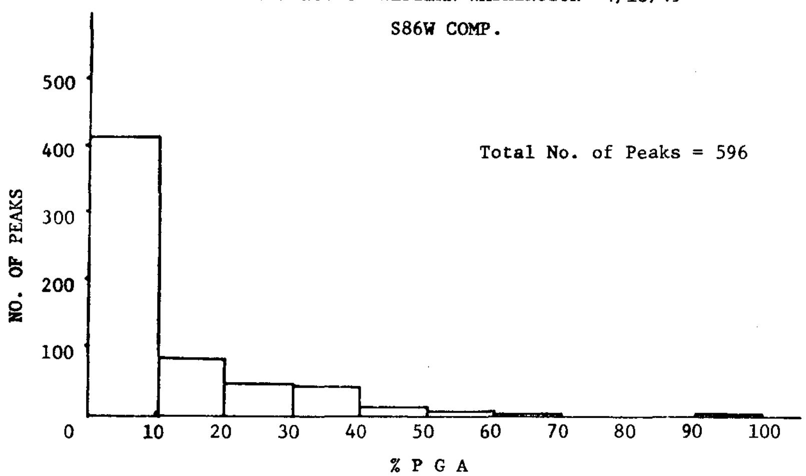
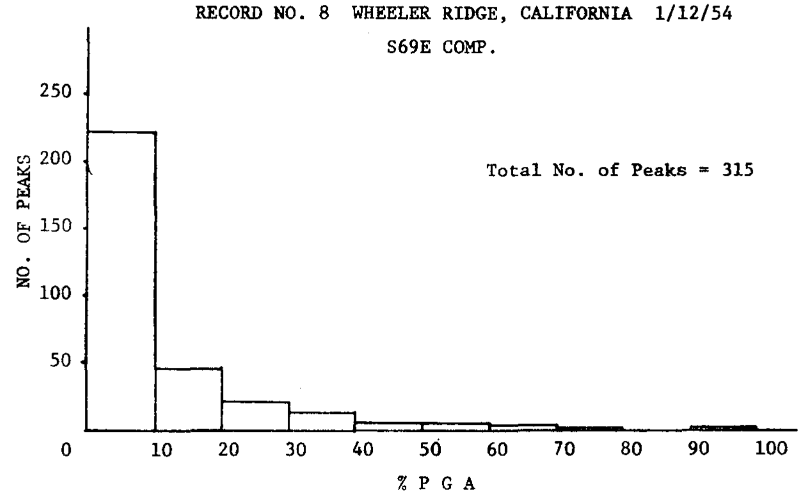
S86W COMP.



RECORD NO. 8 WHEELER RIDGE, CALIFORNIA 1/12/54
S69E COMP.



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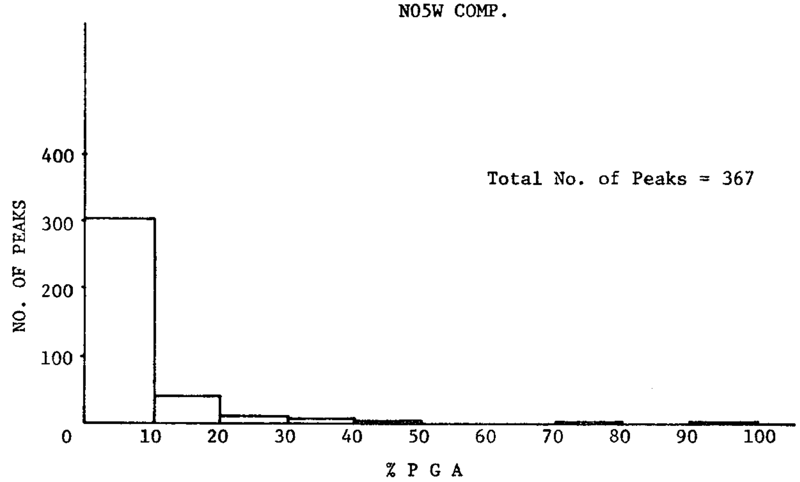
N05W COMP.



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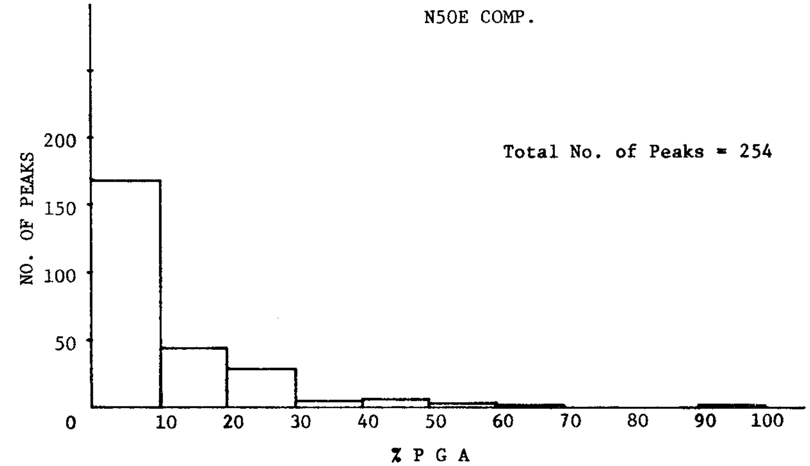
N50E COMP.



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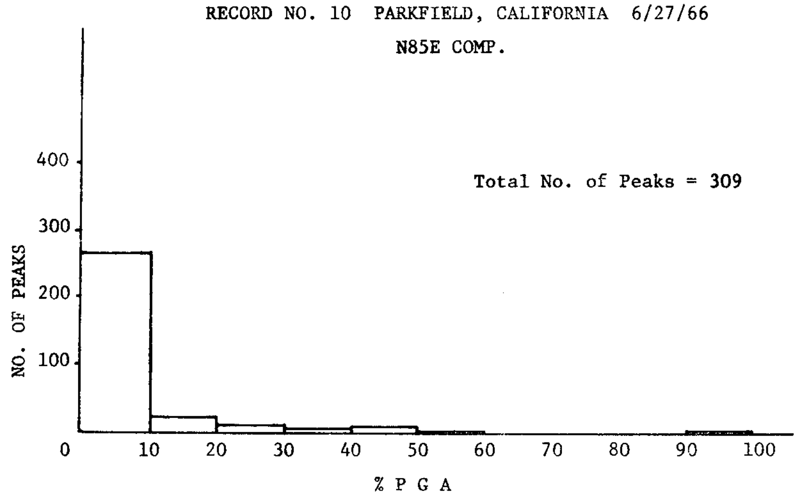

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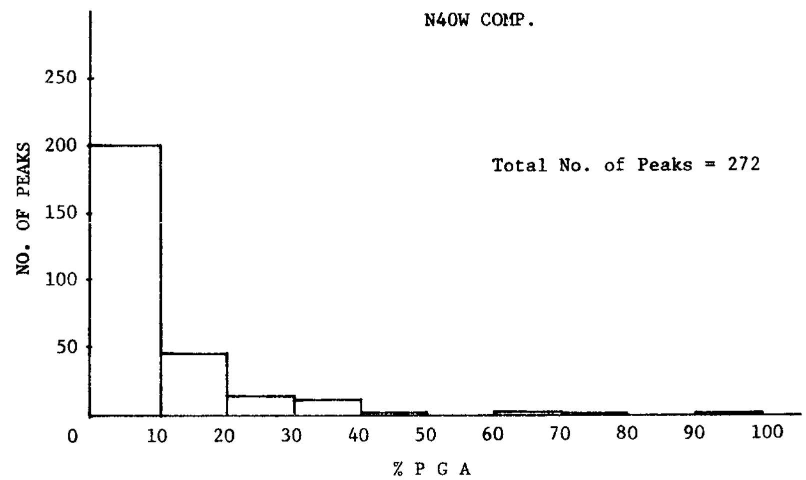
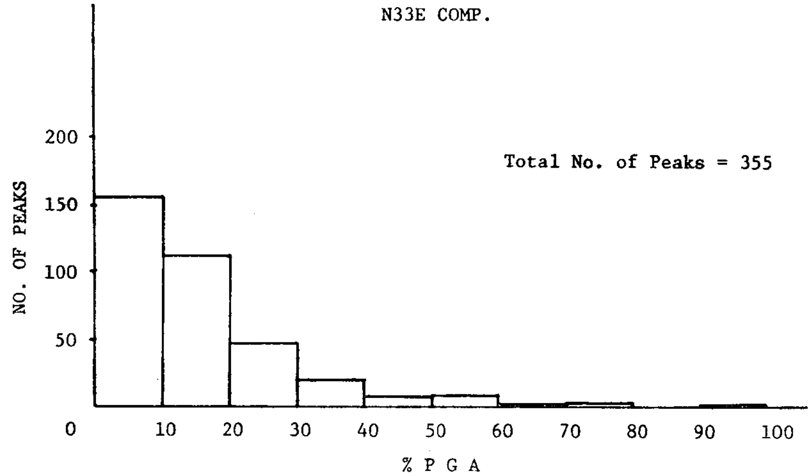
N40W COMP.



RECORD NO. 13 BORREGO MOUNTAIN 4/8/68
N33E COMP.



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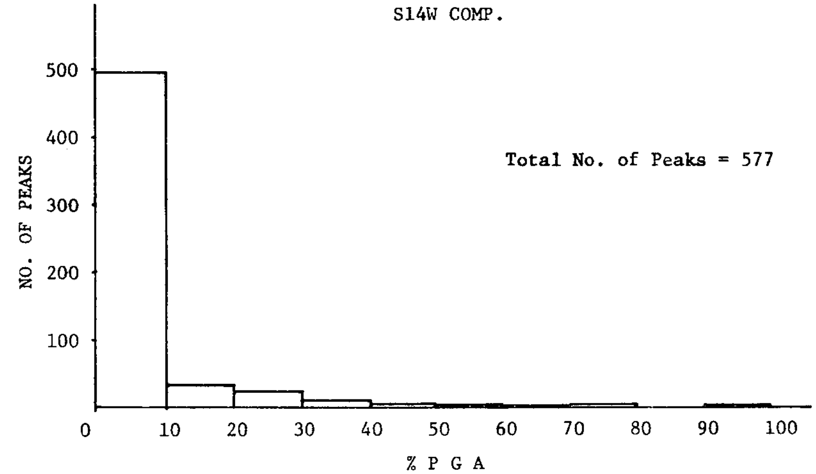
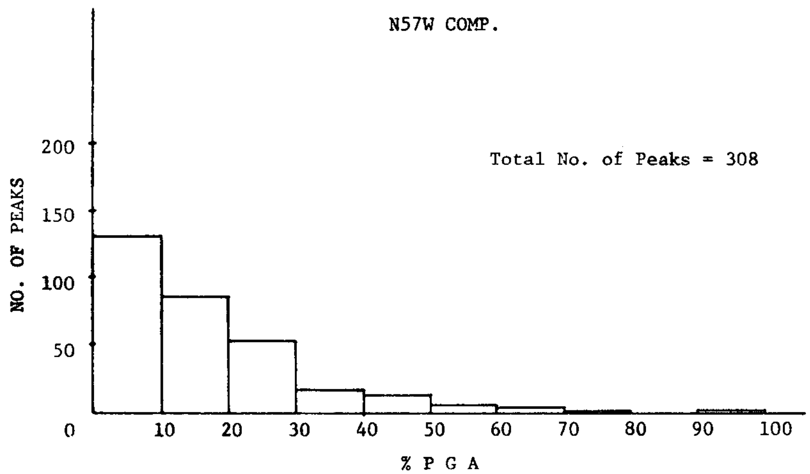
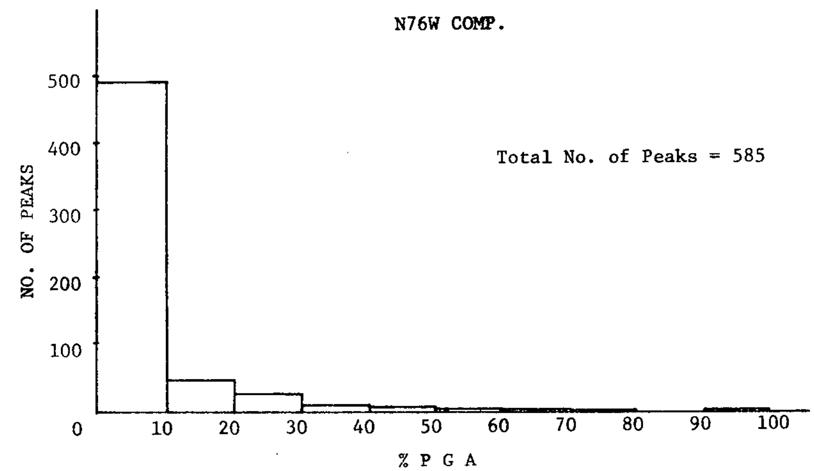
S14W COMP.



RECORD NO. 14 BORREGO MOUNTAIN 4/8/68
N57W COMP.

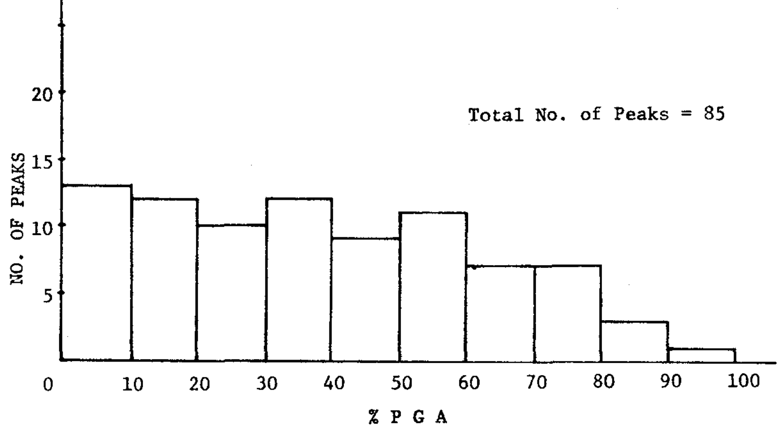


RECORD NO. 16 SAN FERNANDO, CALIFORNIA 2/9/71
N76W COMP.



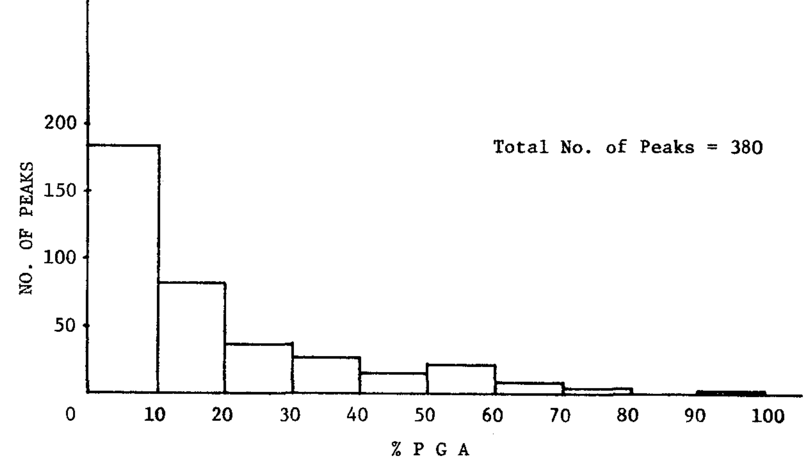
RECORD NO. 17 SAN FERNANDO, CALIFORNIA 2/9/71

SOOW COMP.



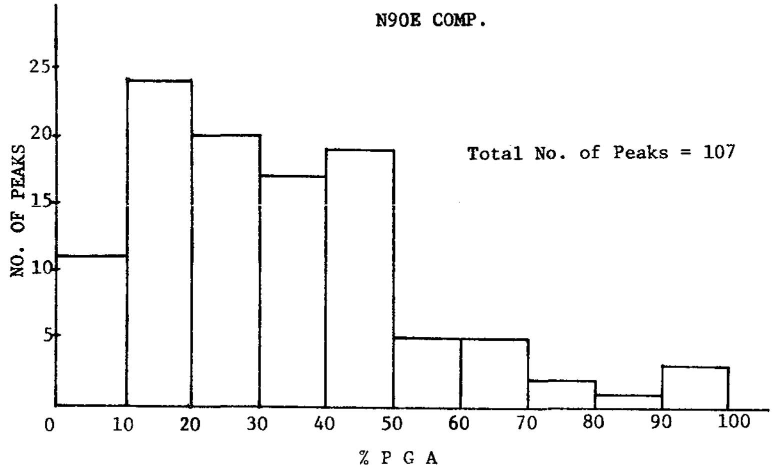
RECORD NO. 19 SAN FERNANDO, CALIFORNIA 2/9/71

NOOE COMP.



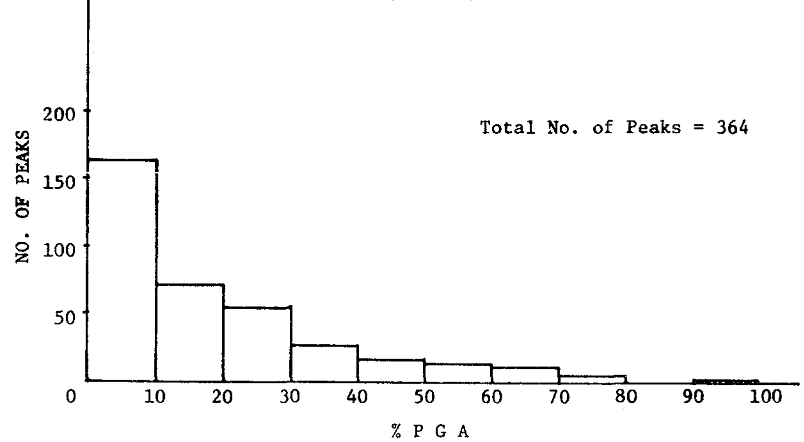
RECORD NO. 18 SAN FERNANDO, CALIFORNIA 2/9/71

N90E COMP.

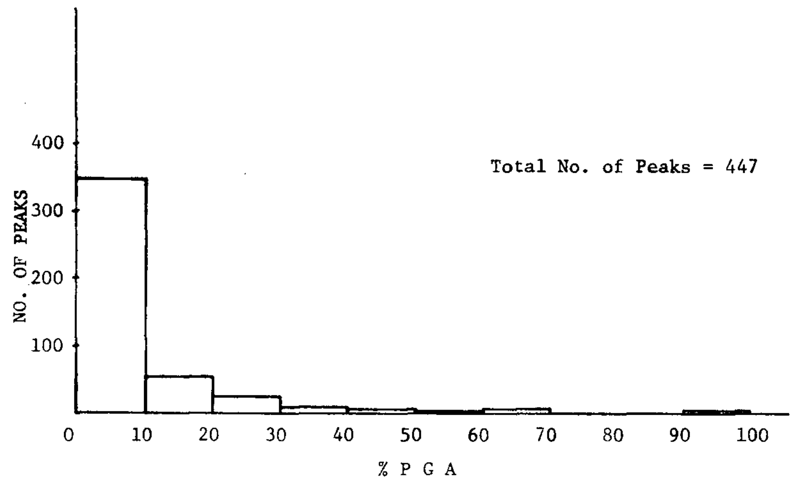


RECORD NO. 20 SAN FERNANDO, CALIFORNIA 2/9/71

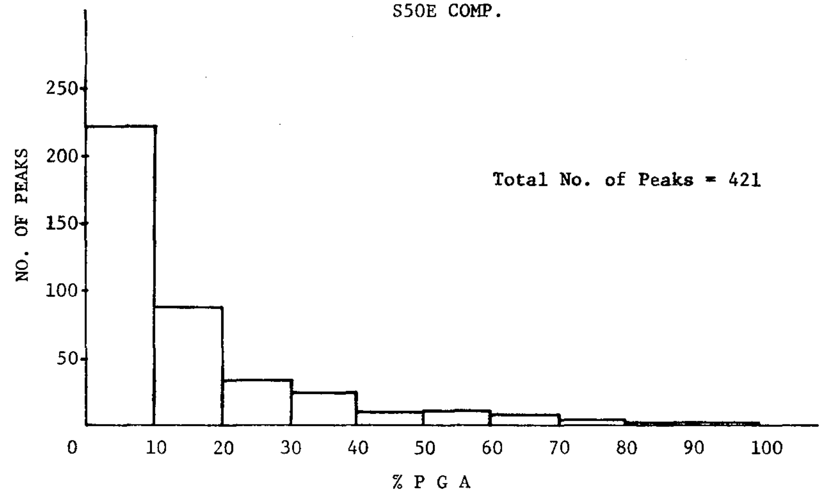
N90W COMP.



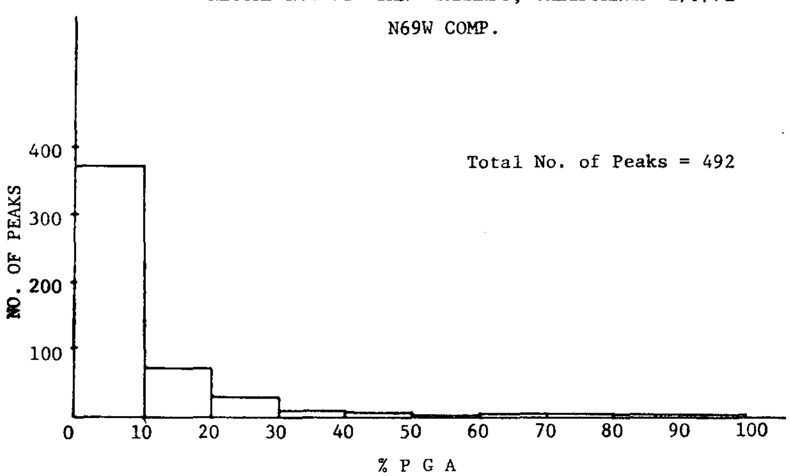
RECORD NO. 21 SAN FERNANDO, CALIFORNIA 2/9/71
N21E COMP.



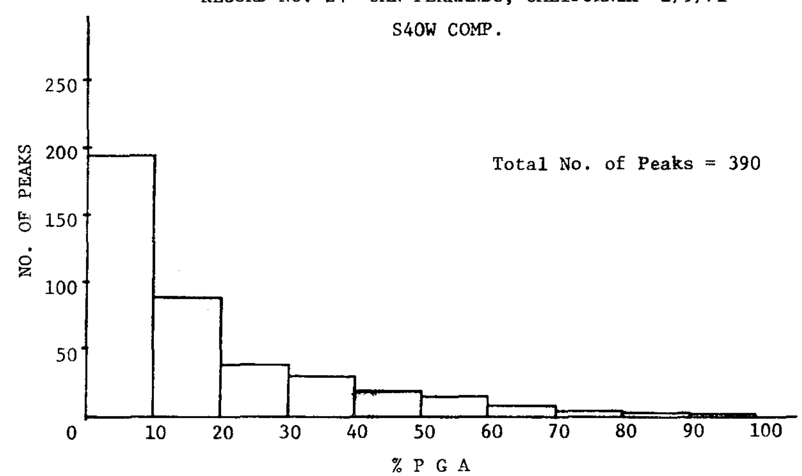
RECORD NO. 23 SAN FERNANDO, CALIFORNIA 2/9/71
S50E COMP.



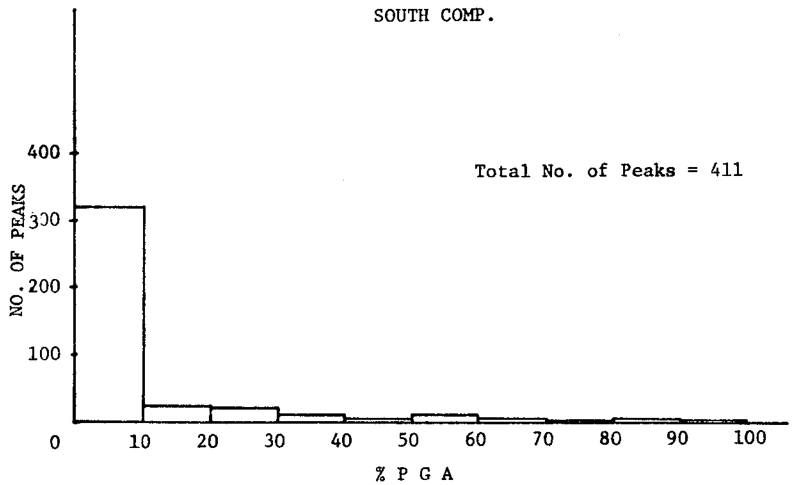
RECORD NO. 22 SAN FERNANDO, CALIFORNIA 2/9/71
N69W COMP.



RECORD NO. 24 SAN FERNANDO, CALIFORNIA 2/9/71
S40W COMP.



RECORD NO. 25 MANAGUA, NICARAGUA 12/23/72
SOUTH COMP.



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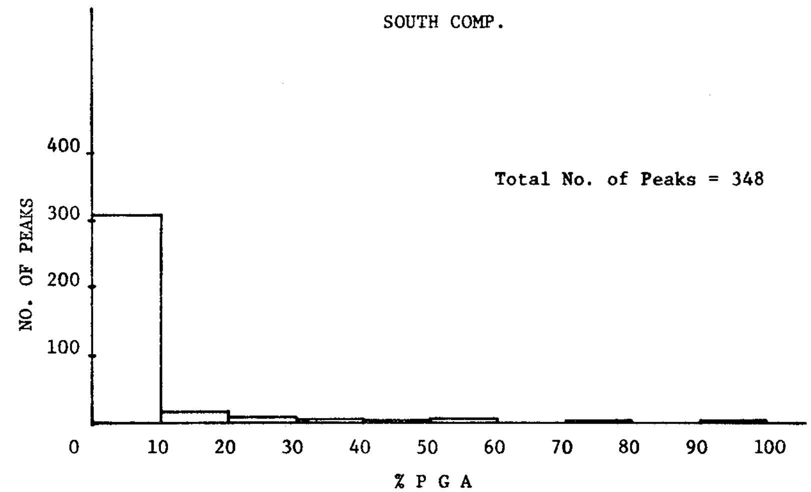
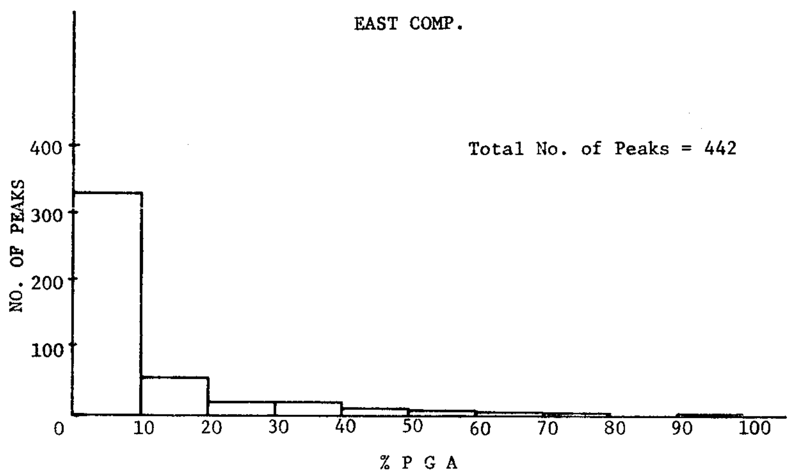
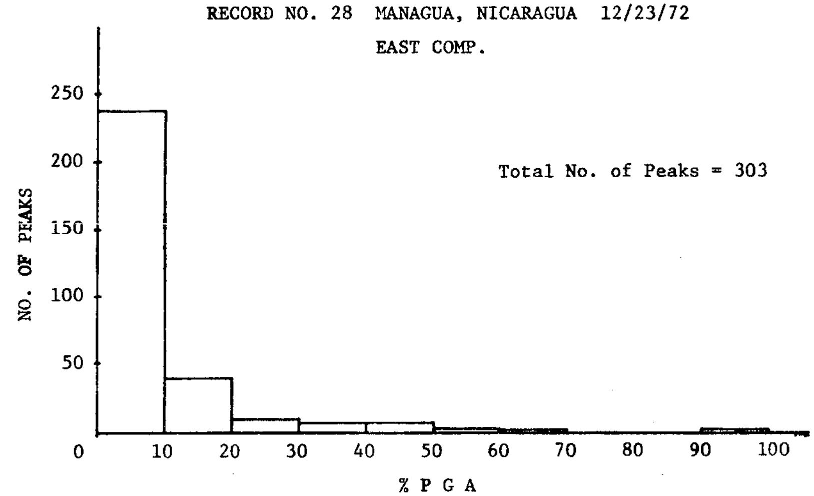


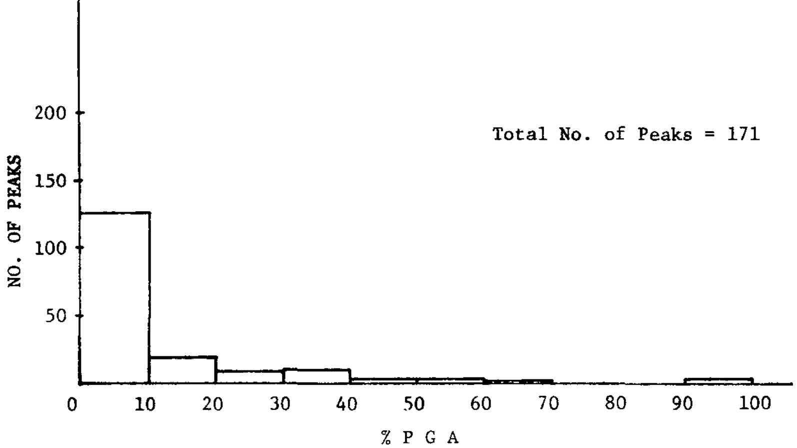
RECORD NO. 26 MANAGUA, NICARAGUA 12/23/72
EAST COMP.



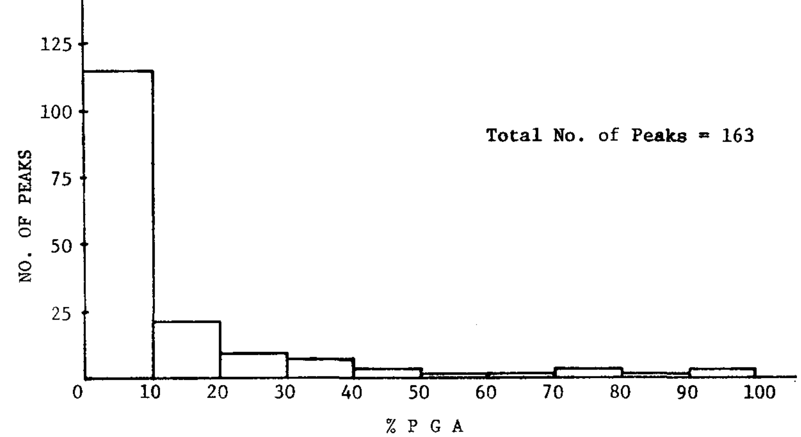
RECORD NO. 28 MANAGUA, NICARAGUA 12/23/72
EAST COMP.



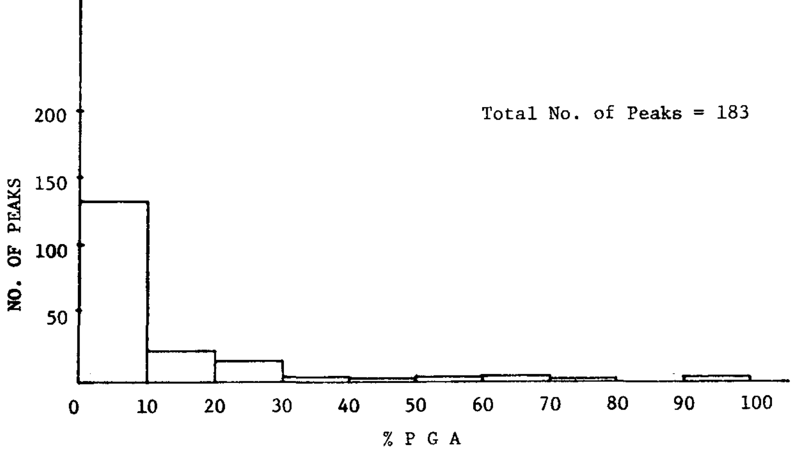
RECORD NO. 29 MANAGUA, NICARAGUA 1/4/68
N84.5W COMP.



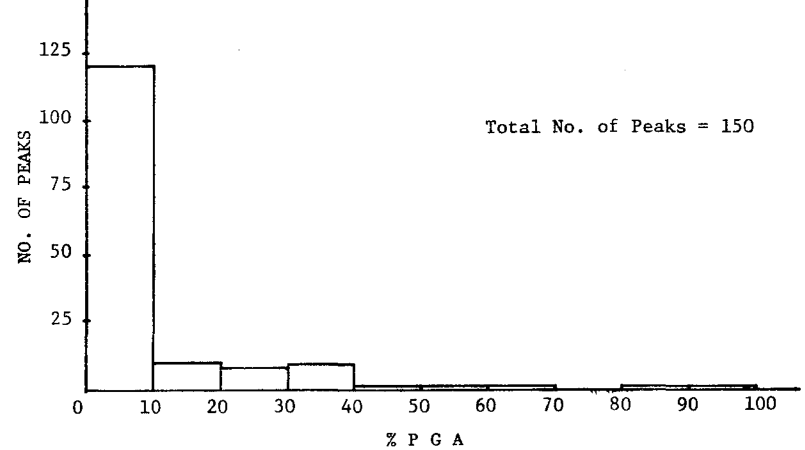
RECORD NO. 31 MANAGUA, NICARAGUA 3/31/73
N-S COMP.



RECORD NO. 30 MANAGUA, NICARAGUA 1/4/68
S05.5W COMP.

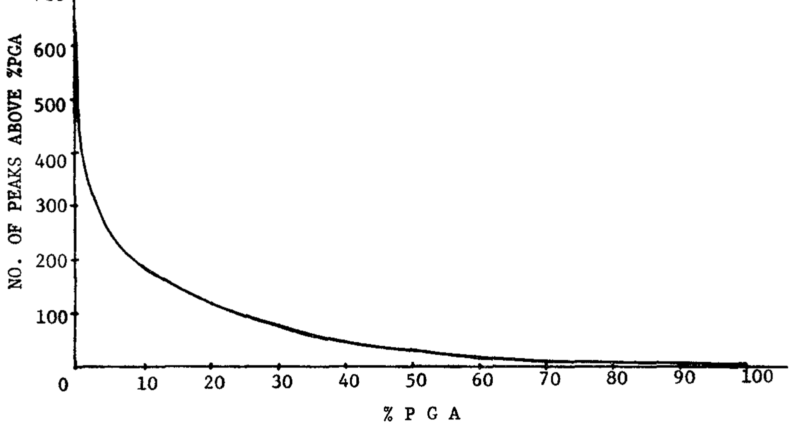


RECORD NO. 32 MANAGUA, NICARAGUA 3/31/73
E-W COMP.



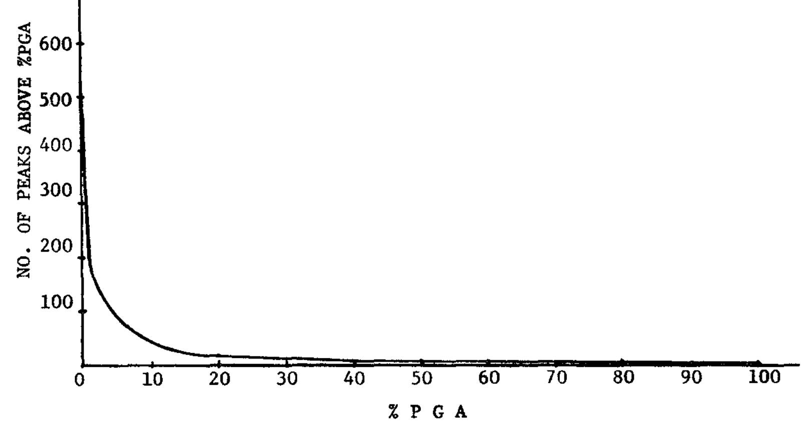
RECORD NO. 1 EL CENTRO, LOWER CALIFORNIA 12/30/34

NOOE COMP.



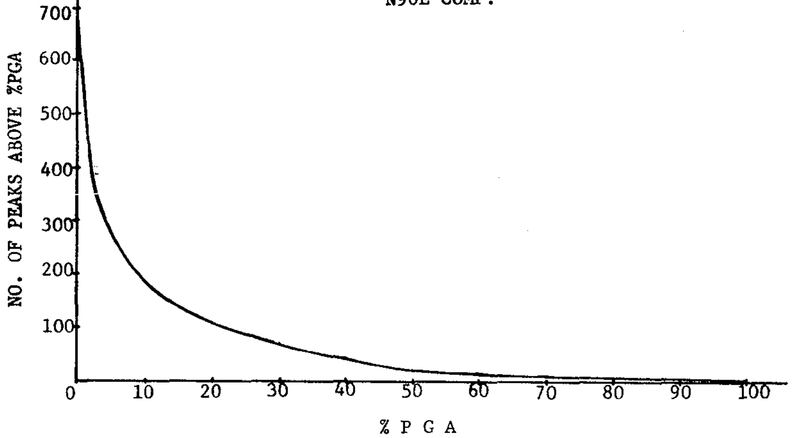
RECORD NO. 3 HELENA, MONTANA 10/31/35

NOOE COMP.



RECORD NO. 2 EL CENTRO, CALIFORNIA 12/30/34

N90E COMP.



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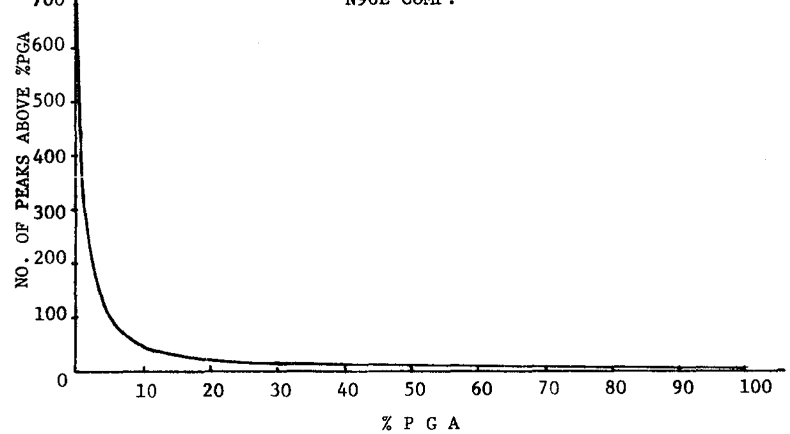
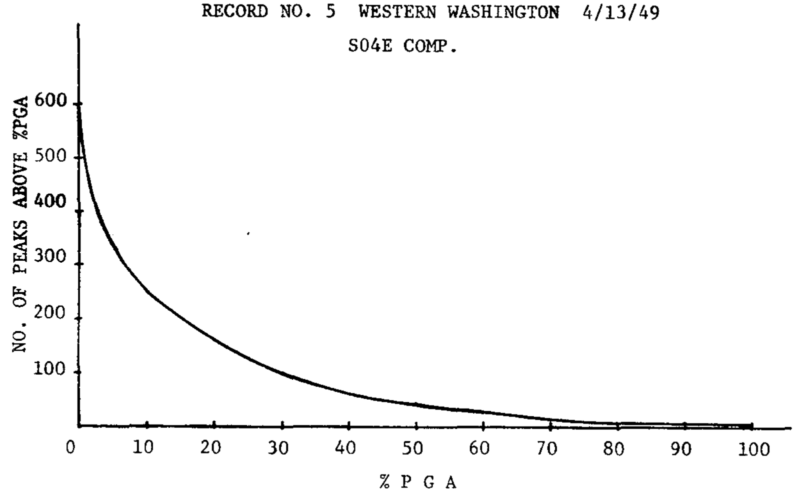
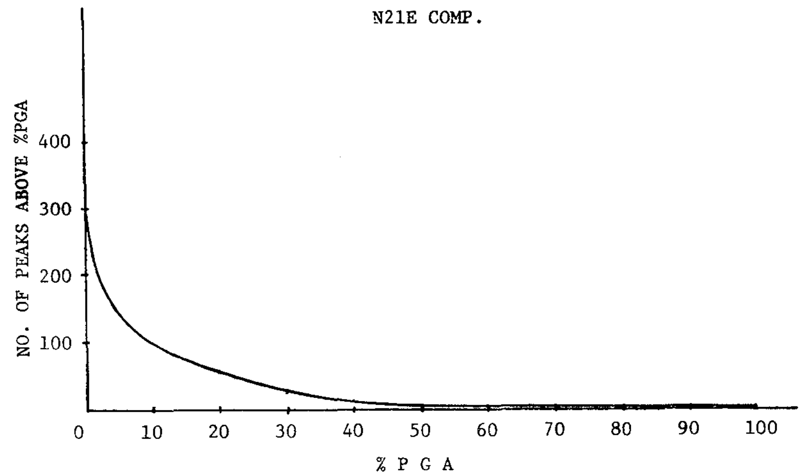
N90E COMP.



RECORD NO. 5 WESTERN WASHINGTON 4/13/49
S04E COMP.



RECORD NO. 8 WHEELER RIDGE, CALIFORNIA 1/12/54
N21E COMP.



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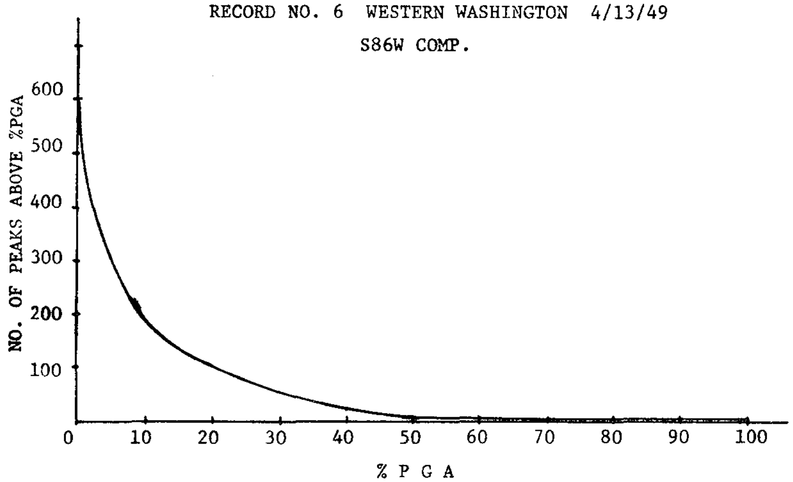
S86W COMP.



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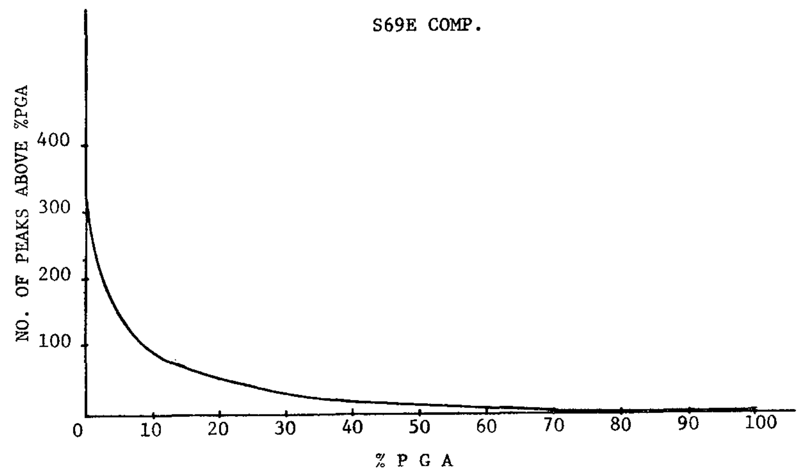
S69E COMP.



RECORD NO. 9 PARKFIELD, CALIFORNIA 6/27/66
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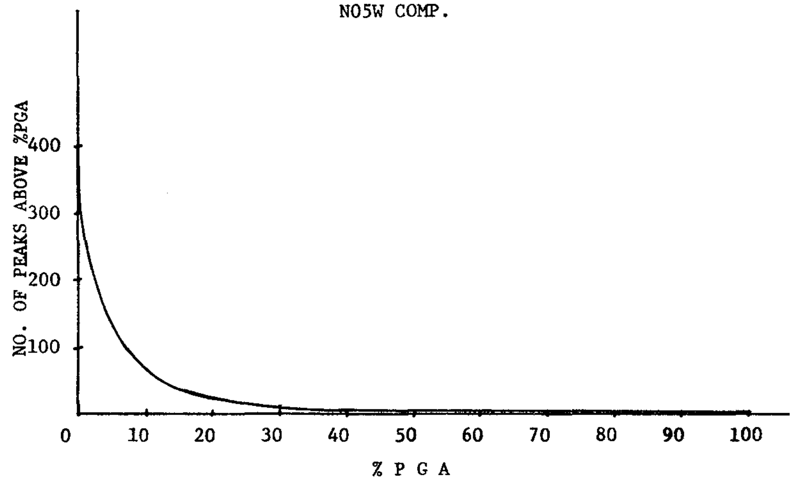

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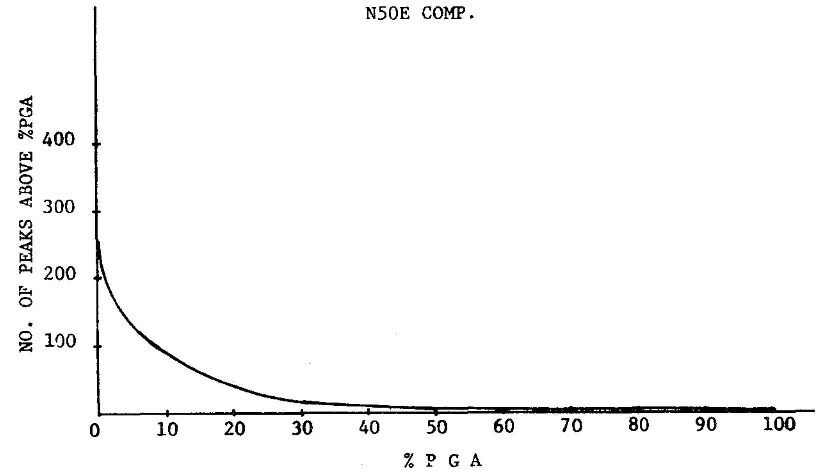
N50E COMP.



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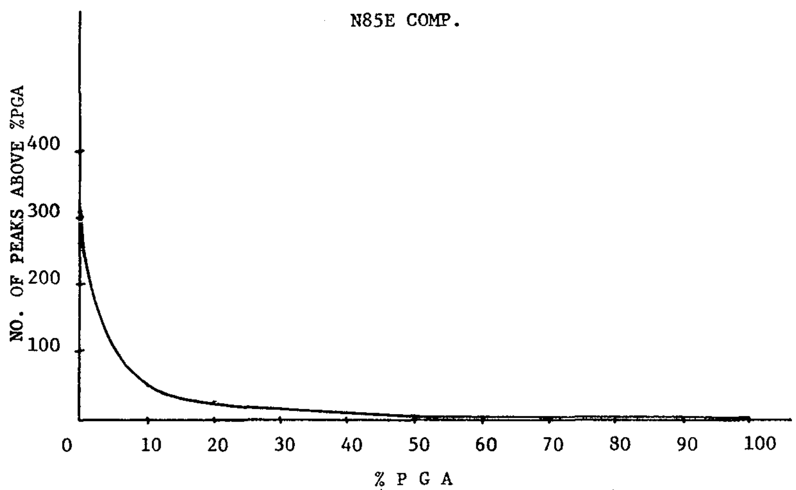
N85E COMP.



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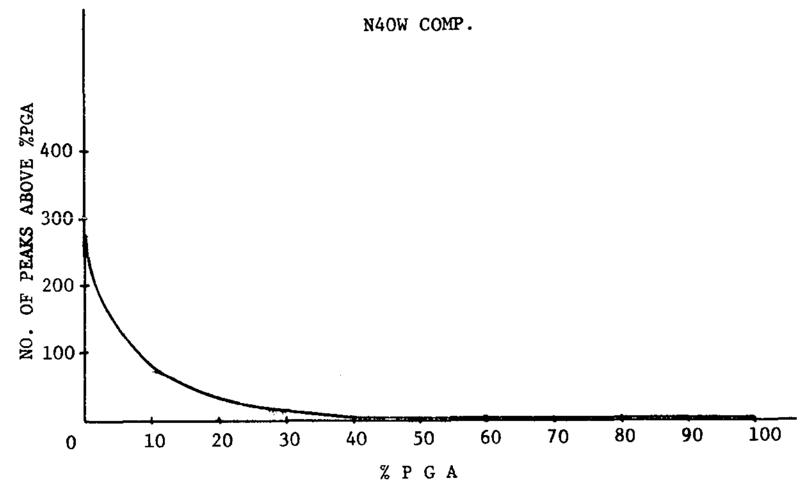

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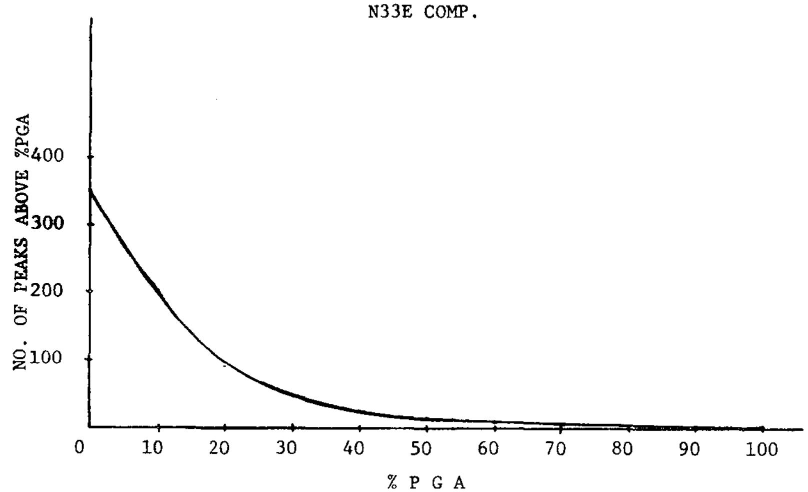
N33E COMP.



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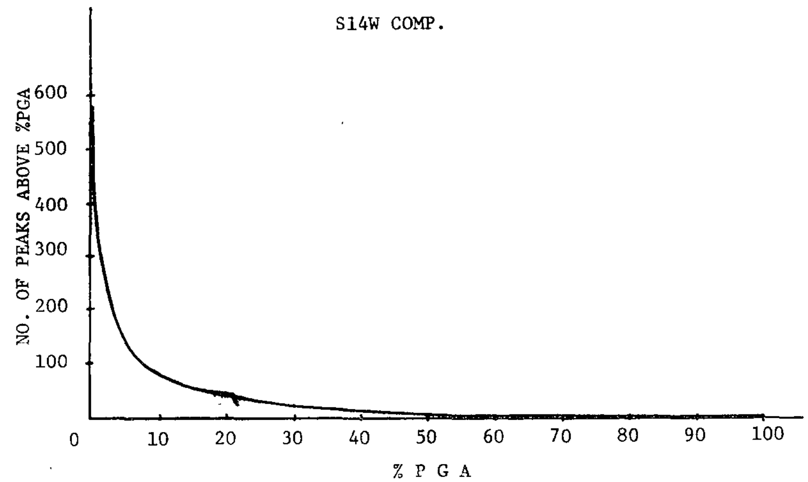

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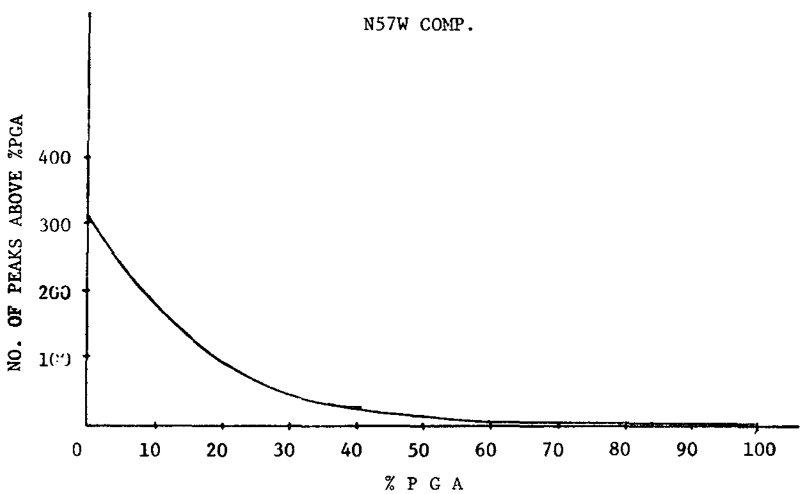

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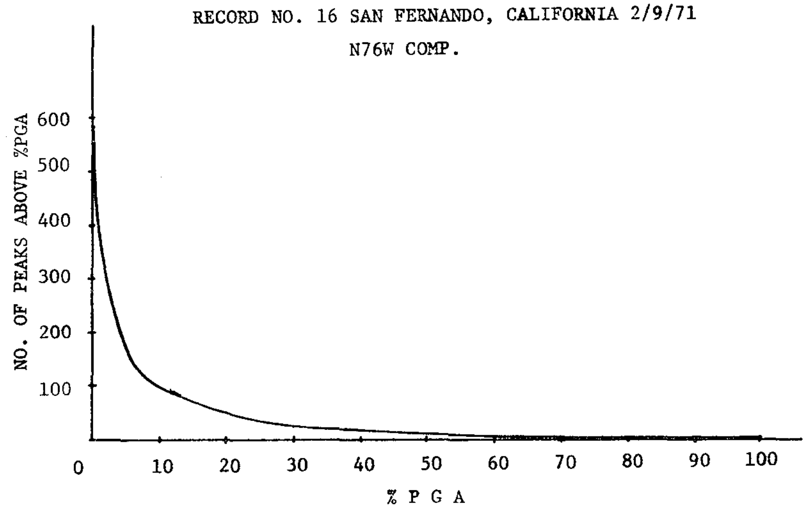

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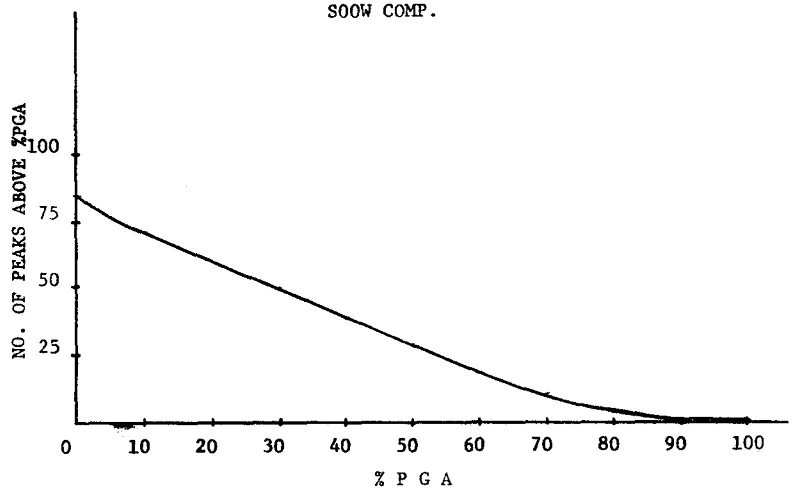
SOOW COMP.



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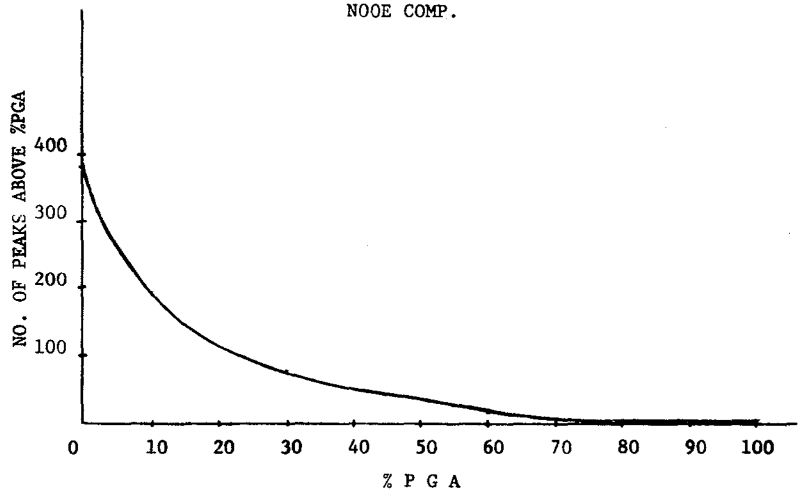
NOOE COMP.



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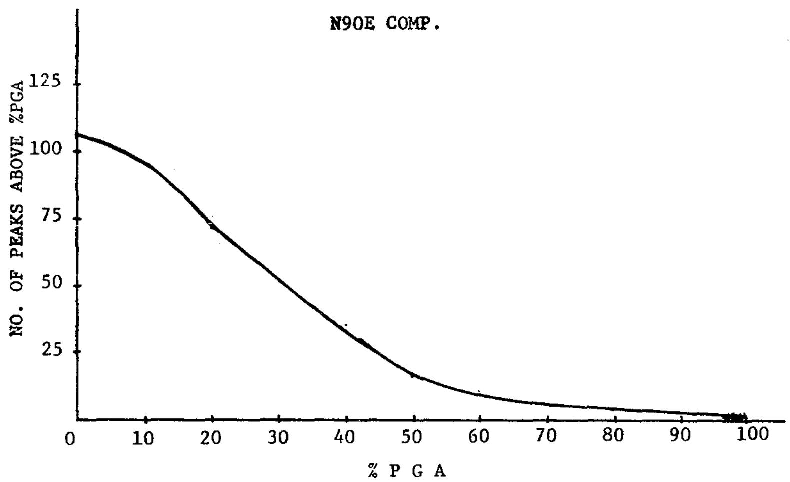
N90E COMP.



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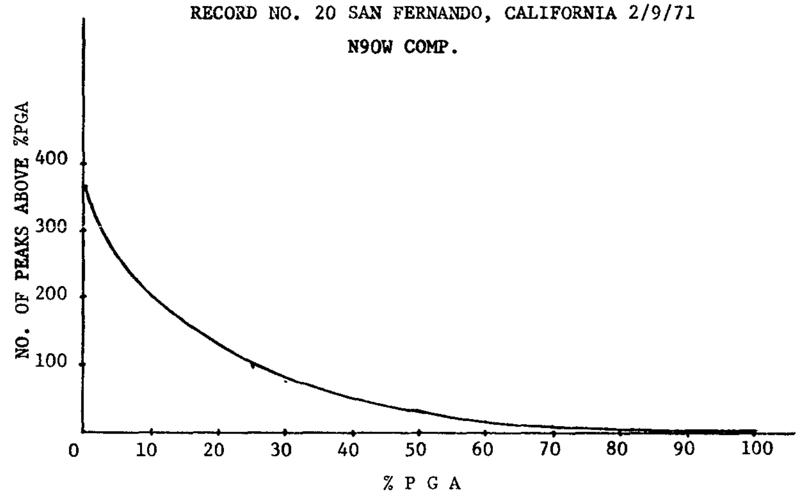

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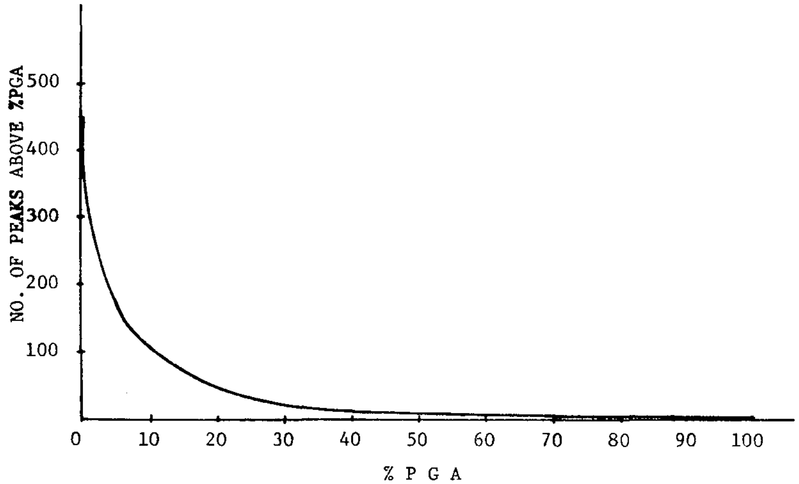

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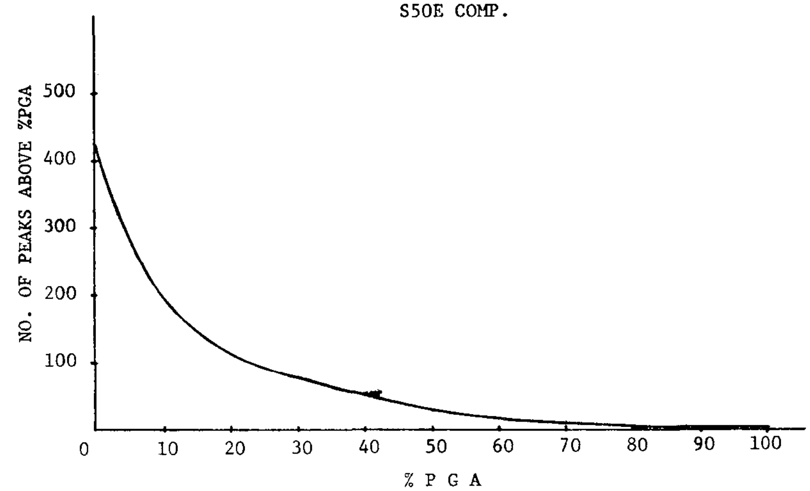

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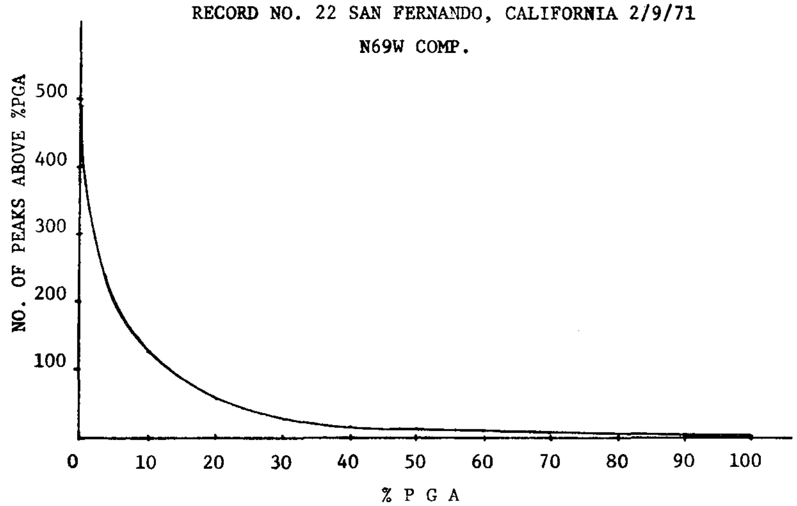

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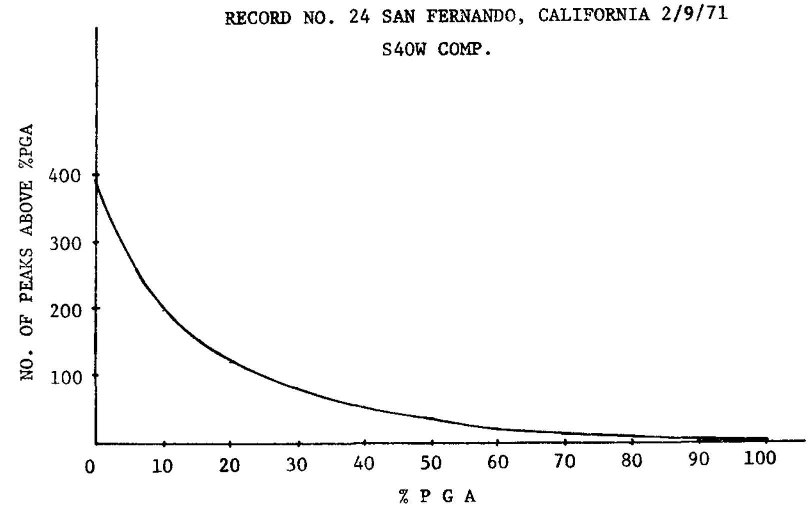

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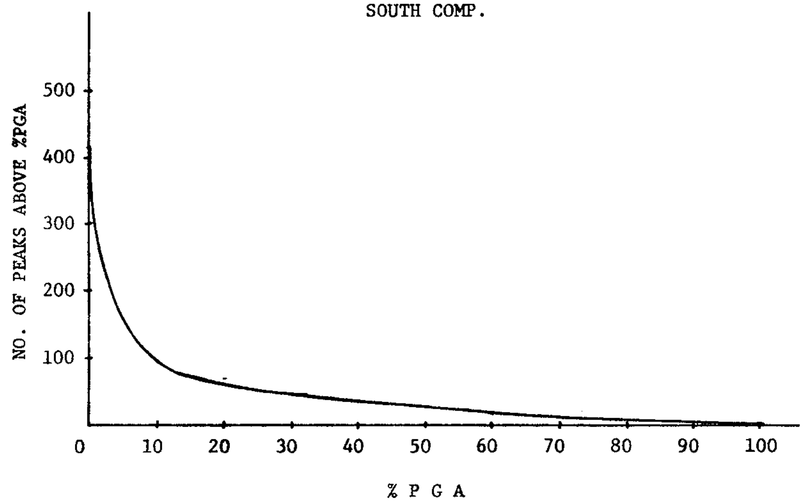
SOUTH COMP.



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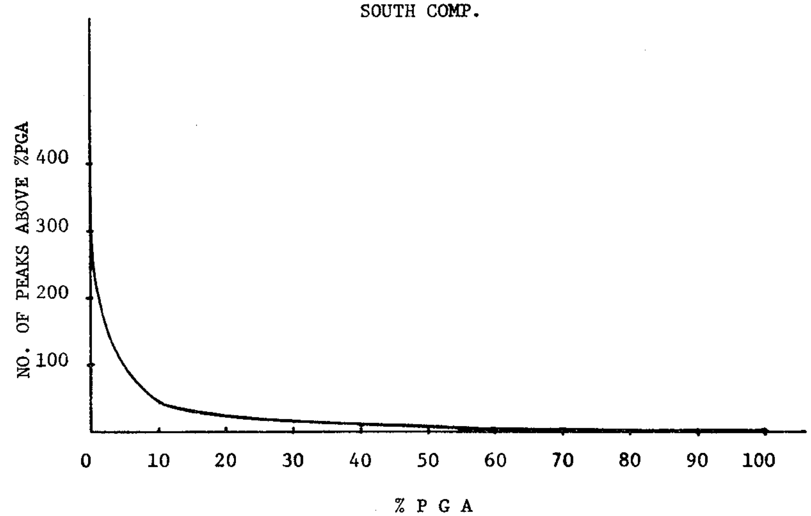

F-20

RECORD NO. 26 MANAGUA, NICARAGUA 12/23/72
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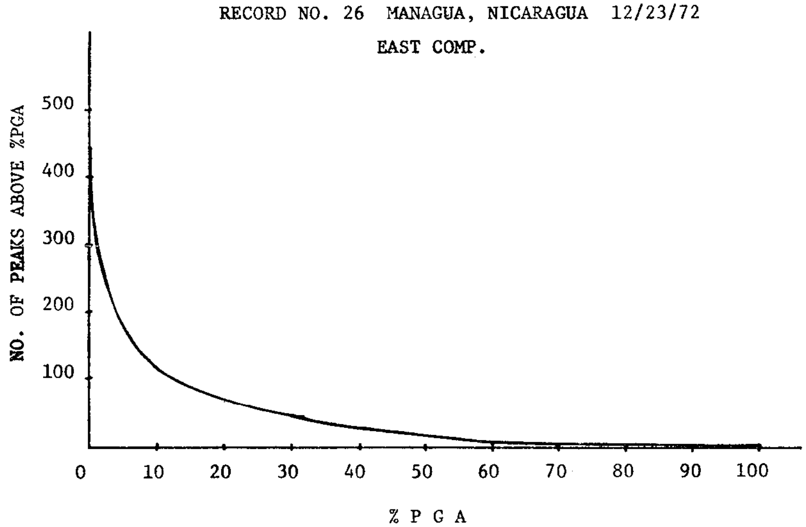

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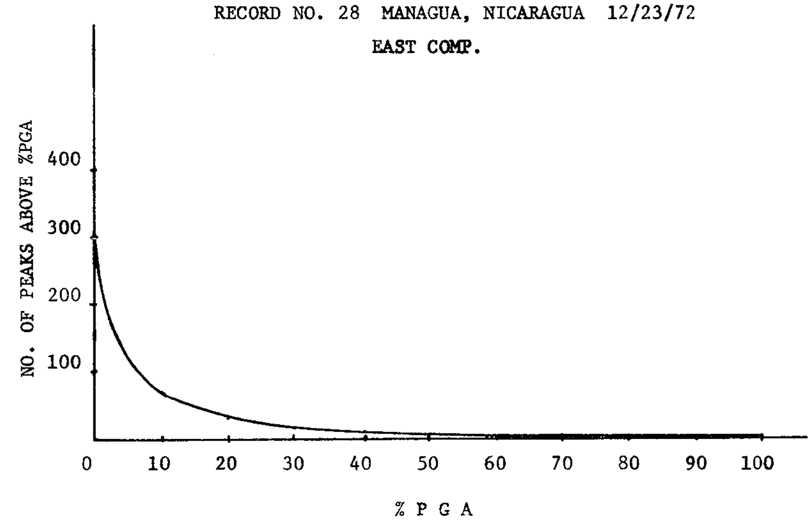

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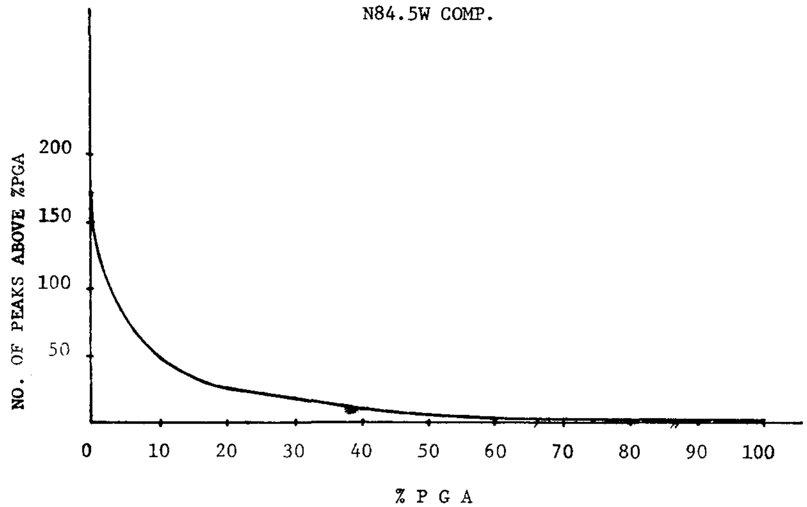
N84.5W COMP.



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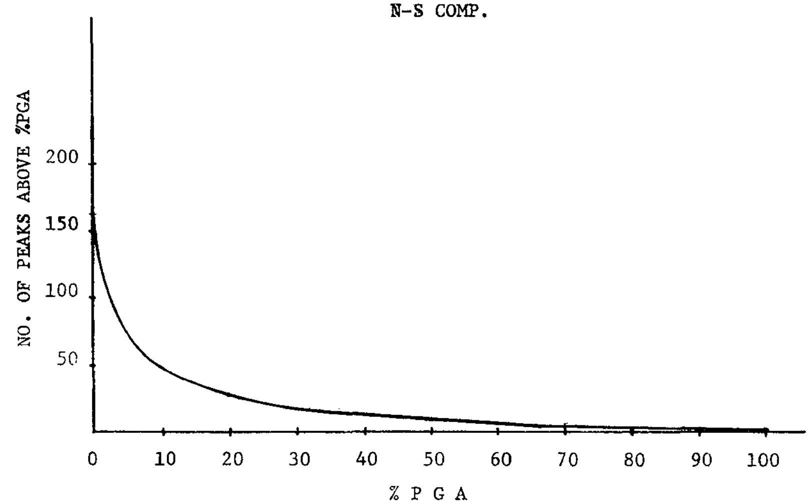
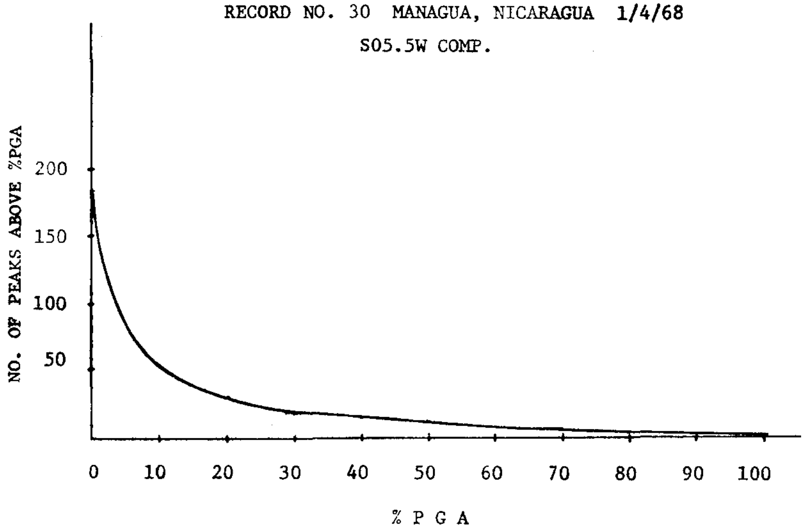
N-S COMP.



RECORD NO. 30 MANAGUA, NICARAGUA 1/4/68
S05.5W COMP.



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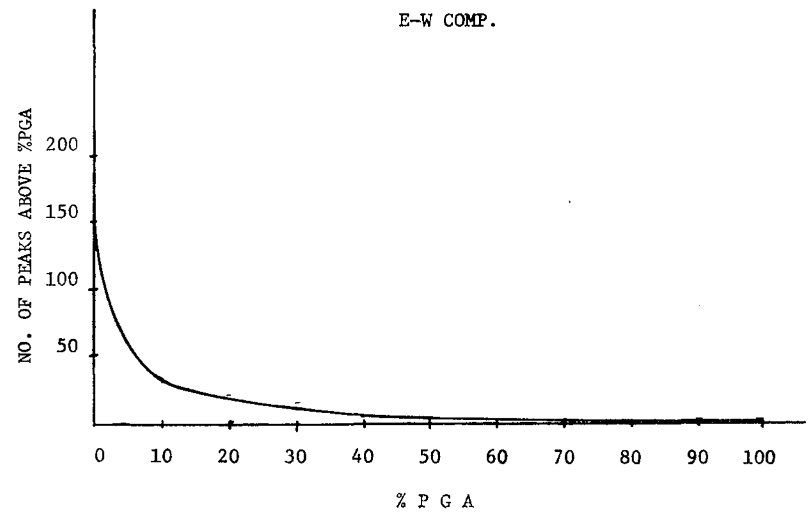
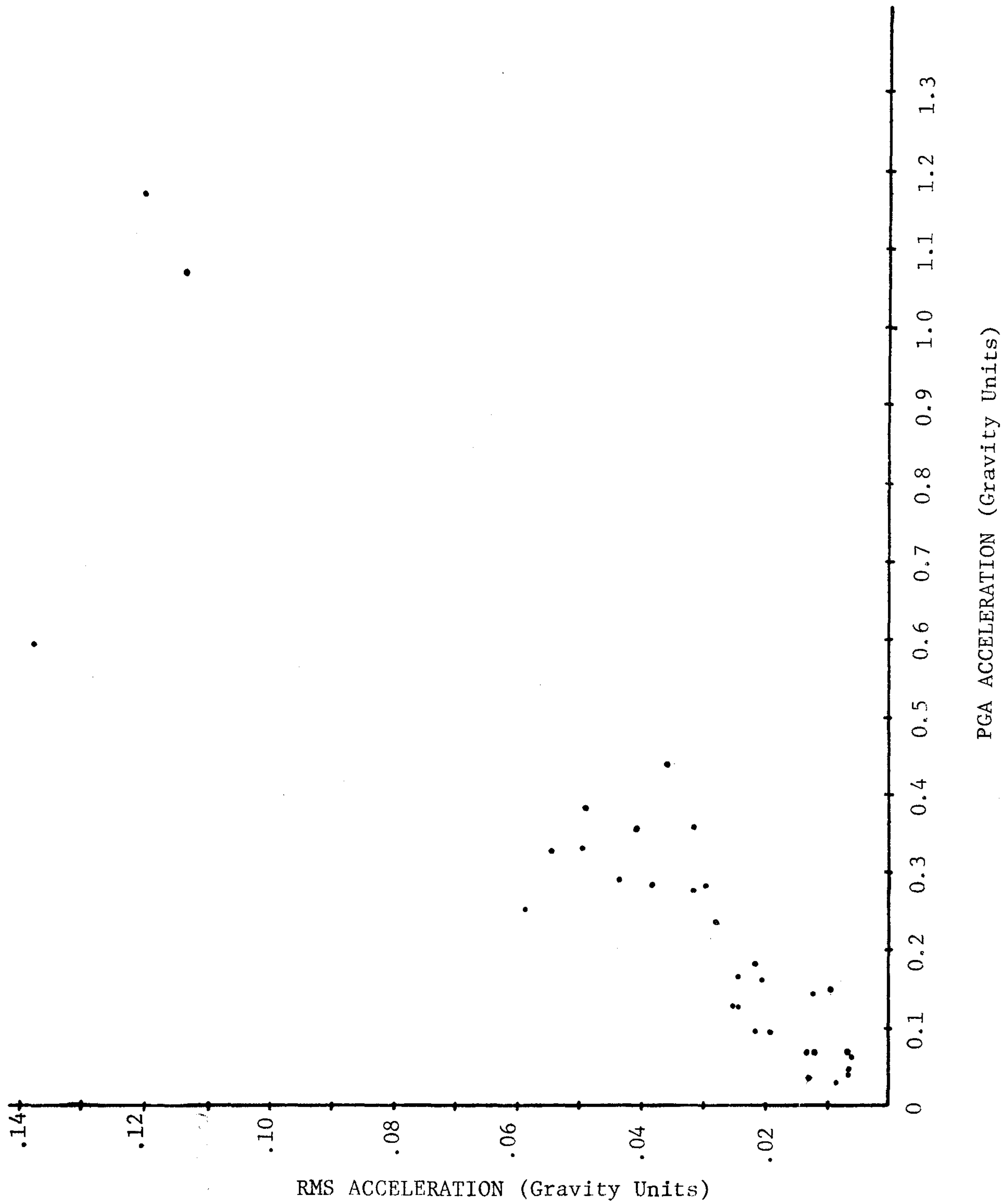
E-W COMP.





PGA ACCELERATION (Gravity Units)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
RMS ACCELERATION (Gravity Units)
.02
.04
.06
.08
.10
.12
.14

APPENDIX G

RISK DATA*

*Data taken from "An Assessment of Accident Risks in U. S. Commercial Nuclear Power Plants". Draft Report Wash 1400 U. S. Atomic Energy Commission August 1974

INDIVIDUAL RISK OF ACUTE FATALITY BY VARIOUS CAUSES

(U.S. Population Average 1969)

Accident Type	Total Number for 1969	Approximate Individual Risk Acute Fatality Probability/yr[1]
Motor Vehicle	55,791	3×10^{-4}
Falls	17,827	9×10^{-5}
Fires and Hot Substance	7,451	4×10^{-5}
Drowning	6,181	3×10^{-5}
Poison	4,516	2×10^{-5}
Firearms	2,309	1×10^{-5}
Machinery (1968)	2,054	1×10^{-5}
Water Transport	1,743	9×10^{-6}
Air Travel	1,778	9×10^{-6}
Falling Objects	1,271	6×10^{-6}
Electrocution	1,148	6×10^{-6}
Railway	884	4×10^{-6}
Lightning	160	5×10^{-7}
Tornadoes	91[1]	4×10^{-7}
Hurricanes	93[2]	4×10^{-7}
All Others	8,695	4×10^{-5}
All Accidents (Table 6.1)		6×10^{-4}
Nuclear Accidents (100 reactors)	0	3×10^{-9}*

[1]Based on total U.S. population, except as noted.

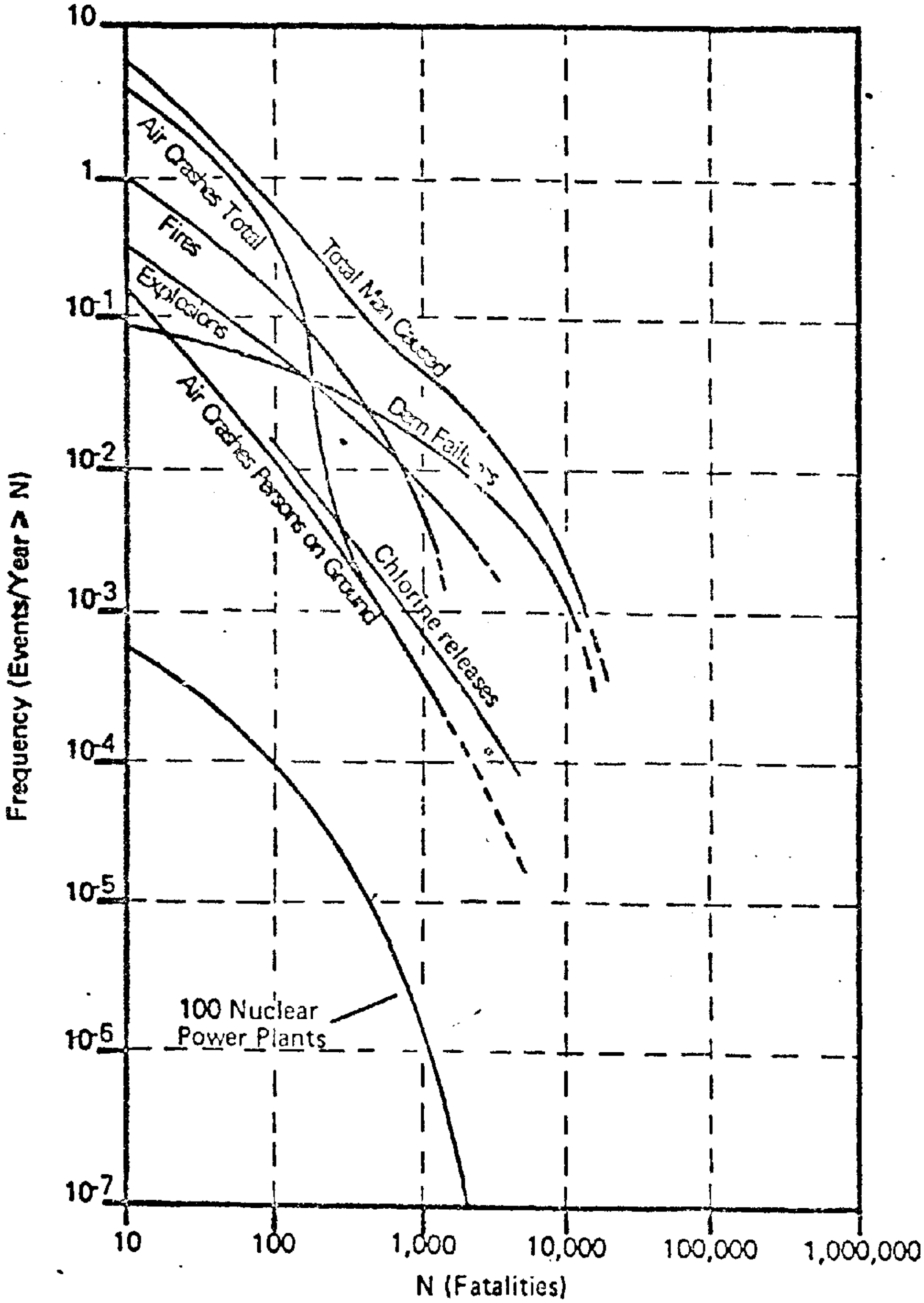
[2](1953-1971 avg.)

[3](1901-1972 avg.)

CONSEQUENCES OF MAJOR U.S. EARTHQUAKES (1900 - 1972)[1]

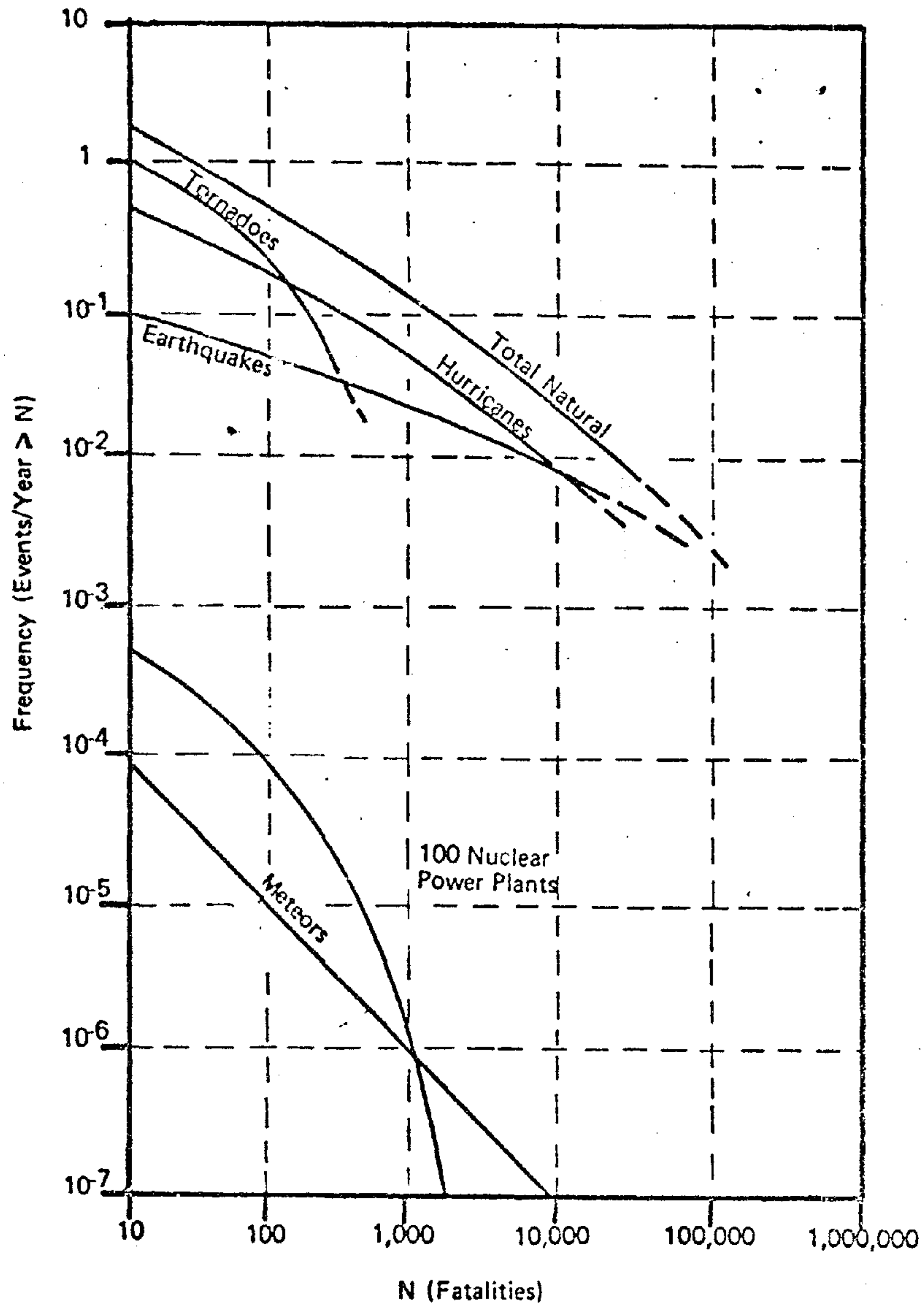
Date	Place	Fatalities	Damage (millions)
1906	San Francisco, California	-750	400
1925	Santa Barbara, California	13	6.5
1933	Long Beach, California	102	45
1935	Helena, Montana	4	3.5
1940	Imperial Valley, California	9	5.5
1949	Olympia, Washington	8	20
1952	Kern County, California	11	48
1954	Eureka, California	1	1
1957	San Francisco, California	0	1
1959	Hebgen Lake, Montana	28	4
1964	Anchorage, Alaska	125	310
1965	Puget Sound, Washington	6	12
1969	Santa Rosa, California	0	7
1971	San Fernando, California	58	480

[1]"A Study of Earthquake Losses in the Los Angeles, California Area," prepared by NOAA for the Federal Disaster Assistance Administration.

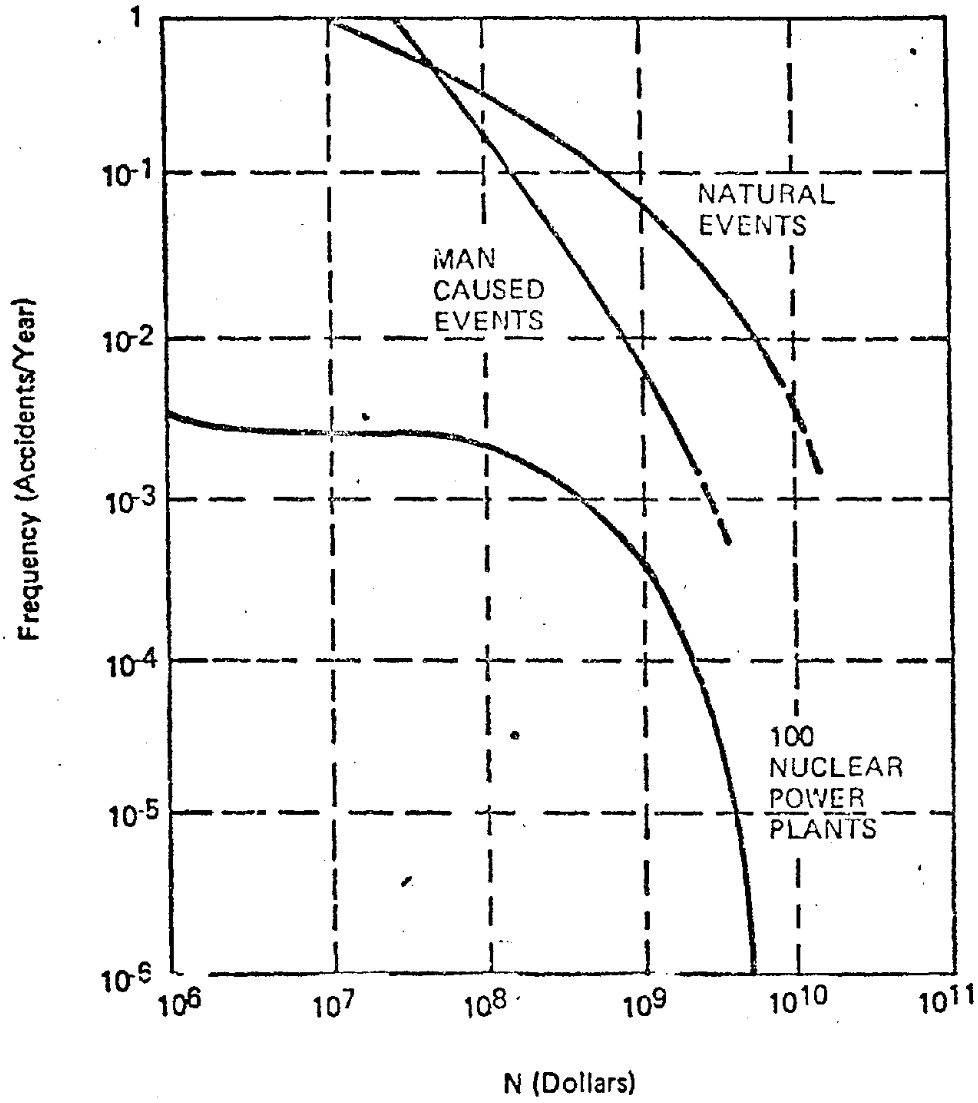


Frequency of Man-Caused Events with Fatalities Greater than N*

*Fatalities due to auto accidents are not shown because data are not available. Auto accidents cause about 50,000 fatalities per year.

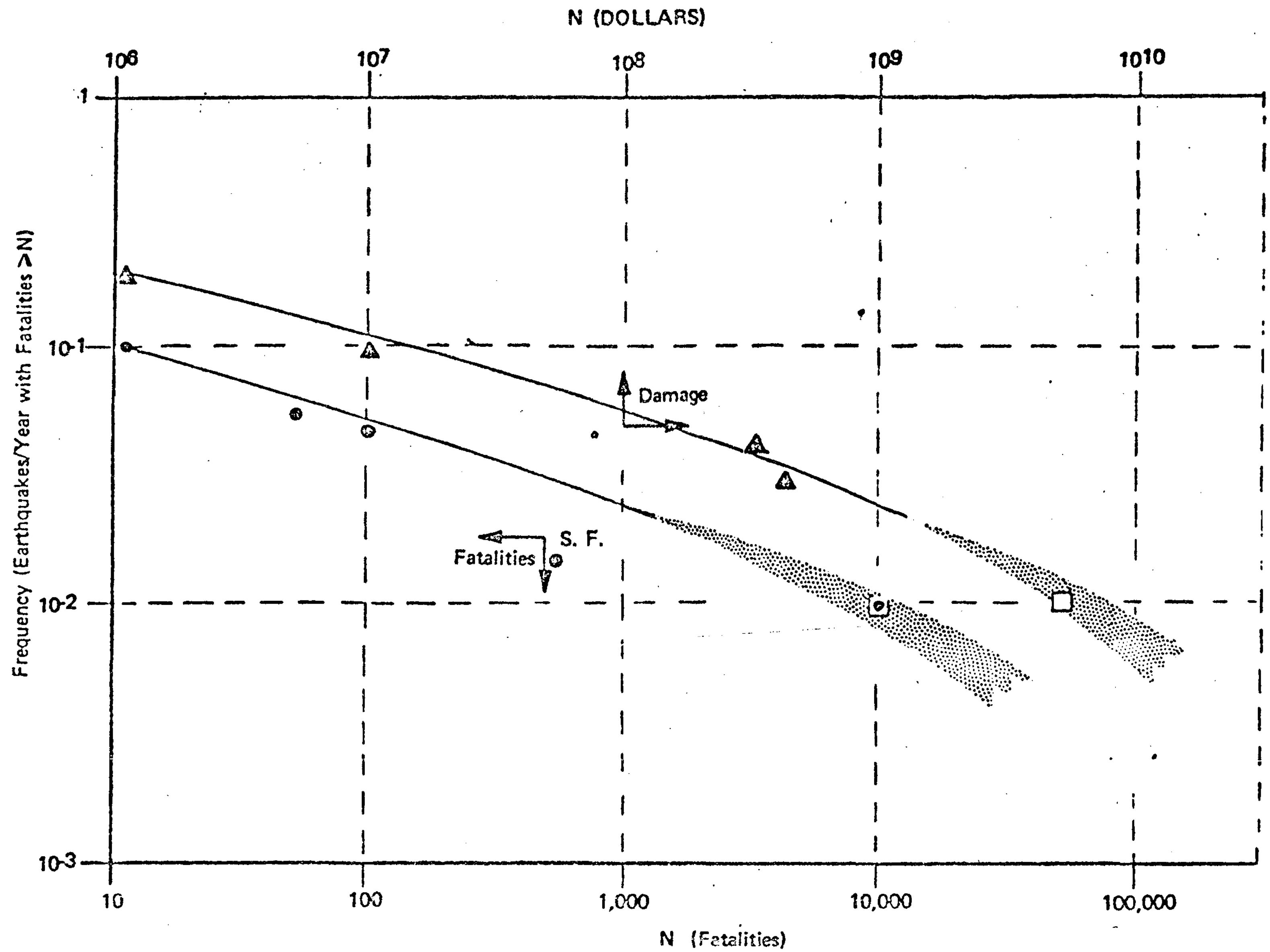


Frequency of Natural Events with Fatalities Greater than N*



Freauency of Accidents With Property Damage Greater than N*

*Property damage due to auto accidents is not included because data are not available. Auto accidents cause about $15 billion damage each year.



Frequency of Earthquakes with Consequences Greater Than N