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An overview of the methodology and results of a proposed design procedure based upon seismic risk analysis is provided. Seismic hazard conditions of Nicaragua are presented in the form of an iso-contour map, and acceleration zone graphs for the principal cities. The map and the graphs provide the peak ground acceleration (PGA) values of earthquakes for a given return period or having a given probability of being exceeded during a given economic life in years. Topics covered in this report are: seismic hazard zoning; design objectives; definition of structure importance or use classes; definition of the type of structural system; definition and formation of the design spectra; the structural design procedure based on the response spectrum method; an equivalent static force method; and design examples. To demonstrate the use of the design methodology developed in this project, two design examples are given. Example I is a two-story two-bay ductile moment resisting frame. Example II is an existing building in Los Angeles which was originally designed according to the 1964 Los Angeles City Building Code. This building survived the 1971 San Fernando earthquake with minor damage. The building is a fourteen story reinforced concrete shear wall building.

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A STUDY OF SEISMIC RISK FOR NICARAGUA PART II SUMMARY

by

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1.a

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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 dèsign 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

INTRODUCTION

The purpose of this summary is to provide an overview of the methodology and results of a proposed design procedure based upon seismic risk analysis. Topics to be covered are:

- Seismic Hazard Zoning
- Design Objectives
- Definition of Structure Importance or Use Classes
- Definition of the Type of Structural System
- Definition and Formation of the Design Spectra
- The Structural Design Procedure Based on the Response Spectrum Method
- An Equivalent Static Force Method
- Design Examples

In this brief overview, the important terms and concepts will be referenced to their relevant chapters in the final report " A Study of Seismic Risk for Nicaragua, Part II, Commentary," published as a technical report No. 12A, The John A. Blume Earthquake Engineering Center, Stanford University, Stanford, California, March 1976. (That report will be referred to as the "Commentary" report.)

SEISMIC HAZARD ZONING

The description of the seismic hazard conditions of Nicaragua is in the form of an iso-contour map, figure S-1, and acceleration zone graphs for the principal cities, figure S-2. The map and the graphs (for better precision) provide the peak ground acceleration values PGA of earthquakes for a given return period or having a given probability of being exceeded during a given economic life in years. The development of the seismic hazard map and graphs is given in reference 1 and in chapter II of the "commentary."

DESIGN OBJECTIVES

For a given economic life of a structure, an adequate seismic design should provide acceptable reliabilities (as measured by a low acceptable risk) against:

(1) excessive damage due to a moderate or <u>damage level</u> earthquake as represented by a PGA of A_D

(2) condemnation due to a major or <u>condemnation level</u> earthquake as represented by a PGA of A_C

(3) collapse due to a catastrophic earthquake.

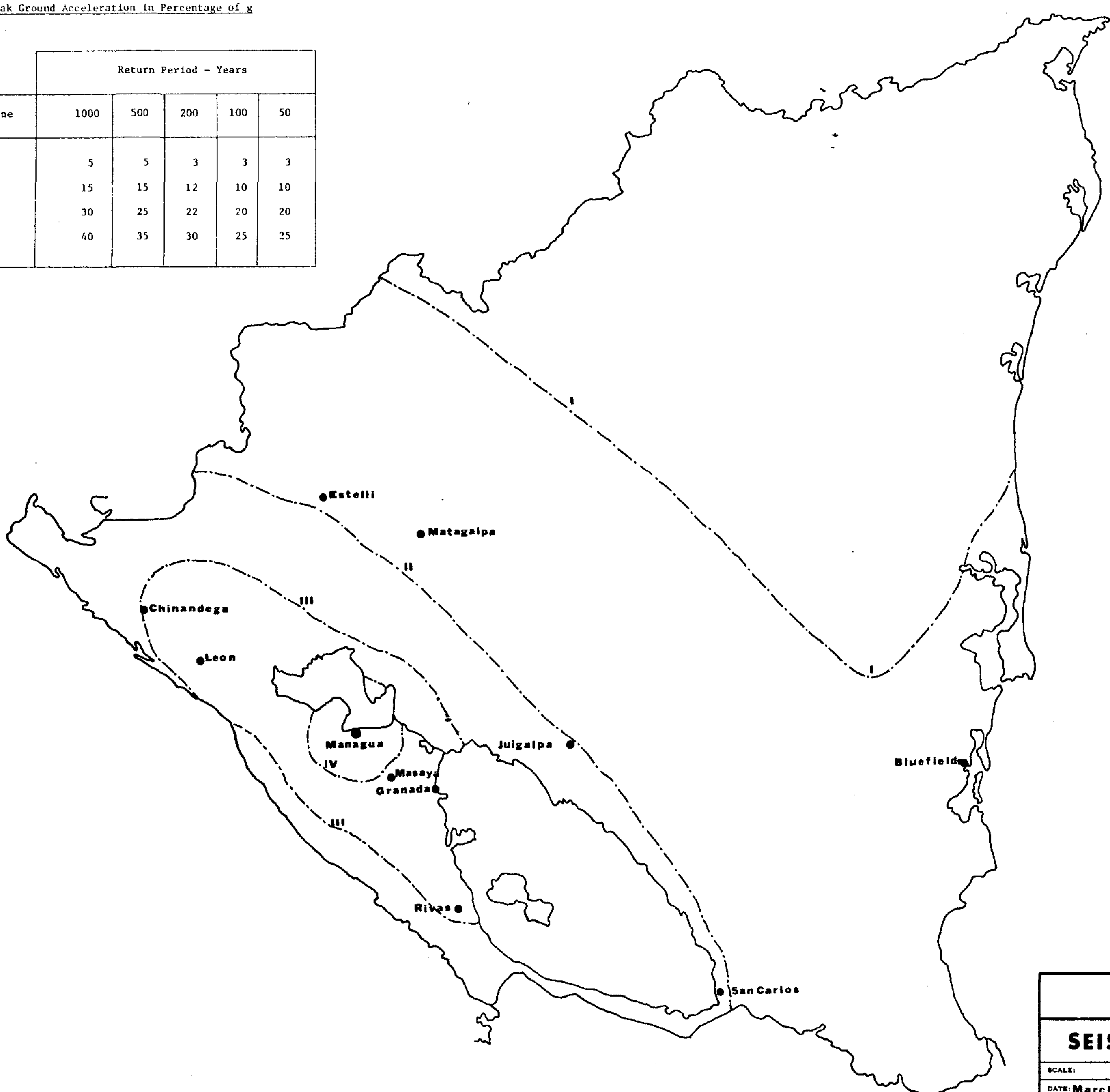
The value of the acceptable reliabilities of protection against each level of earthquake depends on the use class or importance of the structure.

DEFINITION OF STRUCTURE USE CLASS AND CORRESPONDING RISK LEVELS

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.

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



NICARAGUA

SEISMIC HAZARD MAP

SCALE:	FIGURE S-1	DRAWN BY
DATE: March, 1976		REVISED

Stanford University, Dept. of Civil Engineering

Stanford, California 94305

DRAWING NUMBER

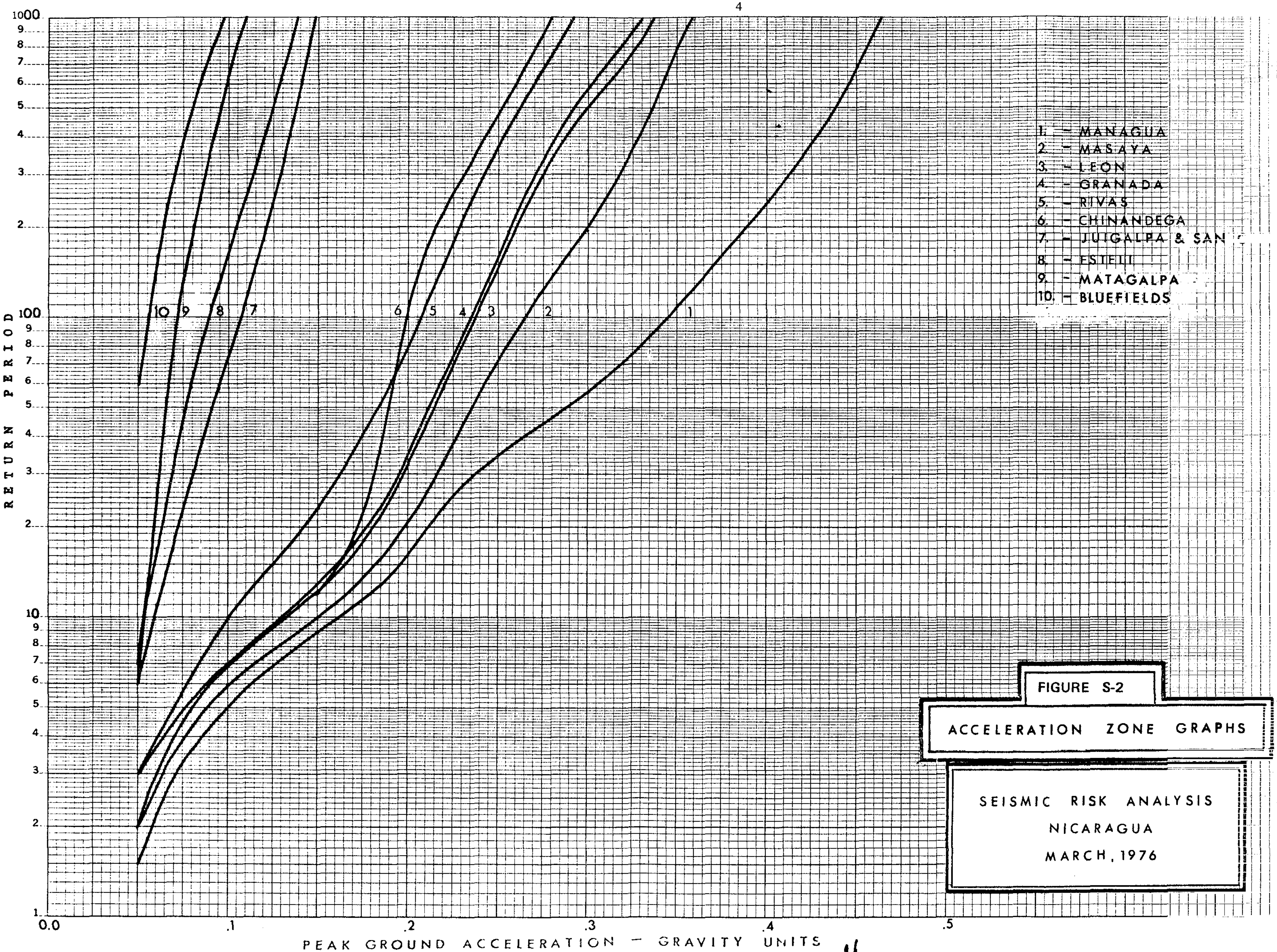


FIGURE S-2

ACCELERATION ZONE GRAPHS

SEISMIC RISK ANALYSIS
NICARAGUA
MARCH, 1976

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 following values are suggested for the economic lives, return periods, and acceptable risks for each use class.

Table S-1. 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

Corresponding to the values suggested in Table S-1, the risk levels for different classes are:

Class 1

(i) Risk P_C of exceeding condemnation level loading (having a PGA of A_C) per year = 0.001

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

(ii) Risk P_D of exceeding damage level loading (having a PGA of A_D) 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.

Example values of the peak ground accelerations A_D and A_C, at sites in Managua and Leon, are given in Table S-2, S-3, S-4 and S-5. These are based on structure lives of 20, 50 and 100 years, and on the acceptable risk values P_D and P_C corresponding to the structure Use Class. 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. See chapter V of the Commentary.

Table S-2. Managua Region

Suggested Damage "Risk" Levels P_D

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

Table S-3. Managua Region

Suggested Condemnation "Risk" Levels P_C

Use 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 S-4. Leon Region

Suggested Damage "Risk" Levels P_D

Use 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 S-5. Leon Region

Suggested Condemnation "Risk" Levels P_C

Use 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

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.

DEFINITION OF STRUCTURAL SYSTEM TYPE

The definition of the basic types of lateral force resisting systems is essentially the same as the K-factor descriptions provided in the 1973 Uniform Building Code. However, in order to better represent the particular qualities or deficiencies of a given structure, a grading system is used for each K-factor type. The grades of A, B, or C are assigned according to the quality or degrees of redundancy, symmetry, accuracy of analysis, past performance record, and construction quality control. The A grade represents excellent qualities and merits a lower design value than systems with B or C grades which have good and fair qualities respectively. The structure type and grading system is discussed in Chapter VIII of the Commentary.

Each structure type (such as K = 1.00B) has its particular structural damping β_T, damage deformation factor d_T, and spectral confidence level $(1 + k_T V_S)$, where V_S is the coefficient of variation of the spectral shape. These type characteristics are given in Table 10-1 of Chapter X from the Commentary and are used to form the appropriate design spectra. Table 10-1 is repeated here as Table S-6. Table S-7 gives numerical values for an example site and for class 2 structures.

Table S-6

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 S-7

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
= $0.8\frac{H}{T}$ for $T > 0.8$ sec

For soft sites

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.

These design spectra are used and formed as follows:

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

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

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 of the Commentary.

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 of the Commentary.

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 of the Commentary.

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

$(1 + k_T V_S)$ = 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 of the Commentary.

The k_T value depends on the quality or grading of A, B, or C of a given structural system. See Figure 4-5 of the Commentary for the 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.

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 verified at the SRSS modal deformation response to the Damage Deformation Spectrum.

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

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

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

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

DESIGN PROCEDURE RULES

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

1. Given a Use class of the structure (Table S-1) 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 of the Commentary). The appropriate design spectra can be constructd 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 S-6).
2. Formulate the linear elastic structure model and determine mode shapes and periods 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 DDS and calculated as the SRSS of the individual modal drifts shall not exceed 1% of the story height. This restriction should be for damage control.
5. The member design procedure has produced known values for the individual member resistance values R_u, where R_u > (D + 0.4L) + E; R_u > 0.8(D + E); R_u > 1.7(D + L) 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 10-6 of the Commentary).

$$\mu_C = \frac{(D + 0.4L + E'_C)}{R_u}$$

$$\text{or } \frac{0.8D + E'_C}{R_u} .$$

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 Beam Joints = 5

Ductile Concrete Beam Joints = 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)

EQUIVALENT STATIC FORCE METHOD

Seismic loads for ultimate strength design are to be calculated from the following base shear

$$V = ADBM_N \qquad \text{S-4}$$

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 of the Commentary).

For medium to hard soil site conditions -

$D = 2$ for $T \leq 0.5$ secs.

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

For soft soil site conditions -

$D = 2$ for $T \leq 0.8$ secs.

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

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

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

$$B = R\,\frac{1}{d_T}\,(1 + k_T V_S) \qquad \text{S-5}$$

where d_T, R, k_T and V_S are discussed in the Commentary. Example numerical values are given in Table S-6.

M_N = Structural mass = W_N/g .

$$W_N = W_D + 0.4W_L \; . \qquad \text{S-6}$$

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 of the Commentary).
- 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 in Chapter X, but where the E_C' value is given by

$$\frac{A_C}{A_D} \cdot d_T \cdot E \qquad \text{S-7}$$

where E is the member seismic load due to the base shear equation S-4.

DESIGN EXAMPLES

To demonstrate the use of the design methodology developed in this project, two design examples are given in the following pages.

Example I is a two story two bay ductile moment resisting frame. For this example, a 0.67B type of grading is assumed. It is also assumed that the structure is located in Managua and the use class is 2.

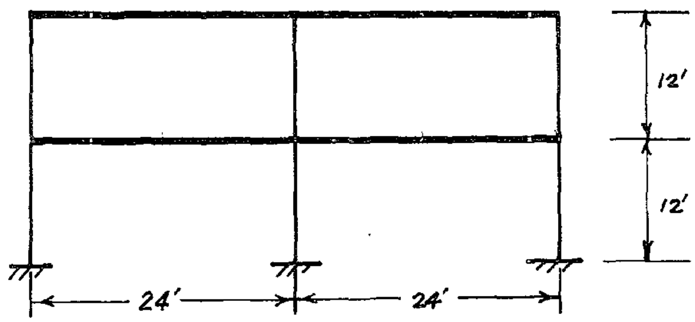
Example II is an existing building in Los Angeles which was originally designed according to the 1964 Los Angeles City Building Code. This building survived the 1971 San Fernando earthquake with minor damage. The building is a fourteen story reinforced concrete shear wall building. Again, it is assumed for this example that the building is located in Managua and belongs to use class 2. In terms of structural type, it is given a 1.33A type. The following calculations and explanations are self-explanatory.

EXAMPLE I

Concrete Ductile Moment Resisting Space Frame

Managua, Class 2

Type 0.67B



$K_2 = 60K/in$

$K_1 = 100K/in$

Figure S-3

Given: Roof: $W_D = 60K$, $W_L = 30K$

Floor: $W_D = 100K$, $W_L = 60K$ (basic LL)

Find: Story forces, story shears, overturning moments and deflections.

Proposed Response Spectrum Method (SRSS)

$$[K] = \begin{bmatrix} 160 & -60 \\ -60 & 60 \end{bmatrix} K/in$$

assumes that girders are very stiff compared to columns.

$$[M] \quad \begin{bmatrix} 0.321 & 0 \\ 0 & 0.186 \end{bmatrix} K\ sec^2/in$$

from $W_D + 0.4W_L$:

$W_1 = 124K$

$W_2 = 72K$

Periods:

$$\begin{vmatrix} 160 - 0.32/\omega^2 & -60 \\ -60 & 60 - 0.186\omega^2 \end{vmatrix} = 0$$

$\omega_1 = 12.23$, $T_1 = 0.514$ sec

$\omega_2 = 25.91$, $T_2 = 0.242$ sec

Mode Shapes:

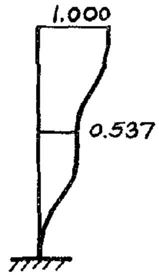
1st Mode: $\phi_{21} = 1$

$$- 60\phi_{11} + 60 - 0.186\omega_1^2 = 0 \ \ldots \ \phi_{11} = 0.537 \ .$$

2nd Mode: $\phi_{22} = 1$

$$- 60\phi_{12} + 60 - 0.186\omega_2^2 = 0 \ \ldots \ \phi_{12} = - 1.082 \ .$$

1st Mode:



2nd Mode:

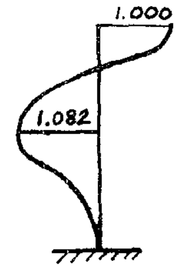


Figure S-4

<u>Modal Forces:</u> $$\alpha_m = \frac{\left|\sum_{i=1}^{N} W_i \phi_{im}\right|}{\sum_{i=1}^{N} W_i\ \phi_{im}^2}$$

Story	St. Weight	Mode 1		Mode 2	
i	W_i	$W_i\phi_{i1}$	$W_i\phi_{i1}^2$	$W_i\phi_{i2}$	$W_i\phi_{i2}^2$
1	124	66.59	35.76	-134.17	145.17
2	72	72.0	72.0	72.0	72.0
Σ	196	138.59	107.76	-62.17	217.17

$$\alpha_1 = \frac{138.59}{107.76} = \underline{1.286}\ , \qquad \alpha_2 = \frac{62.17}{217.17} = \underline{0.286}$$

Mass Participation Factors:

$$\bar{\alpha}_1 = \frac{138.59^2}{107.76 \times 196} = \underline{0.909}, \qquad \bar{\alpha}_2 = \frac{62.17^2}{217.17 \times 196} = \underline{0.091}$$

Mode 1: $T_1 = 0.514$ sec.

For Managua, use class 2: $A_g = 0.35g$;

K = 0.67 structure, Grade BL $\frac{1}{d_T}(1 + k_T V_S) = 0.40$

For T = 0.514 sec., hard soil: (MDAF) = 1.946

$(DFS)_1 = 0.7 \times 0.35 \times 1.946 \times 0.40 = 0.191g$

Story Forces: $F_{j1} = 0.191 \times \alpha_1 W_j \phi_{j1}$

$F_{11} = 0.191 \times 1.286 \times 124 \times 0.537 = 16.4K$

$F_{21} = 0.191 \times 1.286 \times 72 \times 1.0 = \underline{17.7K}$

base shear $V_1 = \underline{34.1K}$

Check: $V_1 = 0.191 \times \bar{\alpha}_1 W = 0.191 \times 0.909 \times 196 = \underline{34.1K}$

Overturning Moments: $M_{21} = 17.7 \times 12 = \underline{212Kft}$

$M_{11} = 212 \times 34.1 \times 12 = \underline{621Kft}$

Story Deflections: (at design level, DFS)

$$\delta_{j1} = 0.191 \frac{1}{\omega_1^2} g\alpha_1 \phi_{j1}$$

$$\delta_{11} = 0.191 \frac{386.4}{149.6} 1.286 \times 0.537 = \underline{0.341}''$$

$$\text{check: } \delta_{11} = \frac{34.1}{100} = \underline{0.341}''$$

$$\delta_{21} = 0.191 \frac{386.4}{149.6} \times 1.286 \times 1.0 = \underline{0.635}''$$

$$\text{check: } \delta_{21} = 0.341 \times \frac{17.7}{60} = \underline{0.635}''$$

Mode 2: $T_2 = 0.242$ sec , $(DFS)_2 = 0.7 \times 0.35 \times 2.0 \times 0.40 = 0.196g$

Story Forces: $F_{12} = 0.196 \times 0.286 \times 124 \times (-1.082) = -\ 7.5K$

$F_{22} = 0.196 \times 0.286 \times 72 \times 1.0 = \underline{4.0K}$

base shear $V_2 = \underline{-\ 3.5K}$

check: $V_2 = 0.196 \times 0.091 \times 196 = \underline{3.5K}$

Overturning Moments: $M_{22} = 4.0 \times 12 = \underline{48\ Kft}$

$M_{12} = 48 - 3.5 \times 12 = \underline{6\ Kft}$

Story Deflections: $\delta_{12} = -\frac{3.5}{100} = -\ \underline{0.035}''$

$$\delta_{22} = -\ 0.035 \times \frac{4.0}{6\ 0} = \underline{0.032}''$$

Summary:

	Mode 1		Mode 2		SRSS $= \sqrt{(M1)^2 + (M2)^2}$	
	Story 2	Story 1	Story 2	Story 1	Story 2	Story 1
Story Force (K)	17.7	16.4	4.0	- 7.5	18.2	18.0
Story Shear (K)	17.7	34.1	4.0	- 3.5	18.2	34.3
Overt. Mom. (Kft)	212	621	48	6	217	621
Story Defl. (in)	0.635	0.341	0.032	- 0.035	0.636	0.343

Check Story Drift: $\frac{\delta}{h} d_T = \frac{0.343}{144} \times 3 = 0.007 < 0.01$ ... o.k.

EXAMPLE II

Fourteen Story Reinforced Concrete Shear Wall Building

Managua, Class 2

Type 1.33A

INTRODUCTION

An actual building example is presented in order to illustrate the design procedure for the proposed response spectrum method. The building selected for the example has been designed by methods presently used or proposed in the United States. The example Building (ATC-2 Building #4)* was originally designed in accordance with the 1964 Los Angeles City Building Code and was updated in the ATC-2 Report to fulfill the requirements of the 1973 Uniform Building Code. This design example and the resulting comparison of the seismic shear forces, overturning moments, and member design forces resulting from the 1973 UBC, the 1974 SEAOC and the proposed response spectrum method should prove helpful for an evaluation of the proposed method. However, general conclusions should be made with caution since modal participation may vary widely for different structural configurations, and the load factors proposed for the Nicaragua Code will have dissimilar effects on different structural systems.

*ATC-2, "An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings." Applied Technology Council (sponsored by NSF and NBS) 171 Second Street, San Francisco, California 94105

1. Design for Strength

1a. Design Force Spectrum, DFS

For each natural period T_m of the significant modes of the structure the value of DFS is given by

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

where

m is the number of the natural mode

R is taken as 0.7

A_D is the PGA value of the ground acceleration at the damage threshold level for the structure use class

d_T, k_T are coefficients pertaining to the type of structural system (see Table 10-1 of the Commentary).

$(MDAF)_m$ is the dynamic amplification factor for the period T_m.
(MDAF) is presented for intermediate to dense soils in Fig. S-5.

1b. Computation of Seismic Forces

1. Compute the lateral forces at each floor level for all significant modes, i.e.,

$$F_{jm} = \frac{DFS_m}{g} \alpha_m W_j \phi_{jm}$$

where

g is the acceleration of gravity

$$\alpha_m = \frac{\left| \sum_{i=1}^{n} W_i \phi_{im} \right|}{\sum_{i=1}^{n} W_i \phi_{im}^2}$$

n is the number of stories in the structure

W_j is the weight of story j, including the total dead load and 40% of the live load

ϕ_{jm} is the amplitude of the mth mode at the jth floor.

2. By methods of statics compute the shear forces and overturning moments at each floor level for all significant modes. Check the resulting base shear in each mode by the equation

$$V_m = \frac{DFS_m}{g} \bar{\alpha}_m \sum_{i=1}^{n} W_i$$

where

$$\bar{\alpha}_m = \frac{\left(\sum_{i-1}^{n} W_i \phi_{im}\right)^2}{\sum_{i=1}^{n} W_i \phi_{im}^2} \cdot \frac{1}{\sum_{i=1}^{n} W_i}$$

3. Compute the design shears, overturning moments and lateral story deflections by

$$V_j = \sqrt{\sum_m V_{jm}^2}$$

$$M_j = \sqrt{\sum_m M_{jm}^2} \frac{d_T}{d_{OT}}$$

$$\delta_j = \sqrt{\sum_m \delta_{jm}^2}$$

where

d_T/d_{OT} is the overturning moment reduction factor (see Table S-6).

1c. Design of Structural Members

All members need to be designed for the internal forces caused by

1. The effects of V_j and M_j plus the effects of D + 0.4L

2. The effects of $0.8V_j$ and $0.8M_j$ plus the effects of 0.8D.

Remarks:

The story shears shall be distributed to the elements of the lateral load resisting elements in proportion to their rigidities. Torsional effects shall be included wherever the center of mass does not coincide with the center of rigidity. This will in general require a three dimensional analysis. The torsional moments in each story shall be computed by multiplying the effective weight of the story (D + 0.4L) with a modified distance between the centers of mass and rigidity which is equal to the actual distance plus or minus 5% of the base dimension of the structure normal to the applied lateral load. This additional 5% torsion is necessary to account for accidental torsion caused by stiffness variations and unequal distribution of live loads.

The P-Delta effects shall be estimated by rational analysis and shall be included when of importance. In lieu of a more exact analysis, it can be estimated that the P-Delta effects will increase the story shears by the amount

$$\overline{V}_j = \frac{P_j(\delta_j - \delta_{j-1})}{h_j}$$

where

P_j is the sum of the axial column loads above level j.

δ_j is the lateral deflection of floor j.

h_j is the height of story j.

If D_j is larger than 5% of the story shear V_j, then $\bar{V}_j$ should be included in the story shear.

2. Design for Stiffness

The elements of the lateral load resisting system shall be designed for stiffness such that the lateral deflection between adjacent stories is smaller than one percent of story height under the actions of the damage level earthquake (DDS). This is equivalent to the requirement that the story drift index δ/h shall be less than $0.01/d_T$ under the actions of the DFS design level, where d_T is the damage deformation factor.

3. Design for Inelastic Deformation Demand

The inelastic deformation demand should be evaluated by either a detailed deformation analysis or an approximate method.

- The detailed analysis involves the evaluation of the elastic and inelastic deformations in the individual elements based on the total story deformation of the CDS load. The resulting inelastic member deformations provide the ductility demand values.
- The approximate method uses the relation for local ductility demand as

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

where R_u is the ultimate design capacity of the member and E'_C can be obtained from the DFS design forces E through the linear relationship

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

Stability of the structure at the CDS level should be investigated carefully; in particular, the P-Delta effects need to be investigated.

DESIGN EXAMPLE ATC-2 BUILDING #4

This building is discussed in detail in the report "An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings," published by the Applied Technology Council, and in the report "San Fernando, California, Earthquake of February 9, 1971," Volume I, Part B, U.S. Department of Commerce, NOAA.

The building is of particular interest insofar that it survived the 1971 San Fernando earthquake with minor damage, despite being designed for only half of shear forces and two thirds of the overturning moments specified in the 1973 UBC. Three strong-motion accelerographs were installed in the building at the ground-floor level, sixth-floor level and roof. The records indicated maximum ground accelerations occurring in the north-south direction with peaks of 26% g. A dynamic analysis of the building subjected to the recorded ground motion is reported in the second of the aforementioned references. This analysis indicated story forces, overturning moments and lateral deflections far in excess of the design values, however, actual damage was restricted to hairline cracks in the exterior north-south shear walls.

The building is a fourteen-story reinforced concrete shear wall building. Floors are framed with an 8-inch flat slab supported on columns at 19' x 20' bays and on bearing walls. Twelve-inch thick concrete shear walls on both major axes of the building provide the major lateral load resisting elements. The analysis reported in the ATC-2 report indicated that the flat slab system would carry up to 10% of the total lateral load in the east-west direction and up to 5% in the north-south direction. This contribution is included in the following study, that was carried out for the north-south direction of the building.

Since the flat slab system should not be considered as a complete vertical load carrying frame, the building was assigned a K value of 1.33. Since the detailing of the structure was done with great care and a dynamic analysis is carried out, the structure was assigned a grade A in the proposed Nicaragua code. All other requirements for grade A are fulfilled with this particular structural system.

The basic gravity load data for the structure are:

$$\text{Average DL} = 156 \text{ psf}$$

$$\text{LL} = 50 \text{ psf} .$$

Floor framing plans and transverse building section are shown on the next two pages (Figures S-6 and S-7).

<u>Proposed Nicaragua Code Spectrum Method:</u>

1. DFS

$$DFS_m = RA_D (MDAF)_m \frac{1}{d_T} (1 + k_T V_S)$$

where

$R = 0.70$

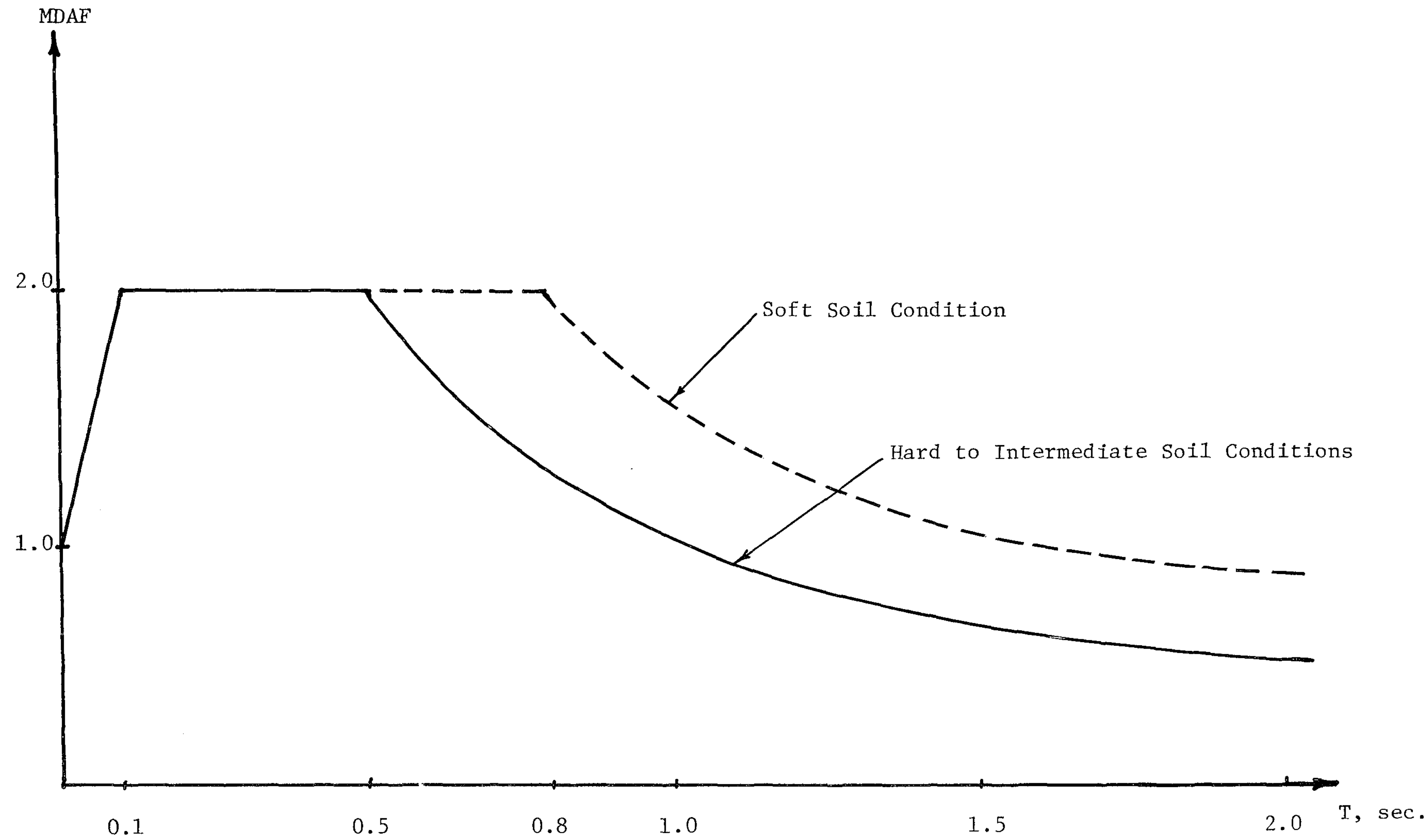
$A_D = 0.35g$ (Managua, use class 2)

$\frac{1}{d_T} (1 + k_T V_S) = 0.80$ (for $K = 1.33$, Grade A)

T_m from ATC-2 report; (MDAF) from Fig. 1

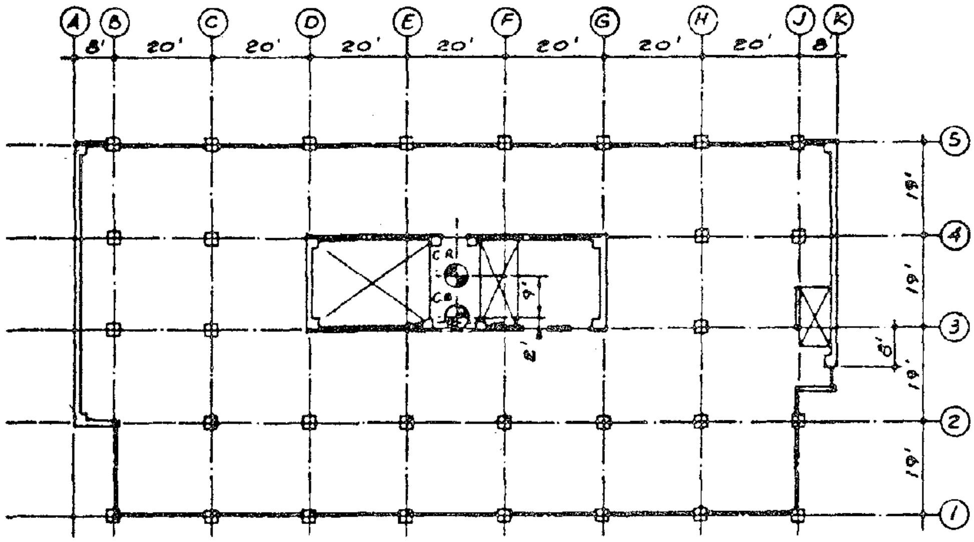
$T_1 = 0.677$ sec $\rightarrow$ $(MDAF)_1 \rightarrow 1.48$ $\quad DFS_1 = 0.290g$

$T_2 = 0.153$ sec $\rightarrow$ $(MDAF)_2 \rightarrow 2.00$ $\quad DFS_2 = 0.392g$

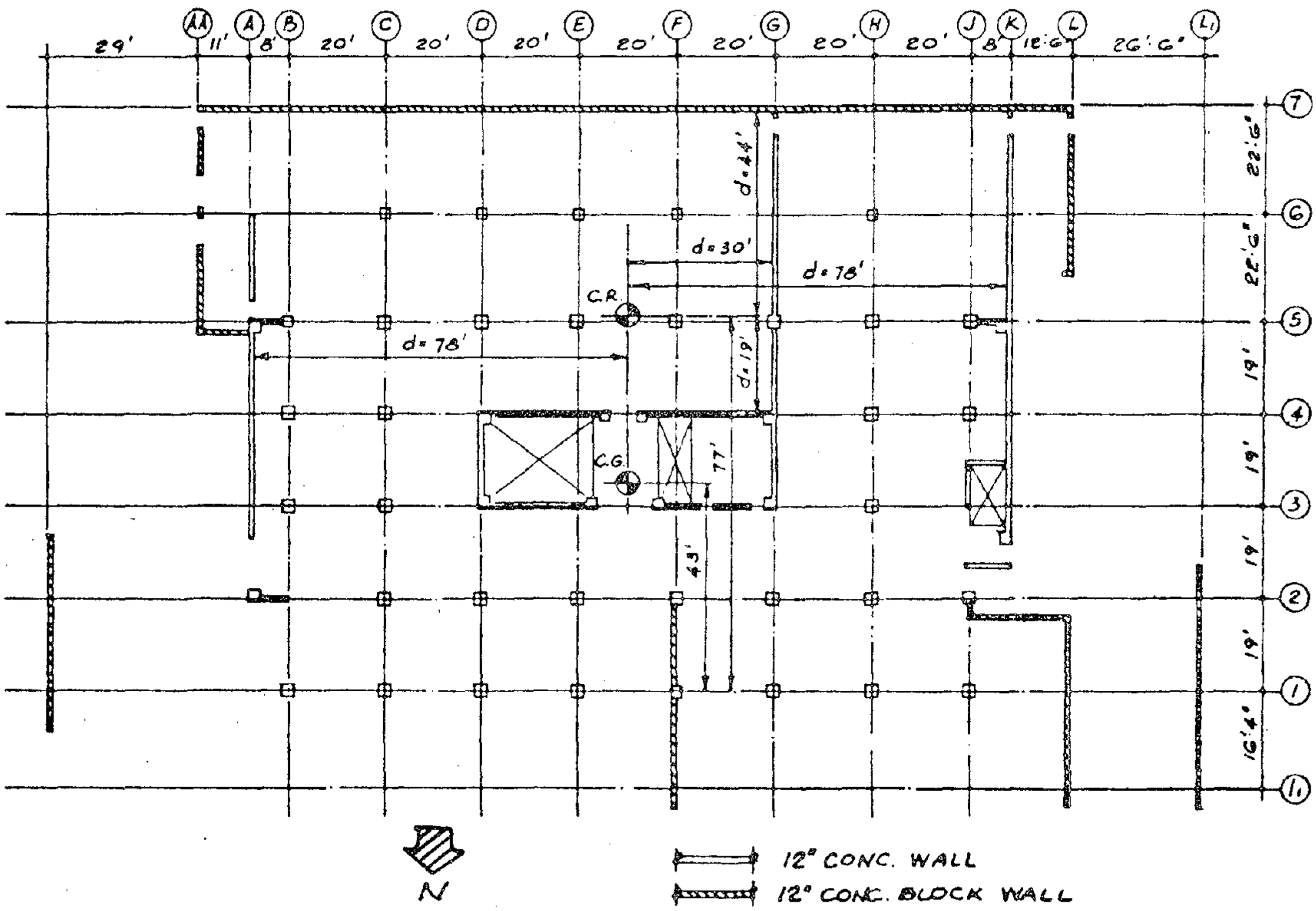


MDAF (for $\beta_T = 10\%$)

Figure S-5

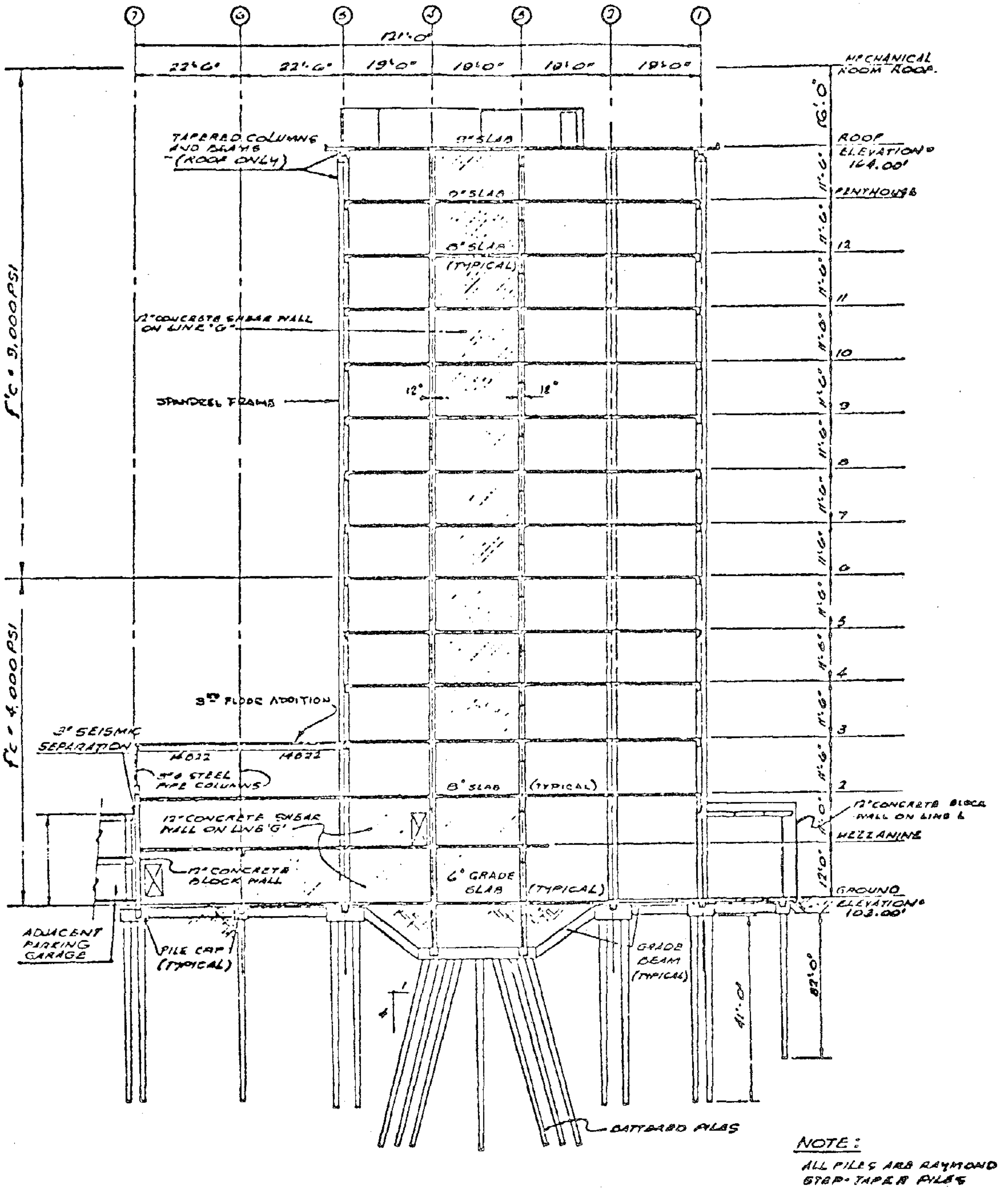


Typical Tower Plan



First Through Second Plan

Figure S-6



Transverse Building Section

Figure S-7

$T_3 = 0.072$ sec $\rightarrow (\mathrm{MDAF})_3 \rightarrow \mathrm{DFS}_3 = 0.392\mathrm{g}$

$T_4 = 0.048$ sec $\rightarrow (\mathrm{MDAF})_4 \rightarrow \mathrm{DFS}_4 = 0.392\mathrm{g}$

2. Computation of Seismic Forces:

$$F_{jm} = \frac{\mathrm{DFS}_m}{g} \alpha_m W_j \phi_{jm}$$

Mode shapes and effective weights from ATC-2.

Effective weights: $D + 0.4L = D + 0.4 \frac{50}{156} D = \underline{1.13D}$

$$\alpha_m = \frac{\left| \sum_{i=1}^{15} W_i \phi_{im} \right|}{\sum_{i=1}^{15} W_i \phi_{im}^2}$$

$$\alpha_1 = \frac{10,650}{5,395} = \underline{1.66} \qquad \alpha_2 = \frac{6,300}{6,130} = \underline{1.03}$$

$$\alpha_3 = \frac{4,170}{6,570} = \underline{0.63} \qquad \alpha_4 = \frac{3,250}{6,715} = \underline{0.48}$$

Mass Participation: $\bar{\alpha}_m = \frac{(\sum W_i \;\; _{im})^2}{\sum W_i \;\; ^2_{im}} \cdot \frac{1}{\sum W_i}$

$$\bar{\alpha}_1 = \frac{10,650^2}{6395 \times 30,148} = 0.588 \qquad \bar{\alpha}_2 = \frac{6300^2}{6130 \times 30,148} = 0.215$$

$$\bar{\alpha}_3 = \frac{4170^2}{6570 \times 30,148} = 0.088 \qquad \bar{\alpha}_4 = \frac{3250^2}{6715 \times 30,148} = 0.052$$

The mass participation for the first 4 modes is 0.588 + 0.215 + 0.088 + 0.052 = 0.943 = 94.3%, i.e., 5.7% of the mass vibrates in higher modes.

Story Forces, Story Shears and Overturning Moments

$$V_{jm} = \sum_{i=j}^{15} F_{im}$$

$$V_j = \sqrt{\sum_{m=1}^{4} V_{jm}^2}$$

$$\overline{M}_{jm} = \sum_{i=j}^{15} V_{im} h_i$$

$$\overline{M}_j = \sqrt{\sum_{m=1}^{4} \overline{M}_{jm}^2}$$

$$M_j = \overline{M}_j \cdot \frac{d_T}{d_{OT}} = \overline{M}_j \cdot \frac{1.50}{3.00} = 0.5\ \overline{M}_j$$

Modal Story Forces:

$$F_{j1} = 0.290 \times 1.66 \times W_j \phi_{j1} = 0.481\ W_j \phi_{j1}$$

$$F_{j2} = 0.392 \times 1.03 \times W_j \phi_{j2} = 0.404\ W_j \phi_{j2}$$

$$F_{j3} = 0.392 \times 0.63 \times W_j \phi_{j3} = 0.247\ W_j \phi_{j3}$$

$$F_{j4} = 0.392 \times 0.48 \times W_j \phi_{j4} = 0.18\ W_j \phi_{j4}$$

Modal Base Shears:

$V_{11} = 5142K$, $V_{12} = 2545K$

$V_{13} = 1038K$, $V_{14} = 615K$

SRSS Base Shear

$$V_1 = 5863K$$

SRSS Overturning Moment at Base:

$$M_1 = 320 \times 10^3 \text{ Kft}$$

The final story shears and overturning moments are shown in Figures S-8 and S-9. For comparison, story shears and overturning moments were also computed by methods presently used in U. S. engineering practice.

1973 UBC (SEAOC):

$$V = KCW$$

$$K = 1.33$$

Tower: $$T = \frac{0.05h_n}{\sqrt{D}} = \frac{0.05 \times 177}{\sqrt{76}} = 1.02 \text{ sec}$$

$$C = \frac{0.05}{\sqrt[3]{T}} = \frac{0.05}{\sqrt[3]{1.02}} = 0.0498$$

$$V = 1.33 \times 0.0498 \times 26{,}680 = \underline{1767K}$$

The effects of the base structure have been neglected in the ATC-2 calculations. Since the results of ATC-2 are used in these calculations, the same will be done here.

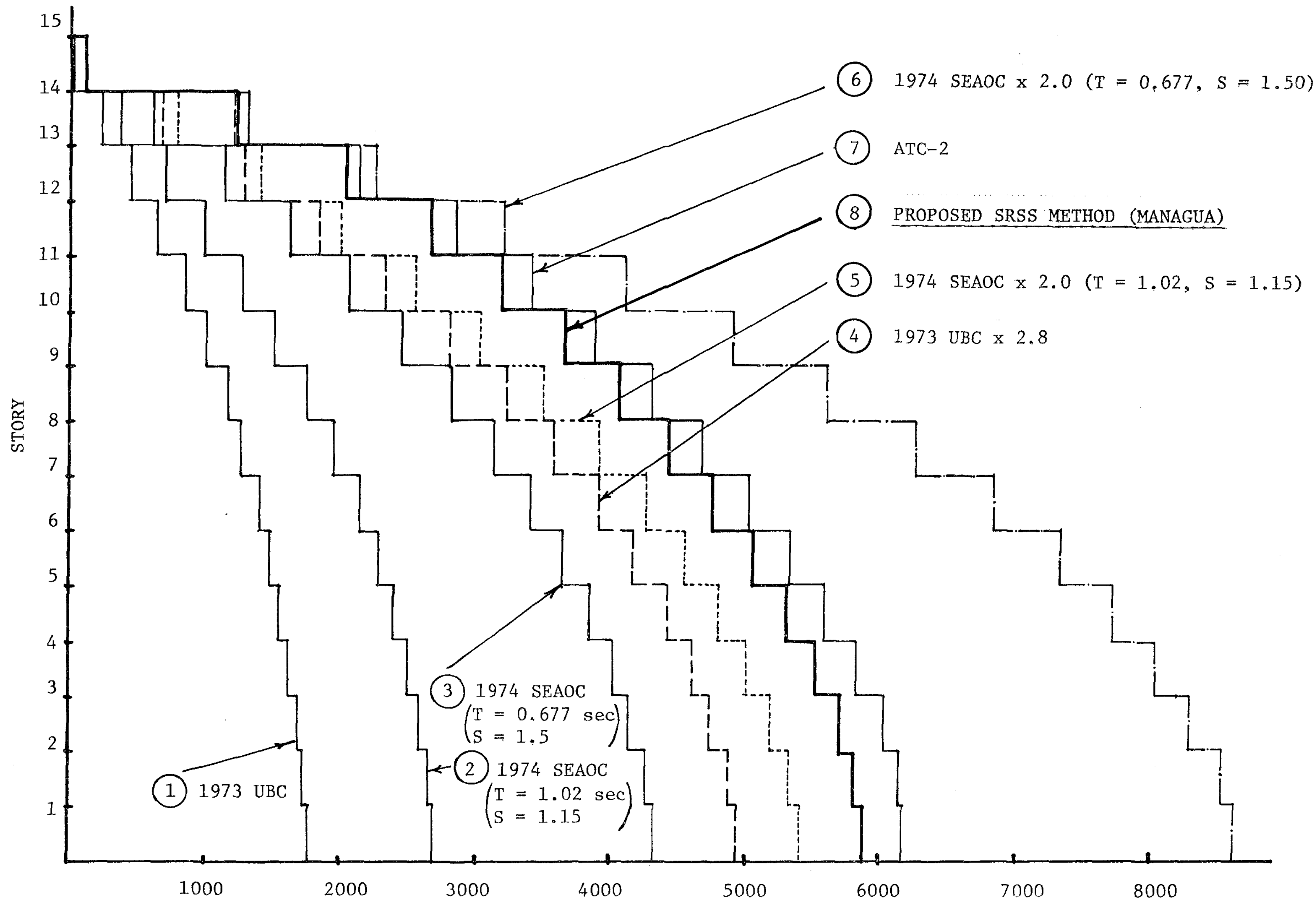
1974 SEAOC:

$$V = ZIKCSW$$

$$Z = 1.0$$

$$I = 1.0$$

$$K = 1.33$$



DESIGN EXAMPLE - STORY SHEARS
Figure S-8

1) 1973 UBC (SEAOC), no load factors.

2) 1974 SEAOC, Case a, $T = 1.02$ sec, $S = S_{min} = 1.15$, no load factors.

3) 1974 SEAOC, Case b, $T = 0.677$ sec, $S = S_{max} = 1.50$, no load factors.

4) 1973 UBC (SEAOC), with load factors 2.8 for shear and 1.4 for overturning.

5) 1974 SEAOC, Case a, with load factors 2.0 for shear and 1.4 for overturning.

6) 1974 SEAOC, Case b, with load factors 2.0 for shear and 1.4 for overturning.

7) ATC-2 strength design spectrum ($DFS_1 = 0.35g$, $DFS_2 = 0.44g$, $DFS_3 = 0.24g$, $DFS_4 = 0.24g$)

8) Proposed Nicaragua Code, SRSS.

DISCUSSION AND COMPARISON TO SEAOC RECOMMENDATIONS

1. The computed seismic forces are significantly higher than those of the SEAOC recommendations, however, once the loads are factored for design shear, the force level is only somewhat higher than that of the 1973 SEAOC design and in general lower than that of the 1974 SEAOC design.
2. The increase in shear reinforcement for a typical shear wall in this structure is approximately 25%, when compared to the 1973 UBC (SEAOC).

3. The required increase in shear wall flexural reinforcement (for overturning) is roughly in the same proportion as that for shear reinforcement.

4. The increase in cost for updating this building from the 1973 UBC to the proposed Nicaragua code is by no means significant.

5. The level of lateral forces due to the DFS spectrum was smaller than expected because of modal participation; only 59% of the seismic mass vibrates in the first mode. If 100% of the mass would participate in first mode vibration, the base shear would increase from 5863 K to 8710 K. This should be kept in mind when one extrapolates from the results of this example to building with only a few stories where the mass participation of the first mode may be close to one.

6. If the structure were to be built on loose soil, which will load to an increase in the value of $(MDAF)_1$ from 1.48 to 2.00, then the base shear would increase roughly to 7500 K, i.e. by 28%.

7. The increase in story shears due to 5% accidental torsion are not considered in this example, since the member forces are obtained from the computer output of the ATC-2 study that does not include this accidental torsion.