REPORT NO.
EERC 76-7
SEPTEMBER 1976

EARTHQUAKE ENGINEERING RESEARCH CENTER

EXPECTED PERFORMANCE OF UNIFORM BUILDING CODE DESIGNED MASONRY STRUCTURES

by

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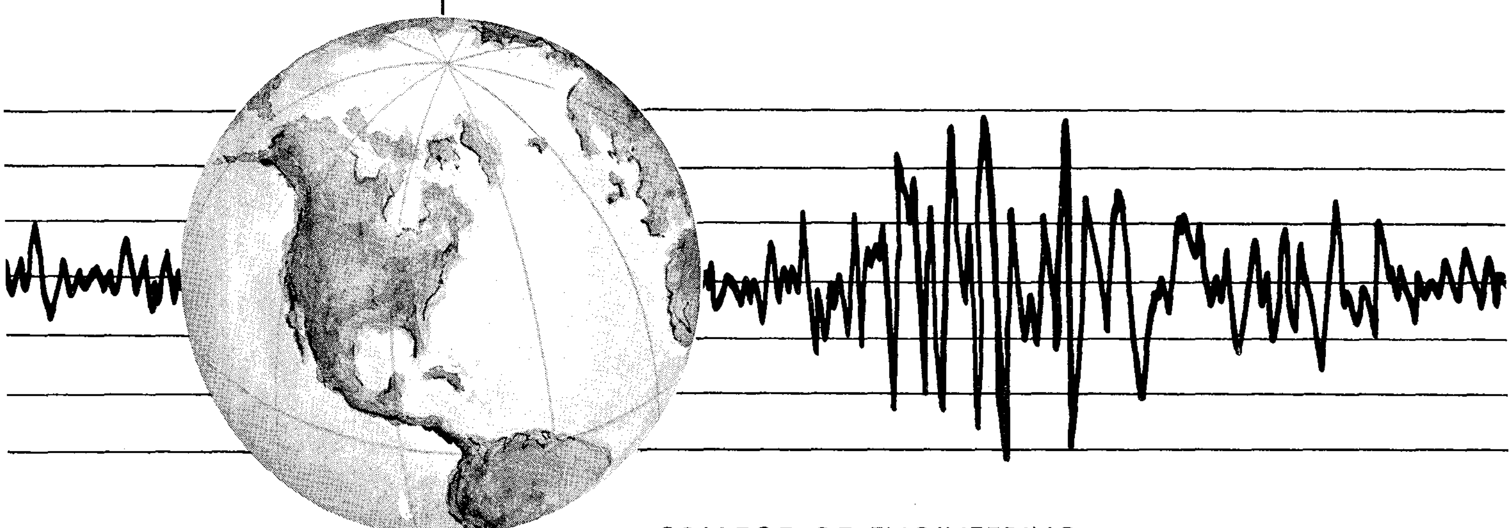
Report to Sponsor:
National Science Foundation



COLLEGE OF ENGINEERING
UNIVERSITY OF CALIFORNIA · Berkeley, California

BIBLIOGRAPHIC DATA SHEET	1. Report No. EERC 76-7	2.	3. Recipient's Accession No.
4. Title and Subtitle Expected Performance of Uniform Building Code Designed Masonry Structures			5. Report Date September 1976
			6.
7. Author(s) R.L. Mayes, Y. Omote, S.-W. Chen and R. W. Clough			8. Performing Organization Rept. No. 76-7
9. Performing Organization Name and Address Earthquake Engineering Research Center University of California, Berkeley 47th Street and Hoffman Boulevard Richmond, California 94804			10. Project/Task/Work Unit No.
			11. Contract/Grant No. AEN-7604265
12. Sponsoring Organization Name and Address National Science Foundation 1800 G Street, N.W. Washington, D.C. 20550			13. Type of Report & Period Covered
			14.

15. Supplementary Notes

16. Abstracts

This report presents an evaluation of the seismic design sections of the 1972, 1973, 1974 and 1976 Uniform Building Codes and the Recommended Comprehensive Seismic Design Provisions for Buildings prepared by the Applied Technology Council (ATC-3). In order to evaluate the various codes a three, a nine and a seventeen story building of similar floor plan were studied. The seismic design stresses in these buildings were calculated by the specified code procedures as well as the stress state predicted by a realistic dynamic earthquake response procedure. The adequacy of the codes was then evaluated by comparing the two types of stress predictions.

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17. Key Words and Document Analysis. 17a. Descriptors

17b. Identifiers/Open-Ended Terms

17c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	
Release unlimited	UNCLASSIFIED	
	20. Security Class (This Page UNCLASSIFIED	22. Price PC A05 / MF A01

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Earthquake Engineering Research Center
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ACKNOWLEDGEMENT

This investigation was sponsored by the National Science Foundation under Grant No. AEN-7604265. The authors wish to thank Dr. D. Williams and Dr. B. Bolt for reviewing the manuscript.

The typing was done by Shirley Edwards and the drafting by Ms. Gale Feazell.

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1. INTRODUCTION

One of the most difficult tasks facing structural engineers today is prediction of the performance of a structure during an earthquake. The problem is compounded with respect to masonry structures because of a lack of experimental data on the performance of masonry structural components. For this reason it is imperative that significant research data are utilized as they become available, to improve the reliability of masonry construction by continually updating the building codes.

The problem of utilizing new research data in evaluating design code requirements is difficult and requires cooperation of research personnel, practicing structural engineers, soils engineers, and seismologists because many facets of earthquake engineering are involved. The first significant attempt to evaluate the expected seismic performance of code designed masonry structures was performed by Young et al[1]. They reported the results of a study on the predicted behavior of two reinforced concrete masonry multi-story (11 and 13 stories) buildings when subjected to specified earthquake ground motions. The purpose of the study was to determine whether these structures would experience severe damage if subjected to earthquake ground motion of an intensity consistent with that which could reasonably be expected to occur during the planned life of the structures. The authors concluded that the buildings would be severely damaged and would probably collapse if subjected to the ground motion considered in the report.

A more recent and broader contribution was made in 1974 with the publication of the Applied Technology Council (ATC-2) report[2] entitled

"An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings". The report addressed the problem "Given response spectra representative of damage-threshold and collapse-threshold earthquake ground motions at a given site, what design procedure should be employed to ensure a given structure an acceptable degree of reliability in protection against damage and prevention of collapse?". The study selected ground motions that were representative of certain sites in Southern California and adopted design procedures based on a response spectrum approach. Eleven existing buildings were chosen for redesign according to these procedures. Included in the study were a three-story and a one-story masonry shear wall building.

The major problem involved in any evaluation of building code requirements is to define an acceptable starting point (i.e., input ground motions or response spectra) and a suitable procedure for evaluating the safety of a given structure. The adequacy of these two definitions will determine the reliability of the evaluation.

Because of time and budget constraints, the scope of the study reported herein is limited to an evaluation of changes that have occurred recently in the seismic section of the masonry portion of the Uniform Building Code (UBC)(3) and in the proposed new ATC-3 seismic design code(4), utilizing the response spectra defined in the ATC-2 report and the results of a State-of-the-Art report on the shear strength of masonry construction performed by the writers(5). Within these constraints the writers envisage this report to be the first part of a continuing effort to utilize relevant research data in evaluating masonry design codes.

1.1 BACKGROUND AND OBJECTIVE OF THE STUDY

Because of the continuing lack of relevant research information on masonry structural assemblages and the associated uncertainty in the seismic behavior of masonry structural components, the UBC masonry seismic design section has been changed substantially several times in the past five years. Although the code allowable shear stresses for seismic loads have remained essentially unchanged (see Table 1), the effective seismic design coefficients have undergone considerable changes, as shown in Fig. 1.

The UBC static method of obtaining the seismic design shear stresses is to calculate the base shear V from the formula

$$V = Z\ K\ C\ W, \tag{1}$$

where Z is a numerical coefficient related to the seismicity of the region, K equals 1.33 for masonry shear wall buildings, W is the total weight of the building and C is the seismic base shear coefficient. According to the 1972 Uniform Building Code, this coefficient is given by

$$C = \frac{0.05}{\sqrt[3]{T}} \tag{2}$$

where T is the fundamental vibration period of the building; but in the 1976 code it has been changed to

$$C = \frac{1}{1.5\ \sqrt{T}} \tag{3}$$

Moreover in the 1976 UBC, equation (1) has been changed to include a site-structure resonance coefficient, S.

TABLE 1. MAXIMUM ALLOWABLE SHEAR STRESSES FOR SEISMIC LOADS

Code	Reinforcement Taking All the Shear	
	$M/V_d > 1$ (psi)	$M/V_d = 0$ (psi)
72-UBC	100	100
73-UBC	100	160
74-UBC	100	160
76-UBC	100	160
ATC-3	112	180

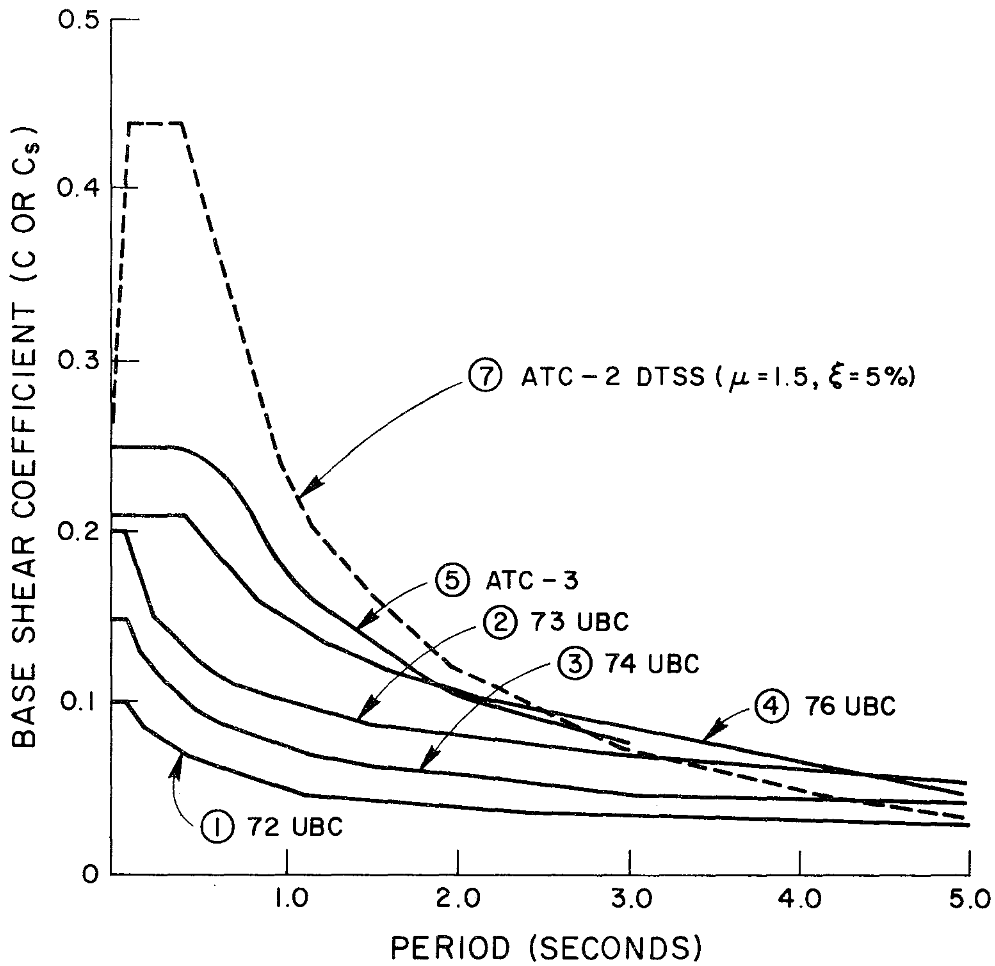


FIGURE 1 EFFECTIVE SEISMIC DESIGN COEFFICIENTS

In both editions of the code, the base shear force V is distributed over the height of the building according to the formula

$$F_x = \frac{h_x \; w_x}{\sum\limits_i h_i \; w_i} \; V, \tag{4}$$

where F_x is the lateral force applied to level "x", h_i or h_x is the height in feet above the base to level "i" or "x", and w_i or w_x is the portion of W which is located at level "i" or "x". The seismic design stresses are then obtained by performing a static analysis of the structure subjected to this force distribution.

In the 1972 UBC, the effective value of C is as shown in curve 1 of Fig. 1. In the 1973 UBC, a footnote to Table 24-H "Maximum Working Stresses for Reinforced Solid and Hollow Unit Masonry" requires that the shear stresses obtained from seismic loads be doubled for design purposes, and in the 1974 code this factor of two is reduced to 1.5. Thus, the seismic loads are effectively increased (for shear stresses but not for overturning moments) by a factor of 2 in the 1973 code and by 1.5 in the 1974 code. These changes of C are shown by curves 2 and 3 in Fig. 1. In the 1976 UBC, the factor C is replaced by CS. The factor of 1.5 still remains as a footnote to Table 24-H and the effective design spectrum obtained for the maximum value of the site-structure interaction factor S is shown as curve 4 in Fig. 1. The effective design spectrum for the ATC-3 proposed seismic design code is shown as curve 5, based on the maximum acceleration value of 0.4g specified for seismic zone 4 and with

$$C_S = \frac{1.2 \; A_2 \; S}{R \; T_R^{2/3}} \tag{5}$$

where C_s is the seismic design coefficient, A_2 is a coefficient representing effective peak ground acceleration, S is a coefficient representing the soil profile, R is the response modification factor and equals 4 for masonry buildings, and T_R is a structural response coefficient related to the fundamental period.

It is clear from Fig. 1 that there has been considerable uncertainty in the past five years as to an appropriate design spectrum for masonry shear wall structures. Consequently the objective of this study was to attempt to evaluate these and other recent code provisions for masonry seismic design. This effort was undertaken after the masonry research program at Berkeley had been in progress for several years, and was in response to a question that has been asked repeatedly: "In the light of your research results, should code allowable shear stresses in masonry remain the same, be increased, or decreased?"

In order to evaluate the various changes that have been introduced in the effective design spectra of recent and proposed building codes, three masonry buildings were studied. The seismic design stresses were calculated in these buildings by the specified code procedures as well as the stress states predicted by a realistic dynamic earthquake response procedure. The adequacy of the codes was then evaluated by comparison of the two types of stress predictions.

The general approach used in the evaluation procedure is outlined in the following chapter of this report. The remainder of the report then presents detailed descriptions of the various steps in the process, as well as tabulations and plots of the numerical results.

2. OVERVIEW OF THE CODE EVALUATION PROCEDURE

This chapter presents an overview of the procedure which was developed to evaluate the protection that recent, current and proposed codes would provide against severe damage or collapse of masonry multistory buildings when subjected to a realistic earthquake. The overview is presented to provide the reader with a framework in which the details of the following chapters are described.

Because this is the first study of this type dealing with masonry buildings, and because of the lack of certain essential research information, it was necessary to make several simplifying assumptions in this work. Acknowledging the limitations of the assumptions and realizing that these should be modified in future studies as more research information becomes available, the procedure developed for this study is as described in the following sections.

2.1 SELECTION OF BUILDINGS

Three buildings with indentical reasonably symmetric floor plans and which have vertical shear walls with openings similar to those found in many multistory masonry buildings were selected for the study. They were three, nine, and seventeen stories high, respectively. Data for the buildings were obtained from the design example presented in "Multistory Load Bearing Brick Walls," a publication of the Brick Institute of California[6]. The buildings were designed originally according to the 1968 Uniform Building Code.

Their plans, elevations and dynamic characteristics are presented in Chapter 3.

2.2 CODE SEISMIC DESIGN STRESSES

The calculated seismic design stresses resulting from the earthquake loads specified by the 1972 UBC, 1973 UBC, 1974 UBC, 1976 UBC and the proposed ATC-3 code for Zone 3 (Zone 4 after 1974) were obtained by performing an equivalent first mode static analysis of the buildings using the computer program ETABS[7]. The seismic design shear stresses and the design vertical stresses resulting from overturning and dead load, calculated for each wall panel of the lower level of each building, are presented in Chapter 5.

2.3 DESIGN FACTOR OF SAFETY

With respect to the shear stresses resulting from the code seismic loads, the design factor of safety, designated A, was evaluated as the ratio

$$A = \frac{\text{Code Allowable Shear Stress}}{\text{Code Calculated Seismic Shear Stress}}, \tag{6}$$

where the denominator is the shear stress described in Section 2.2 above, and the numerator is specified by each code.

It is clear that the minimum permissible value of A is 1.0 and the higher the value of A the greater the design factor of safety. A value of A greater than 1.0 may result from including either a larger number of shear walls or greater wall thicknesses than might be required. Tablulations of the ratio A for the three buildings are included in Chapter 3.

2.4 STRESSES RESULTING FROM A REALISTIC EARTHQUAKE

The stresses determined by application of the response spectrum specified in the ATC-2 report[2] were evaluated for each of the three buildings. This spectrum represents an earthquake having about a 50% probability of being exceeded in 70 years. It was developed for a typical site in Los Angeles area and is based on an inelastic response spectrum concept. The spectrum chosen was the Damage Threshold Spectrum for Strength Determination, with a ductility factor of 1.5 and a damping value of 5%. Each building was analyzed for its response to this input by the method of mode superposition, using the computer program ETABS[7]. The shear and vertical normal stresses resulting from the realistic spectrum were calculated for each of the lower level panels. The results of these analyses, together with a description of the development of the response spectra are presented in Chapter 5.

2.5 SIMPLIFIED FAILURE CRITERIA

Although investigations are currently in progress to determine a realistic failure citerion for masonry structural elements, it was necessary for the purpose of this study to define a simplified failure criterion in order that the work might proceed. Failure was assumed to depend on the maximum tensile stress developed in the shear wall panels. The stress distribution acting on the top and bottom sections of a typical lower level panel is shown in Fig. 2. The normal and shear force acting on the panel due to dead load and the response spectrum analysis were determined first; then the resulting stress distribution was defined by elementary beam theory. Thus the vertical normal stress was assumed to vary linearly across the panel, and the shear stress to be distributed parabolically, as shown in the figure. The maximum tensile stress was assumed to occur at the center of the panel, point A, and was calculated by means of Mohr's circle to be

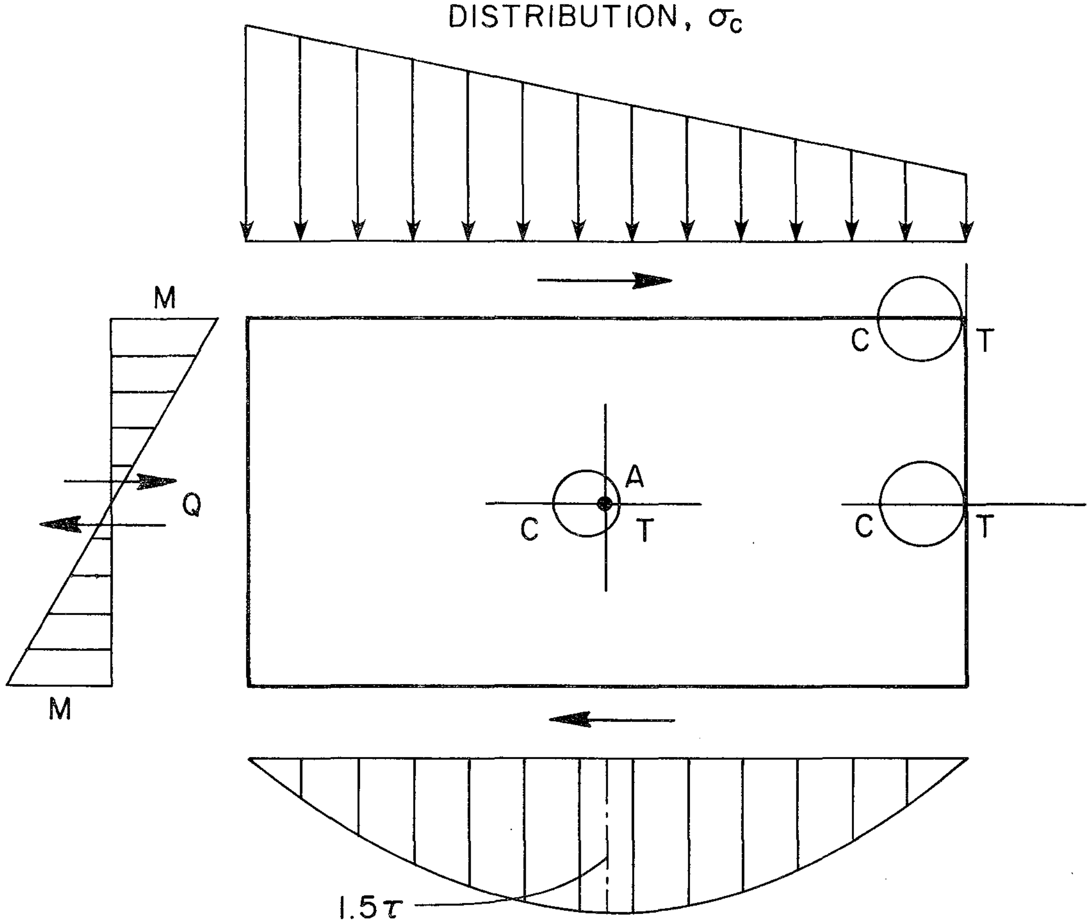
$$\sigma_t = \sqrt{(1.5\tau)^2 + (\sigma_c/2)^2} - \sigma_c/2 \qquad (7)$$

where σ_c and 1.5τ represent the normal and shear stress on a horizontal section at this point. A summary of the maximum tensile stresses calculated in this way in the lower level panels of the three buildings is presented in Chapter 5.

2.6 EXPECTED PERFORMANCE BASED ON STRENGTH MEASUREMENTS

In order to evaluate the expected performance of the three buildings when subjected to the realistic earthquake, a critical tensile strength for the lower level panels was evaluated from available test data. Since no tests have been performed on test specimens of the size of the lower level panels, data obtained on other types of test panels were used. A summary and evaluation of the test data are included in Chapter 6. To ensure a conservative evaluation, the lower bound of the available test data was defined as the critical tensile strength.

ASSUMED VERTICAL NORMAL STRESS DISTRIBUTION, σ_c



ASSUMED PARABOLIC SHEAR STRESS DISTRIBUTION, τ

FIGURE 2 ASSUMED STRESS DISTRIBUTION OF THE PANELS

The expected performance of the buildings was then expressed by the ratio B, representing the expected factor of safety and defined as

$$B = \frac{\text{Critical Tensile Strength}}{\text{Calculated Principal Tensile Stress}} \quad . \qquad (8)$$

A value of B greater than 1.0 indicates that a panel would perform adequately during the expected earthquake, while a value significantly less than 1.0 would postulate failure of that particular panel. A tabulation of the ratio B for the lower level panels of the three buildings is presented in Chapter 7.

2.7 EVALUATION OF THE CODES

Although the ratios A and B, given by Equations 6 and 8, represent the design factor of safety and the expected actual performance of a particular building subjected to a realistic earthquake, they cannot, when considered separately, be used to evaluate the various codes. This is because the value of A, shown in Chapter 3, varies for each code for a given building design. The ratio B, considered separately, indicates the adequacy of a given code only when the ratio A is the same for all codes. However, the ratio B/A provides a direct measure of a code's suitability. If B/A is greater than 1.0 the code may be considered adequate, but if B/A is less than 1.0 the code may be assumed to provide inadequate protection against severe damage or collapse during a particular earthquake.

The results and a discussion of the code evaluation using the method outlined above are presented in Chapter 7. The authors believe that the procedure outlined will provide some perspective on recent code changes that have occurred in the seismic design of multistory

masonry buildings. As more research information becomes available, the procedures should be modified for future studies, thereby contributing to the development of more reliable design codes.

3. STRUCTURAL DETAILS AND DYNAMIC CHARACTERISTICS OF THE BUILDINGS

As was mentioned before, the three buildings considered in the study all had identical floor plans, the design data for the buildings were obtained from a publication of the Brick Institute of California[6], and the dynamic characteristics of the buildings were calculated using the computer program ETABS[7]. In addition to considering the fixed base dynamic characteristics of the three buildings, a simplified model for foundation flexibility was included to evaluate its effect on the dynamic response of the buildings. The structural details of the buildings, the structural modelling techniques used, and the computed dynamic characteristics are described in the following sections.

3.1 PLAN AND ELEVATIONS OF THE THREE BUILDINGS

The general floor plan of the three buildings is shown in Fig. 3. The overall plan dimensions are 74 ft x 152 ft. Wall thicknesses for the buildings are given in Fig. 4, together with a cross section of the nine story building. Typical elevations of the nine story building are shown in Figs. 5, 6 and 7; the other two buildings were of similar form, but with different numbers of stories.

The shear wall arrangement is reasonably symmetric, with walls varying in width from 16 ft. to 40 ft. Four of the seventeen shear walls have openings at each floor level. Figure 8 shows the floor plan of the analytical model used for the computer analysis.

3.2 STRUCTURAL MODELLING TECHNIQUES

The computer program ETABS, written by Wilson, Hollings, and Dovey, is a three dimensional dynamic building analysis program, the features of which are described in Reference 7.

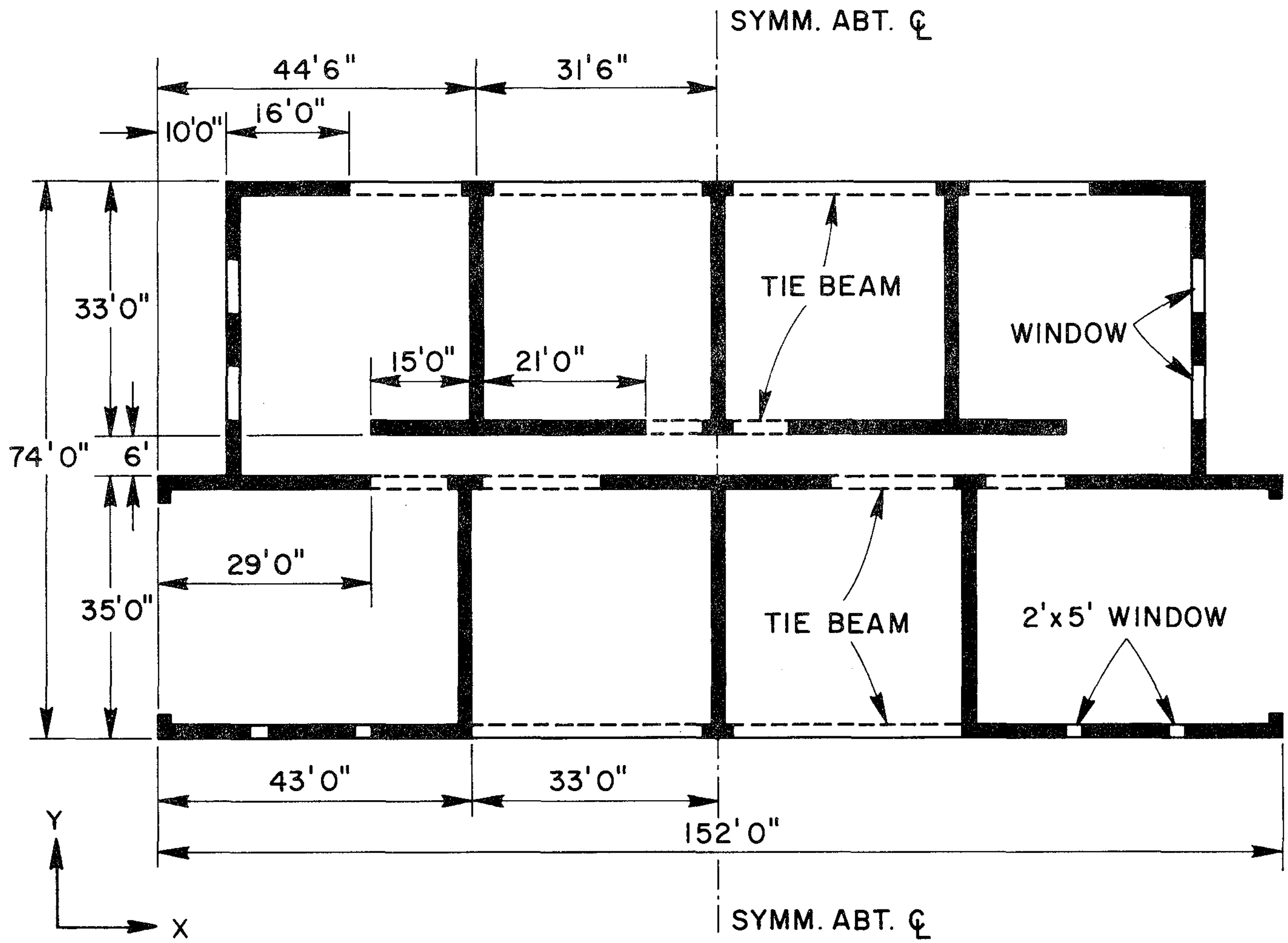


FIGURE 3 FLOOR PLAN OF THE BUILDINGS

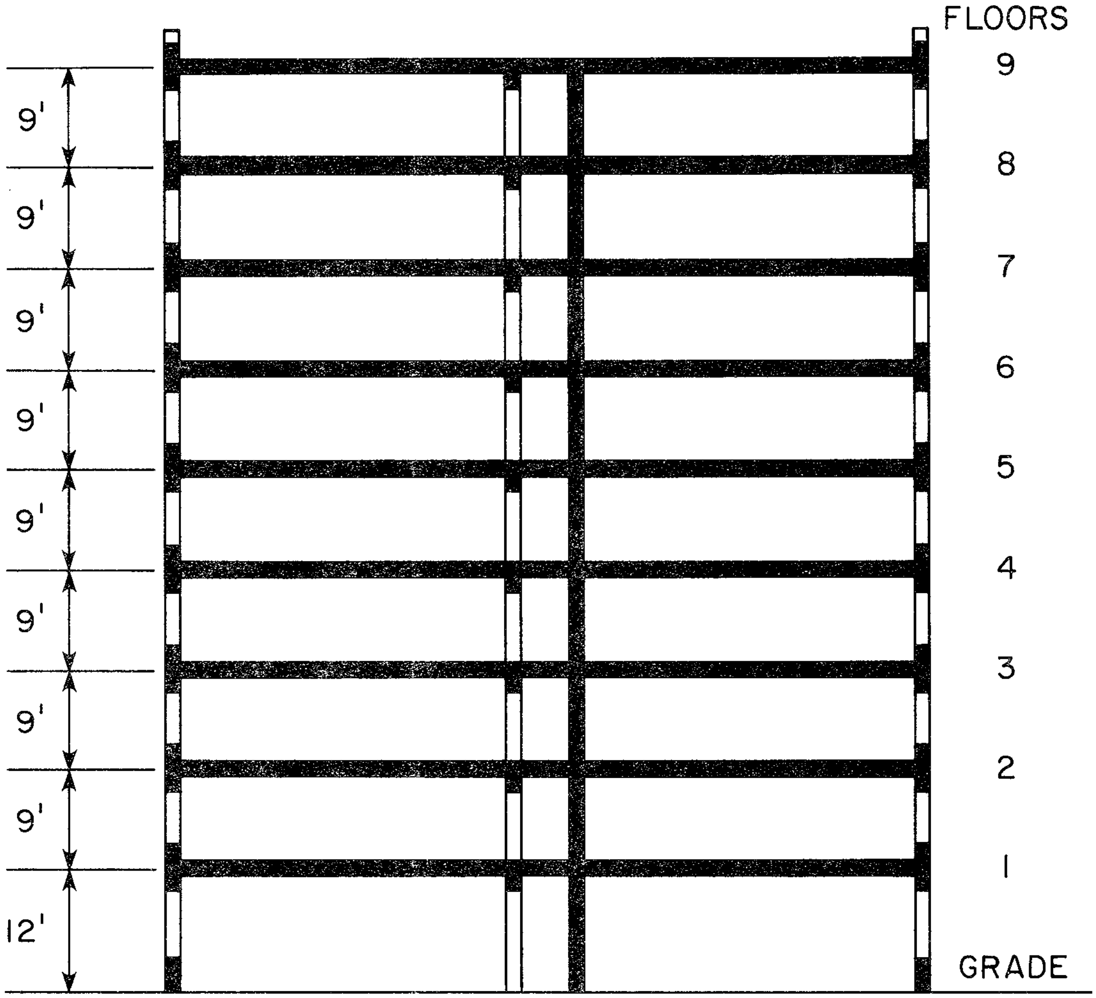


FIGURE 4 BUILDING CROSS-SECTION AND WALL THICKNESSES

STUDY CASES	WALL THICKNESS		
	9 inches	11 inches	13 inches
"A" (3-story)	FL 1-FL 3	——	——
"B" (9-story)	FL 1-FL 5	FL6-FL 9	
"C" (17-story)	FL 1-FL 5	FL6-FL 11	FL11-FL 17

Note: For each case 1^{st} FL. HT. = 12' 0"

Remaining floors FL. HT. = 9' 0"

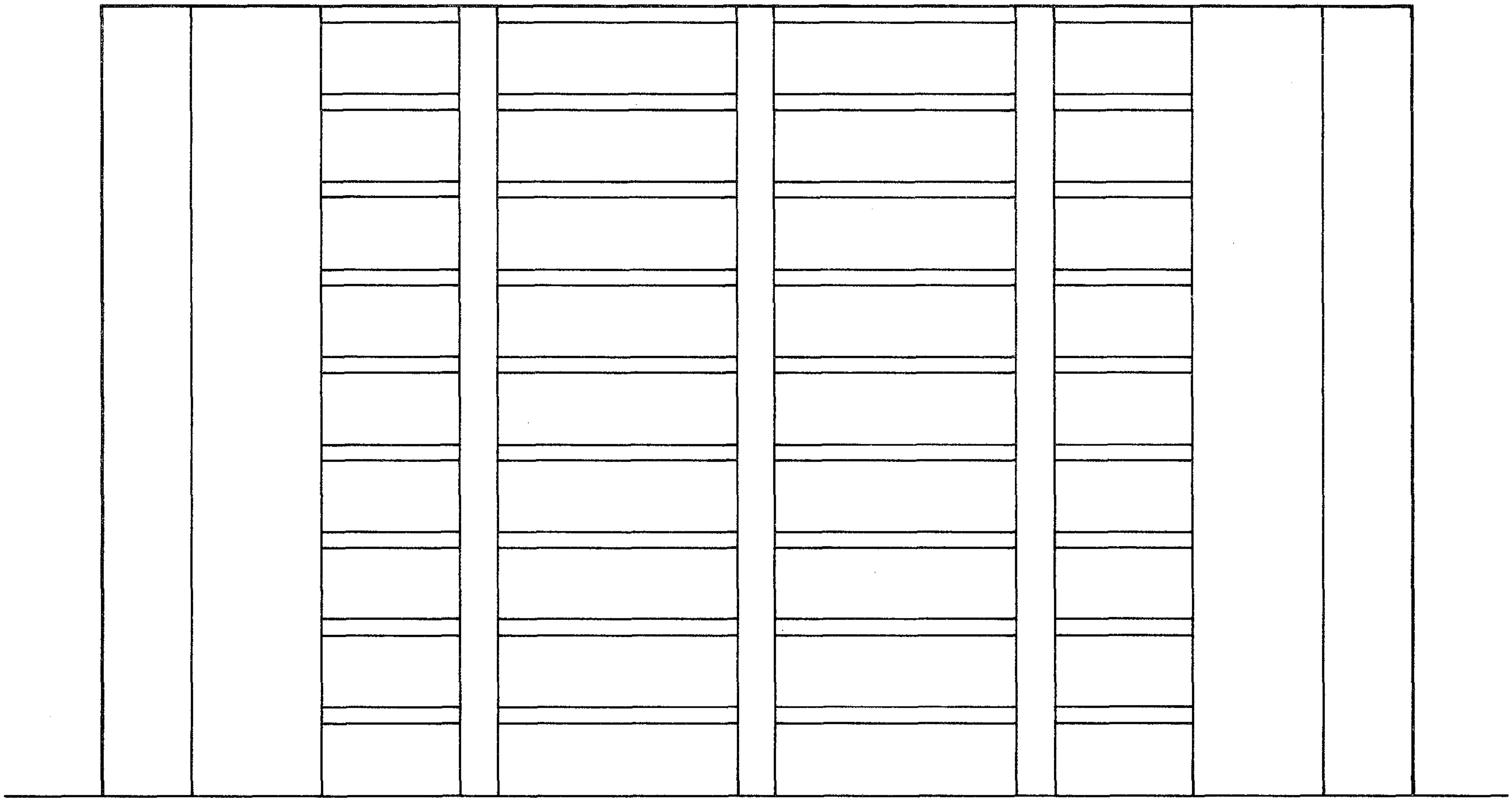


FIGURE 5 NORTH ELEVATION

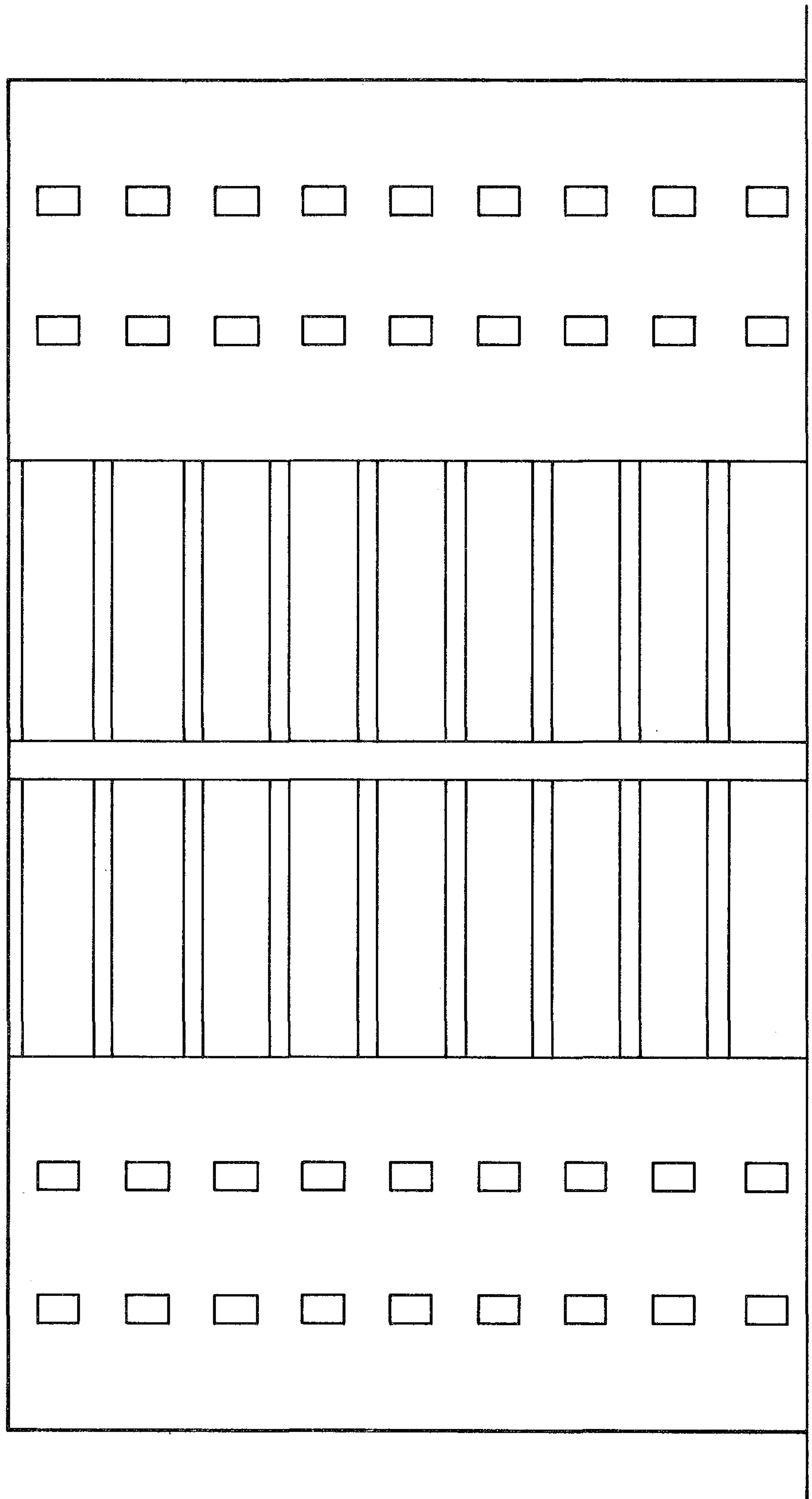


FIGURE 6 SOUTH ELEVATION

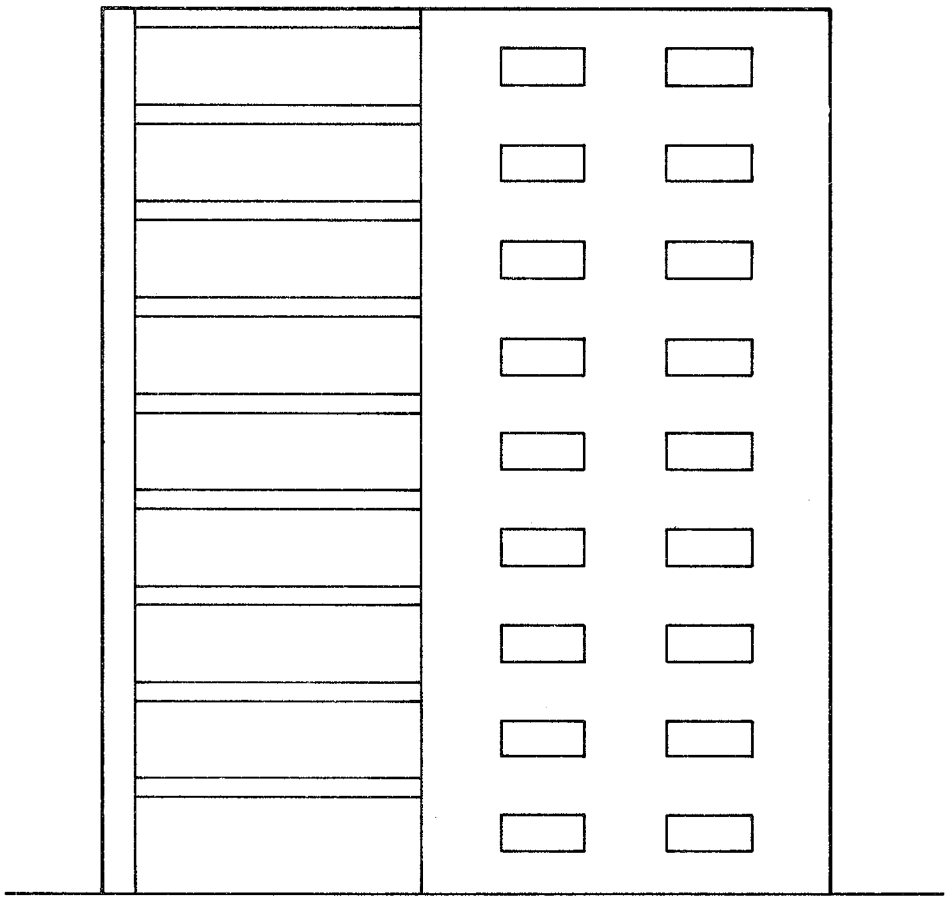


FIGURE 7 WEST ELEVATION

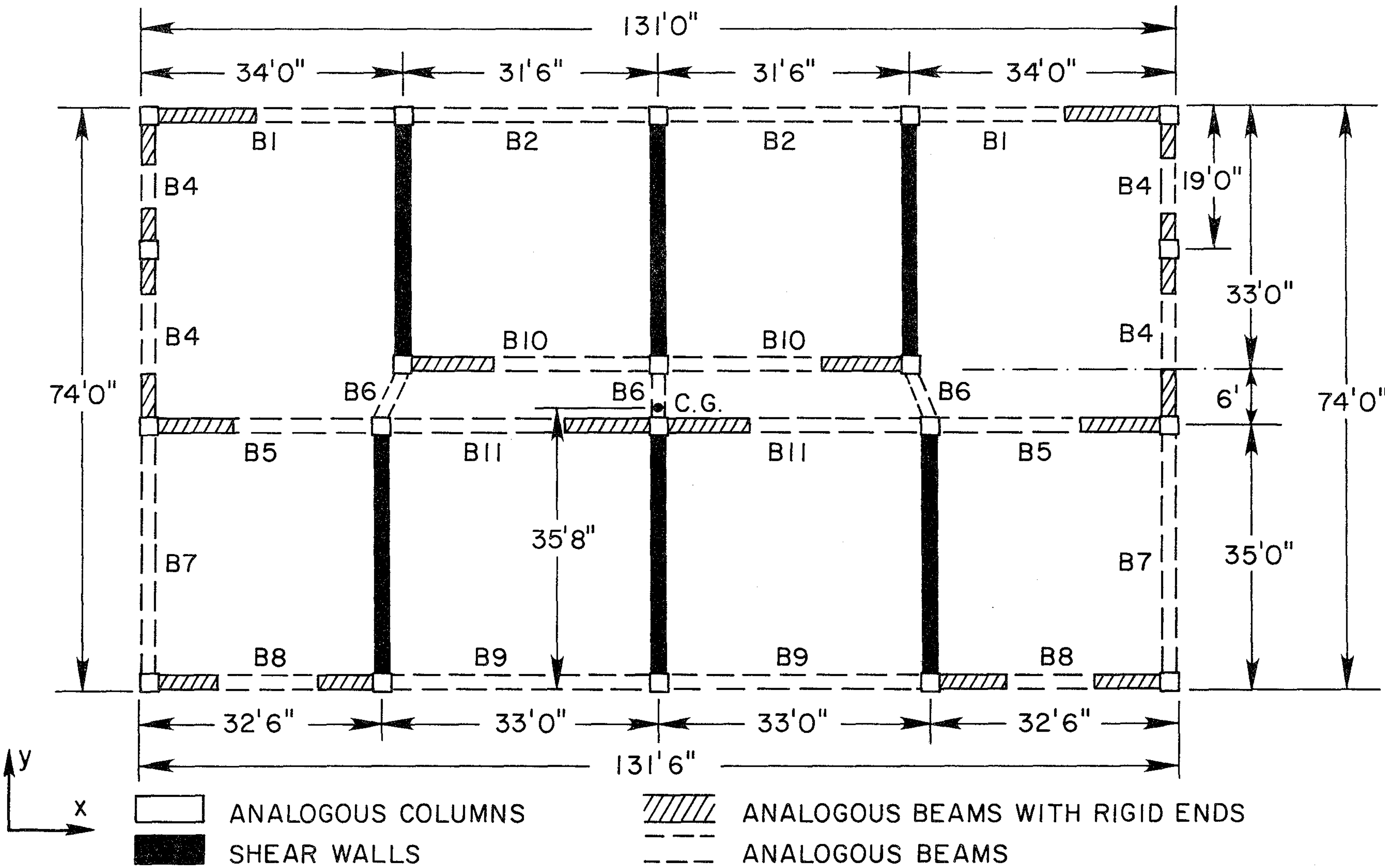


FIGURE 8 ANALYTICAL MODEL OF MULTISTORY MASONRY APARTMENTS

The structural model used in the analysis consisted of the following:

(1) Shear walls with uniform openings up the height of the building were modelled by the "equivalent frame" or "deep column analogy" concept[8], which may be described as follows:

a) The center lines of the wall sections and of all connecting beams form the equivalent frame.

b) The cross-sectional properties of the wall sections in the equivalent frame are identical with those of the corresponding wall sections.

c) The central portions of all model beams have the same cross-sectional areas as the connecting beams of the actual shear wall structures. The fictitious portion of the beam contained within the shear wall is modelled as a rigid link as shown in Fig. 9.

(2) Flexural and shear stiffnesses of all members are based on uncracked sections.

(3) The inner shear walls are represented by shear panels connecting the columns of the equivalent frame. The shear walls have both shear and flexural stiffness as described in Reference 8.

(4) The floor system is assumed to be rigid, in its own plane.

(5) Both rigid and flexible foundation supports are considered.

(6) The flexible foundation was included to approximate the soil-structure interaction effect; it was assumed that the foundation flexibility was represented by an additional story level below the building.

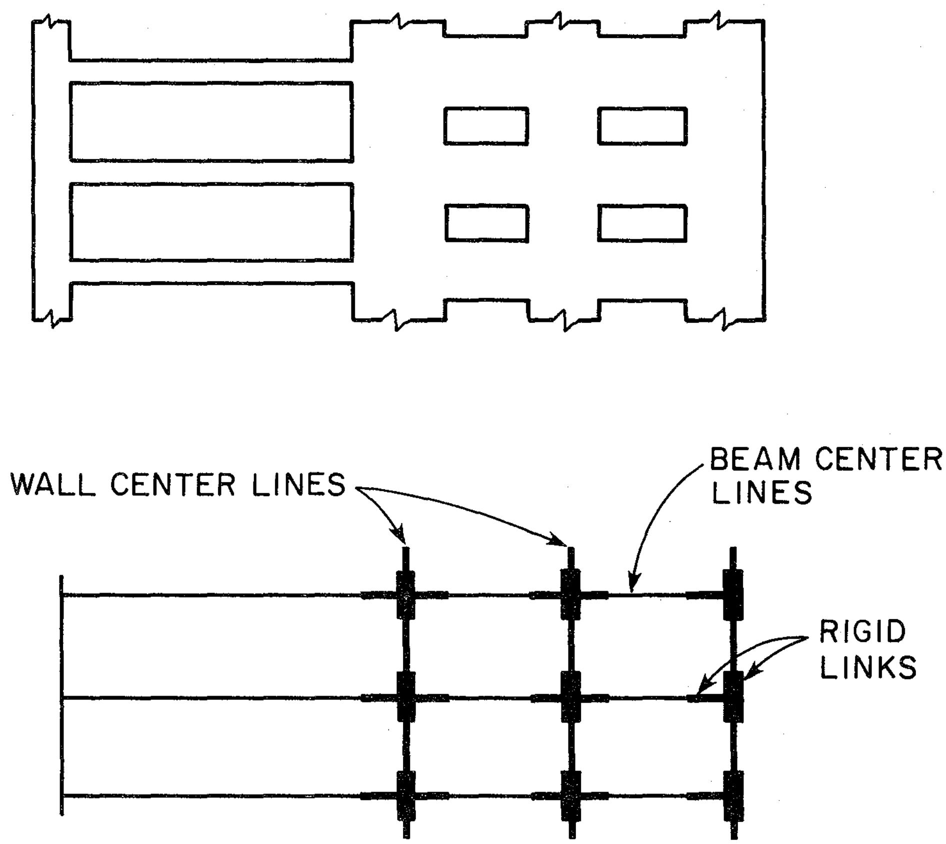


FIGURE 9 RIGID BEAM LINK MODEL

3.3 DYNAMIC CHARACTERISTICS OF THE BUILDINGS

The three different base fixity conditions for which the dynamic characteristics of each of the three buildings were obtained, were as follows:

(1) The fixed base condition, designated FIX.

(2) The first simplified foundation flexibility model, designated SSI-1. In this case the foundation flexibility was represented by an additional story level of columns. The properties of the columns were obtained from forced vibration tests on an eleven story masonry shear wall building[9], in which the column properties were evaluated so that the analytical dynamic characteristics correlated with the experimental results.

(3) The second simplified foundation flexibility model, designated SSI-2, in which the foundation flexibility was represented by an additional story level of combined shear and flexural panels. The properties of the panels were obtained from Reference 10 and represented the stiffness conditions of a layer of San Francisco Bay mud.

The dynamic characteristics for Cases 2 and 3 were considered because it is believed that soil-structure interaction may have a significant effect on the dynamic properties of masonry shear wall buildings because of the inherent rigidity of such buildings.

Figures 10, 11, 12 and Table 2 present plots and listings of the calculated natural mode shapes and periods of the three buildings. Each building is symmetric in the Y-direction and asymmetric in the X-direction (see Fig. 3); in each case (except SSI-2 for Building C) the lowest mode of vibration is in the X-direction, and is combined

TABLE 2. BUILDING PERIODS OF VIBRATION

Building	Period Sec. / Base	1st. Mode	2nd. Mode	3rd. Mode	4th. Mode
3-story	FIX	0.080	0.075	0.020	0.019
	SSI-1	0.095	0.090	0.027	0.025
	SSI-2	0.140	0.100	0.040	0.026
9-story	FIX	0.36	0.27	0.07	0.07
	SSI-1	0.37	0.28	0.08	0.08
	SSI-2	0.38	0.34	0.10	0.09
17-story	FIX	0.95	0.65	0.21	0.17
	SSI-1	0.97	0.66	0.21	0.17
	SSI-2	0.97	0.72	0.22	0.20

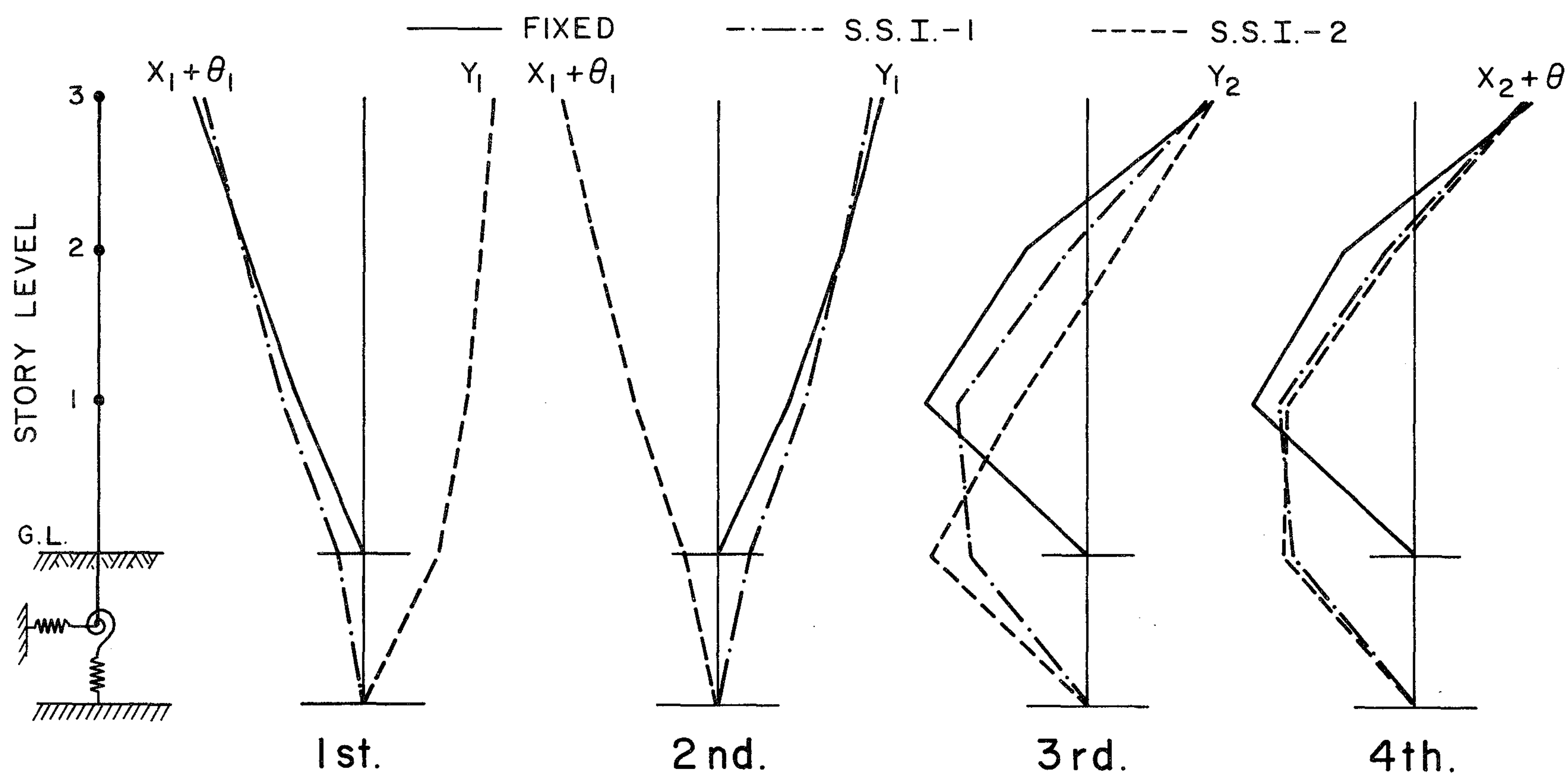


FIGURE 10 MODE SHAPES OF 3-STORY BUILDINGS

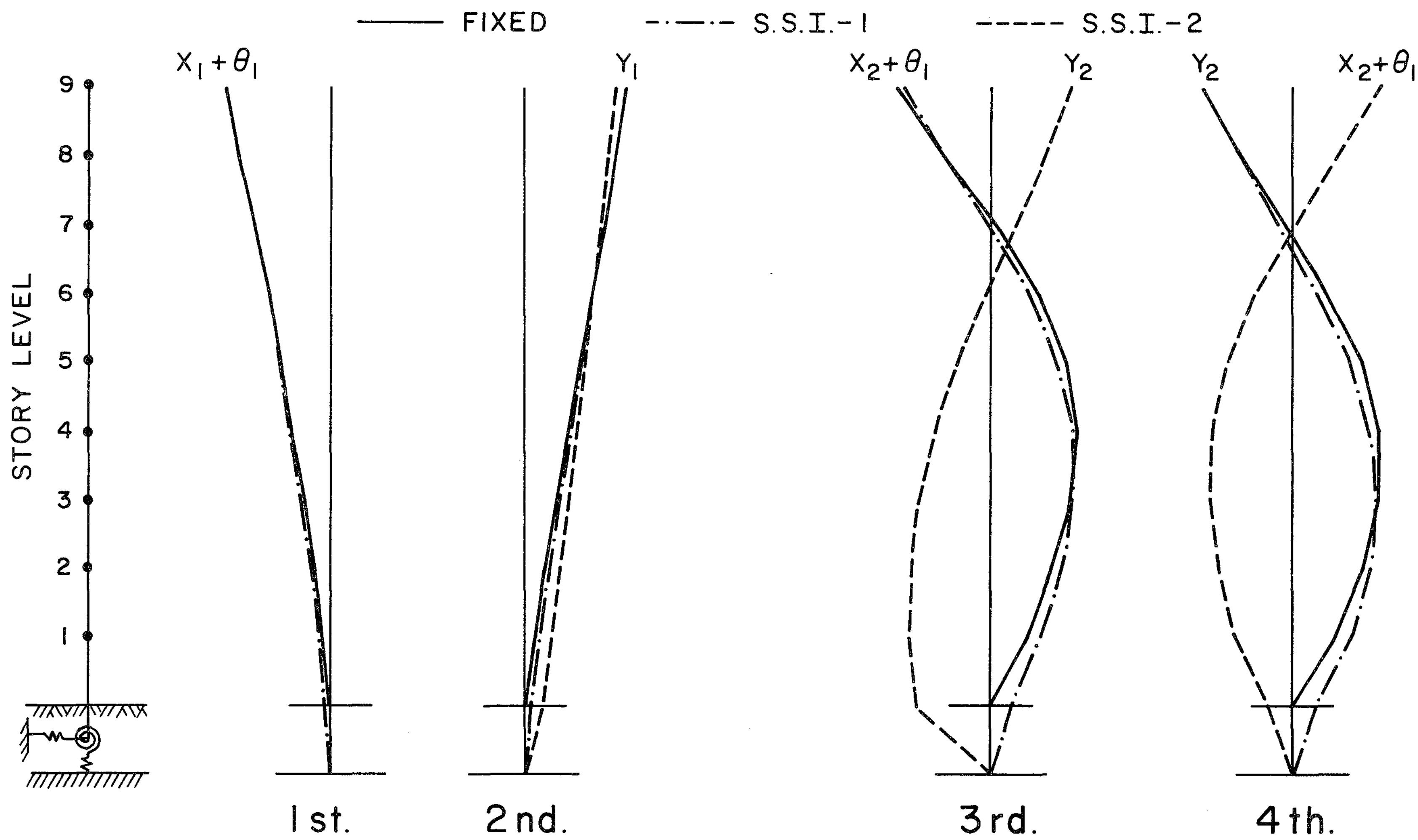


FIGURE 11 MODE SHAPES OF 9-STORY BUILDING

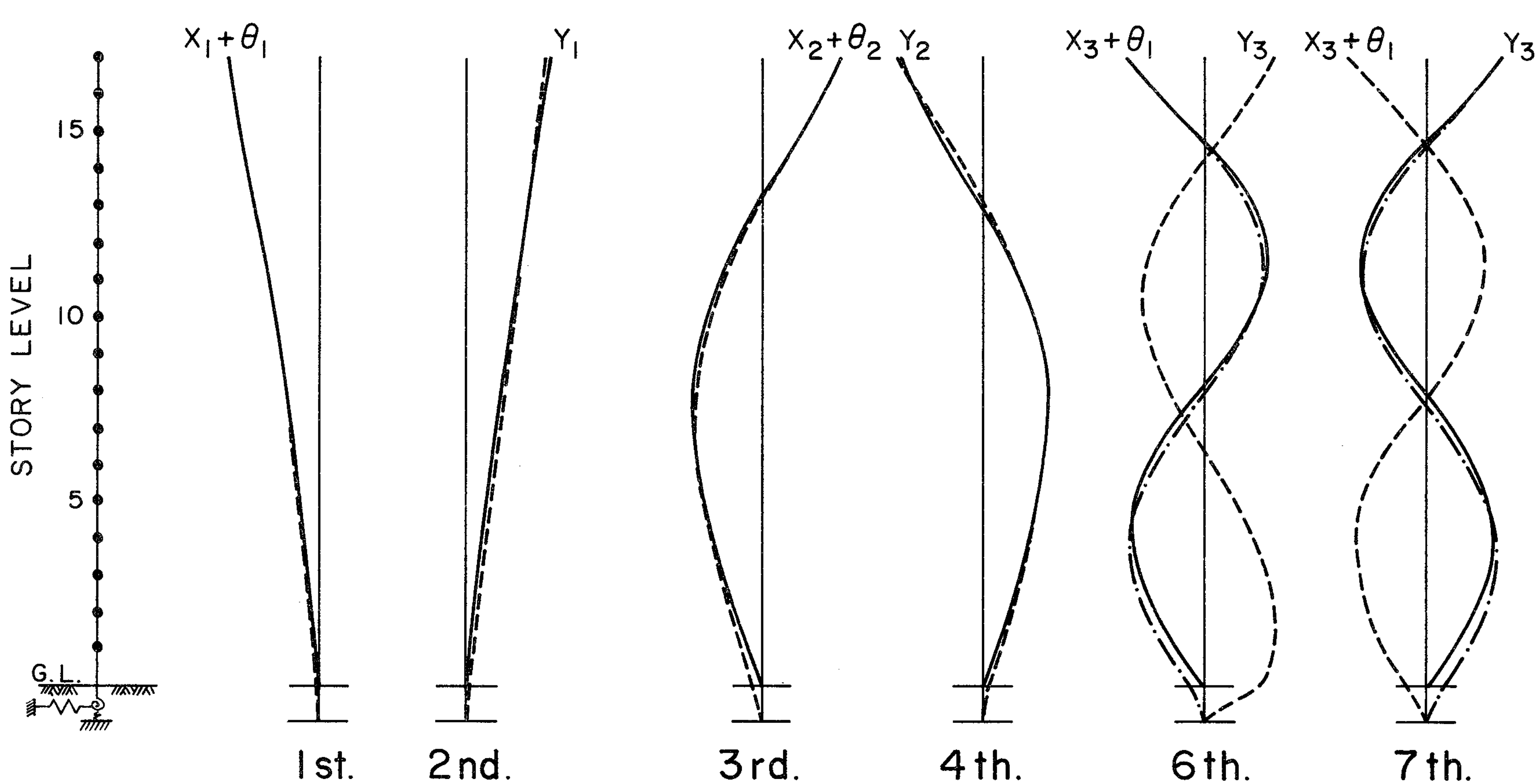


FIGURE 12 MODE SHAPES OF 17-STORY BUILDING

with some floor rotation (torsion). As may be seen in Table 2, the Y-direction modes of vibration of the buildings have periods of 0.075 sec, 0.269 sec, and 0.651 sec, for the fixed base condition for the three, nine and seventeen story buildings, respectively.

The 1973 SEAOC code states that the lowest mode period of a building in a given direction may be estimated by the following formula:

$$T = \frac{0.05\ H}{\sqrt{D}} \tag{9}$$

where H is the total height of the building and D is the plan dimension of the building in the direction of vibration. The estimated periods using this formula are 0.17 sec, 0.48 sec, 0.91 sec, respectively, for the 3, 9 and 17 story buildings. These are significantly greater than the values calculated from the actual building properties, and from a design point of view they are non-conservative because in general the seismic coefficient C of Eq. 1 increases as the period decreases.

Table 2 also shows that the period of the 3 story building is the most sensitive to foundation flexibility, as might be expected since it is the stiffest structure. The ratio T_{SS1-2}/T_{FIX} for the first mode in the Y-direction is 1.9, 1.3 and 1.1 for the 3, 9 and 17 story buildings, respectively. Considering this effect from the design point of view, it is clear that foundation flexibility tends to reduce the code predicted earthquake response. This effect will be considered more fully in Chapter 4.

4. CODE DESIGN STRESSES

The code design procedure was outlined in the introduction to this report. In essence, it involves evaluation of the base shear force V resulting from the specified seismic coefficient C, and then distribution of the force over the height of the building to obtain the effective earthquake loads. The drastic changes that have been made in the seismic coefficient C in recent versions of the code, as shown by Fig. 1, have led to corresponding changes in the resulting earthquake loads to be used in building design. The effects of these changes will be demonstrated graphically in this chapter by means of plots of the most significant structural response quantities.

Plots of the design story shear forces calculated for each building using each of the five codes are given in Figs. 13, 15 and 17. Similarly, the overturning moments derived for each building using each code are given in Figs. 14, 16 and 18. Finally, the resulting shear stresses at the base of the shear wall panels for each story in each building are given in Figs. 19-21. As can be seen from these last figures, the distribution of story shear stresses in the lower level is reasonably uniform among the larger inner panels.

Using these results, the design factor of safety with respect to seismic loads is given by the ratio A, (Eq. 6) representing the ratio of code allowable stress to code predicted stress. Tabulations of the maximum shear stress on the lower level panels, together with the code allowable shear stress and the resulting ratio A, are given in Table 3 for the three buildings. Although the ratio A nominally has a minimum value of 1.0, it is less than one in certain cases here because the buildings considered in the study were designed by the 1968 Uniform Building Code, rather than the codes being considered.

TABLE 3. DESIGN FACTOR OF SAFETY A (Based on the lowest level shear panels)

Building	Code	Code Allowable Shear Stress* (psi)	Seismic Design Shear Stress (psi)				$A = \frac{\text{Code Allowable Shear Stress}}{\text{Seismic Design Shear Stress}}$			
			P-2**	P-3	P-11	P-12	P-2**	P-3	P-11	P-12
3	72 UBC	100	14.5	15.1	15.0	12.6	6.90	6.62	6.67	7.94
	73 UBC	147	29.1	30.1	30.1	25.1	5.05	4.88	4.88	5.86
	74 UBC	147	21.8	22.6	22.5	18.8	6.74	6.50	6.53	7.82
	76 UBC	147	33.9	35.3	35.1	29.5	4.34	4.16	4.19	4.98
	ATC-3	165	36.3	37.8	37.5	31.5	4.5	4.4	4.4	5.24
9	72 UBC	100	37.0	37.3	37.3	30.9	2.70	2.68	2.68	3.24
	73 UBC	147	74.0	74.6	74.6	61.8	1.99	1.97	1.97	2.38
	74 UBC	147	55.5	56.0	56.0	46.4	2.65	2.63	2.63	3.17
	76 UBC	147	122.1	123.1	123.1	102.0	1.20	1.19	1.19	1.44
	ATC-3	165	131.4	132.8	132.8	110.0	1.26	1.25	1.25	1.5
17	72 UBC	100	65.4	64.0	64.7	53.6	1.53	1.56	1.55	1.87
	73 UBC	147	130.8	128.0	129.4	107.2	1.12	1.15	1.14	1.37
	74 UBC	147	98.1	96.0	97.0	80.4	1.50	1.53	1.52	1.83
	76 UBC	147	199.1	194.8	197.0	163.2	0.74	0.75	0.75	0.90
	ATC-3	165	245.3	240.0	242.6	201.0	0.68	0.69	0.68	0.83

* $M/V_d = 0.26$

** P-2, P-3, P-11, P-12 are identified in Figure 19

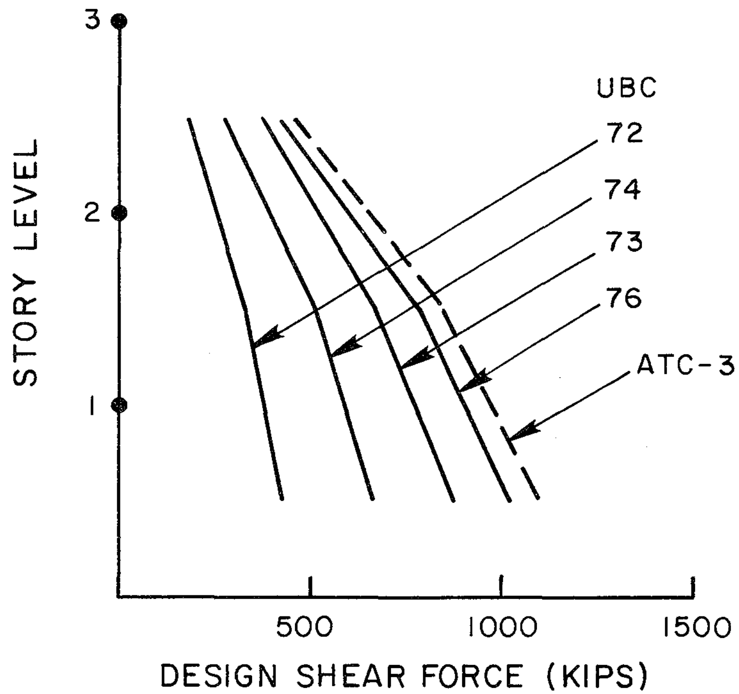


FIGURE 13 DESIGN SHEAR FORCES FOR THE 3-STORY BUILDING

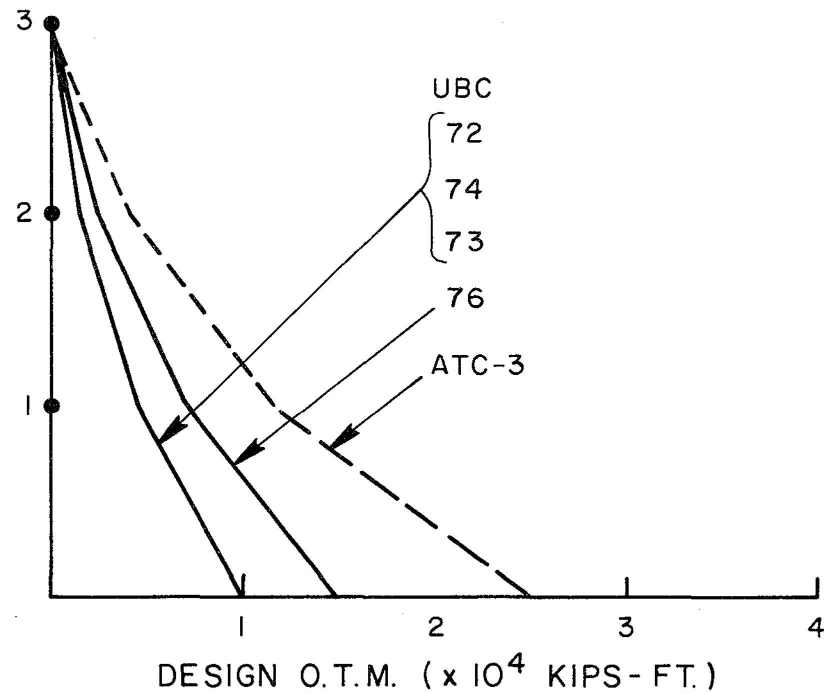


FIGURE 14 DESIGN O.T.M. FOR THE 3-STORY BUILDING

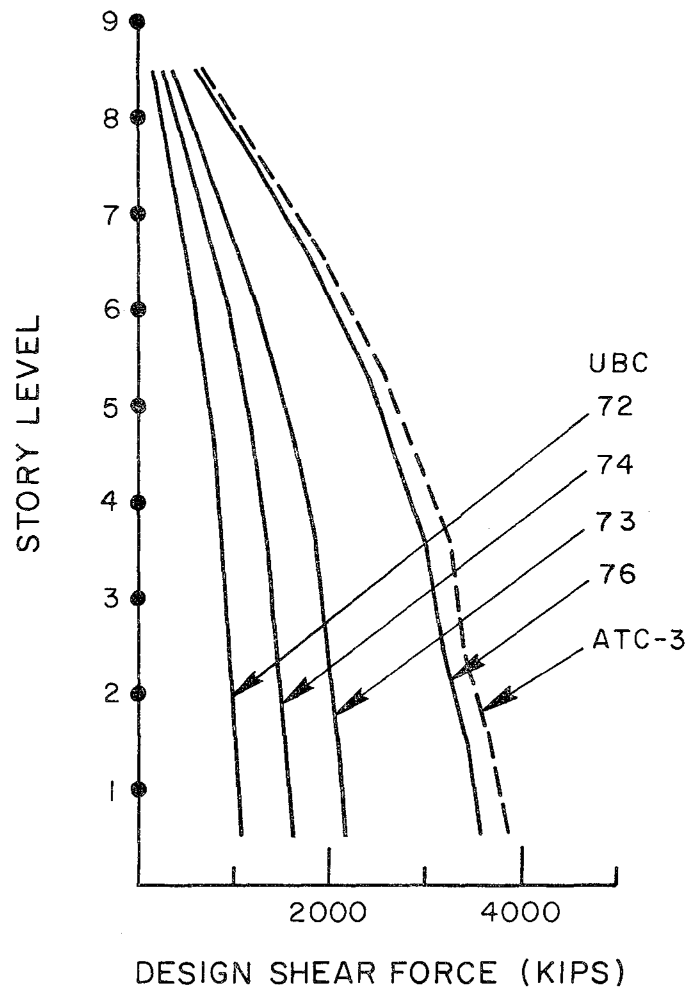


FIGURE 15 DESIGN SHEAR FORCES FOR THE 9-STORY BUILDING

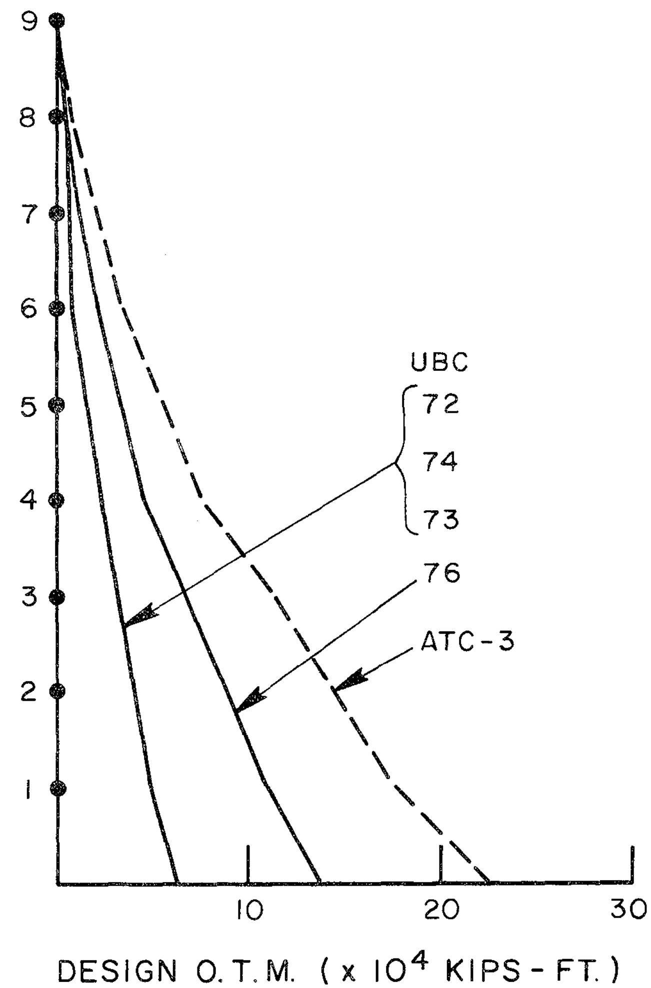


FIGURE 16 DESIGN O.T.M. FOR THE 9-STORY BUILDING

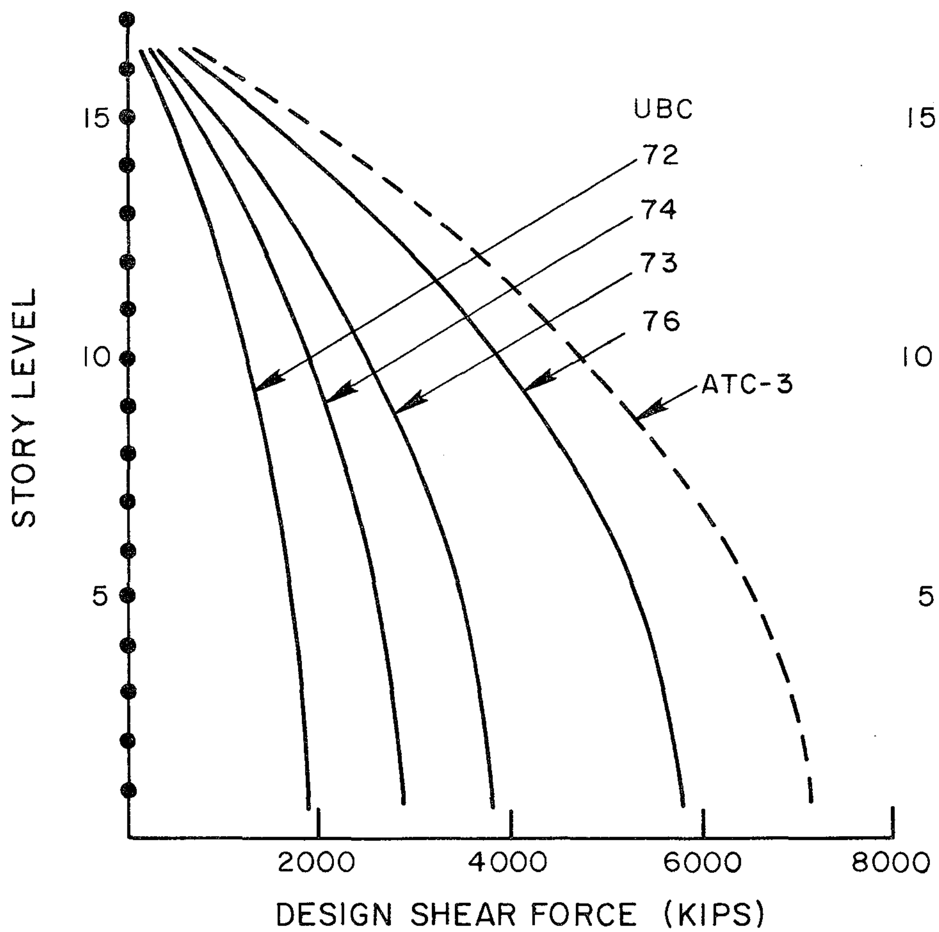


FIGURE 17 DESIGN SHEAR FORCES FOR THE 17-STORY BUILDING

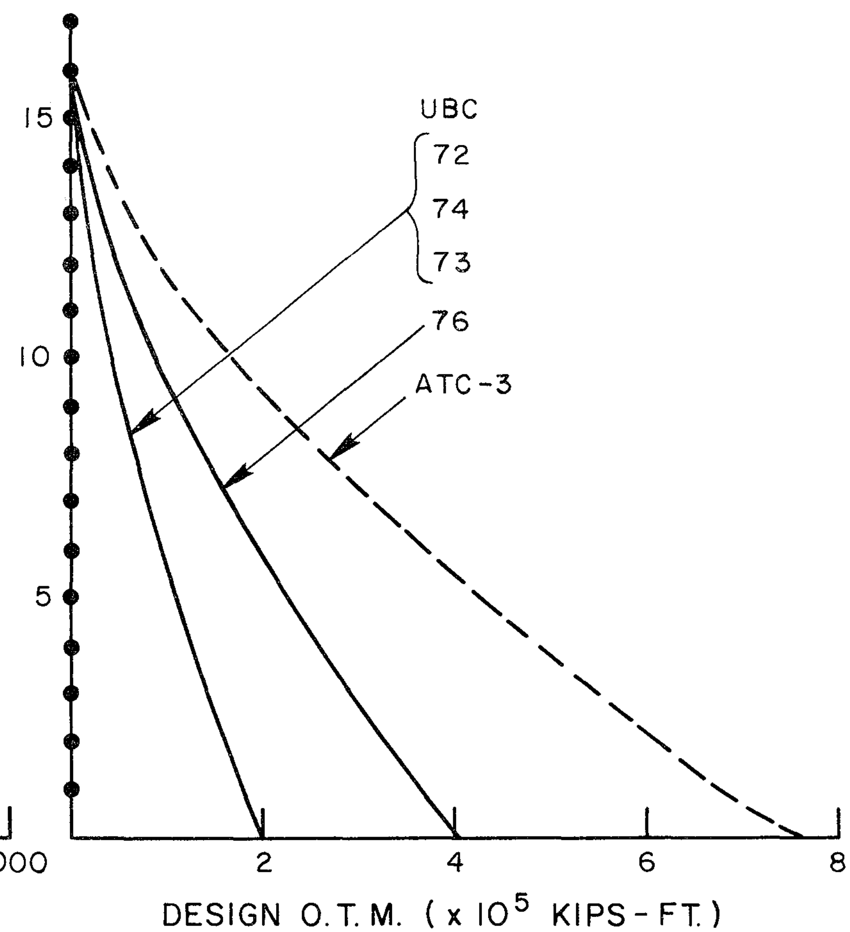


FIGURE 18 DESIGN O.T.M. FOR THE 17-STORY BUILDING

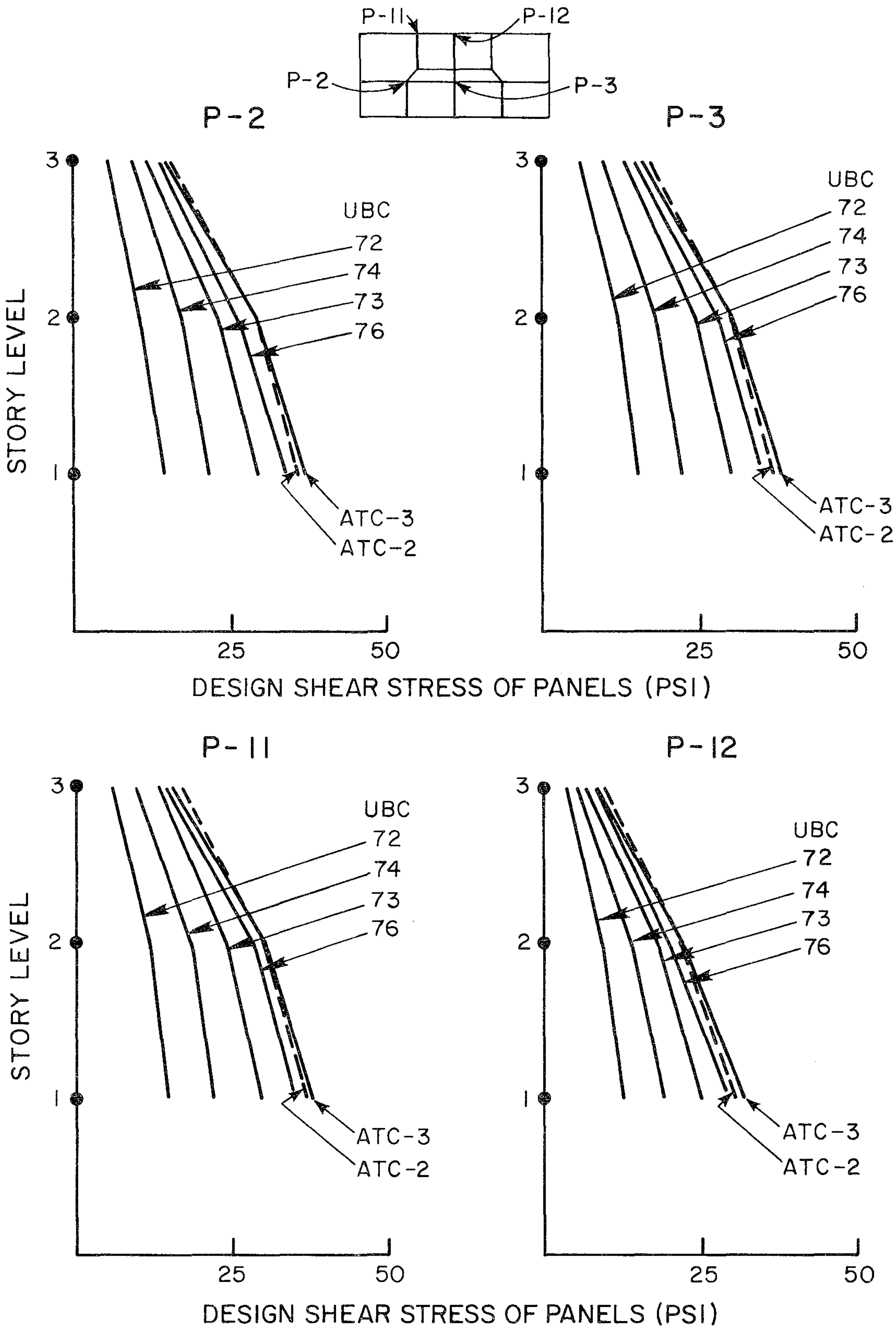


FIGURE 19 DESIGN SHEAR STRESSES OF PANELS FOR 3-STORY BUILDING

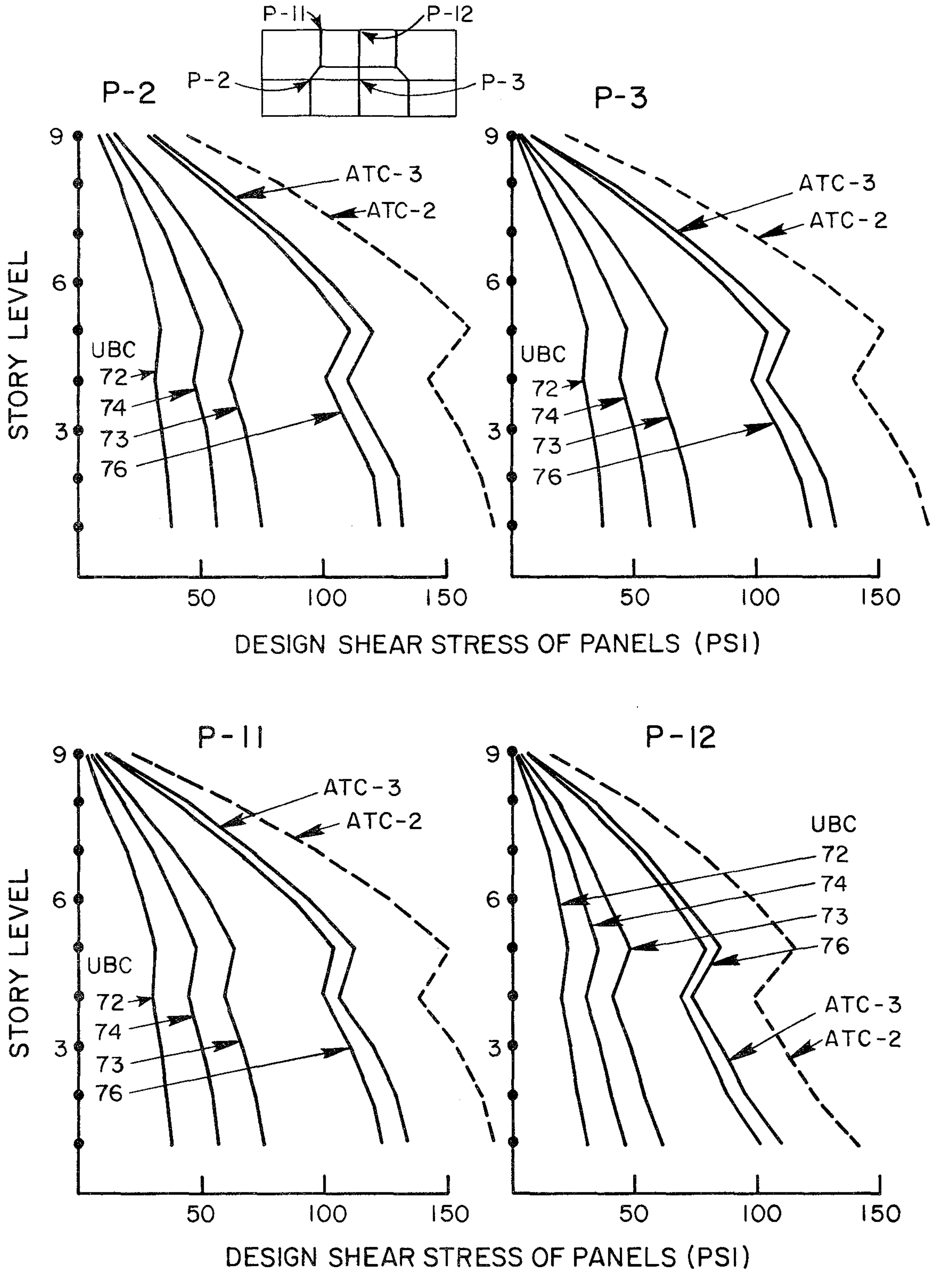


FIGURE 20 DESIGN SHEAR STRESSES OF PANELS FOR 9-STORY BUILDING

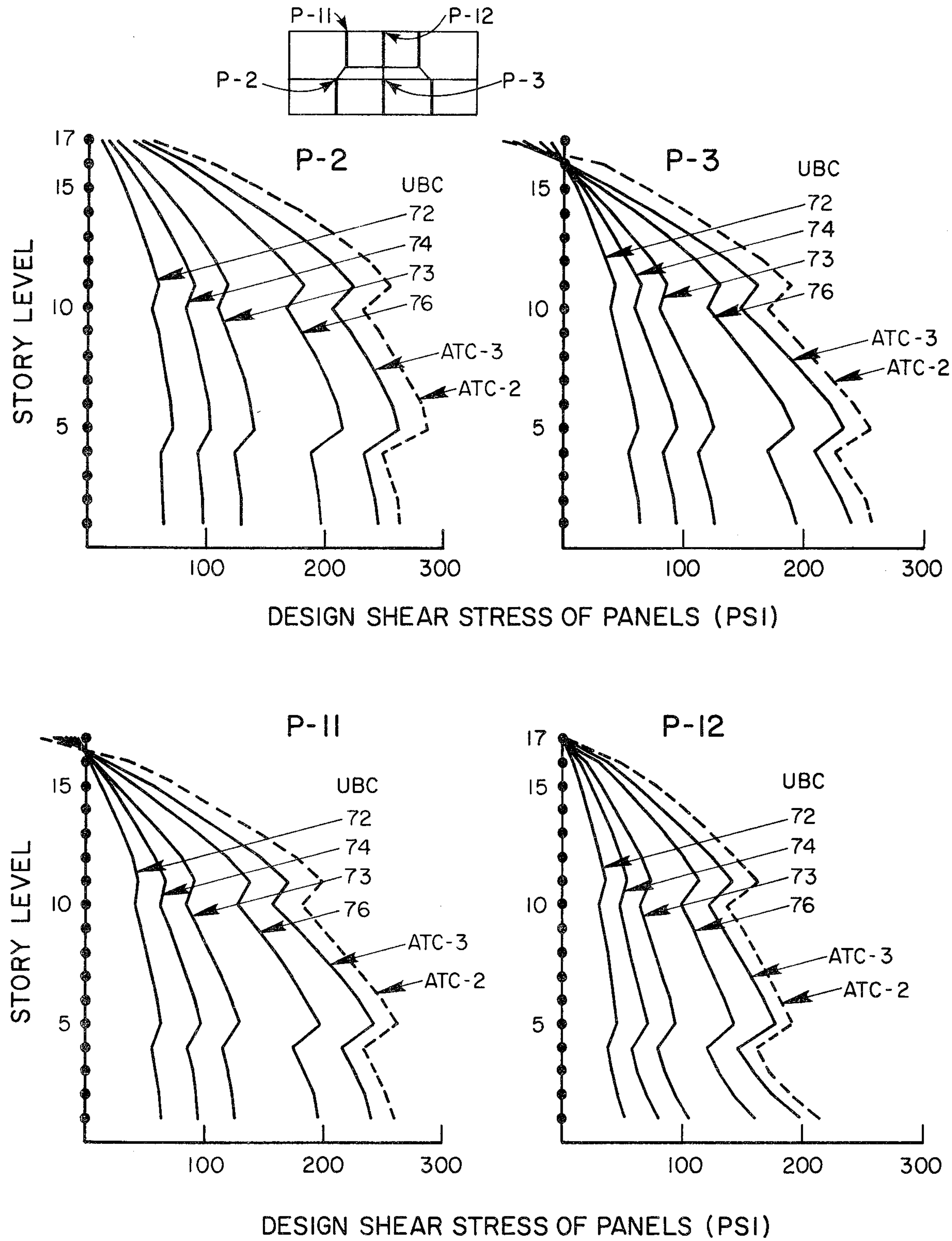


FIGURE 21 DESIGN SHEAR STRESSES OF PANELS FOR 17-STORY BUILDING

5. STRUCTURAL RESPONSE RESULTING FROM THE REALISTIC EARTHQUAKE

5.1 THE REALISTIC EARTHQUAKE INPUT

Probably the most difficult phase of evaluating the seismic safety of any structure is defining the maximum earthquake that may reasonably be expected to occur during the life of the building. Because considerable expertise was devoted to that task in the ATC-2 study mentioned earlier(2), it was decided to use the response spectra developed there for the purposes of the present investigation. A brief description of the ATC-2 spectra and the procedure used in deriving them follows.

The ATC-2 study was directed toward developing methods of seismic design that would provide structures with adequate resistance to earthquake damage and also avoid the possibility of collapse during its expected life. To achieve these two objectives, a "dual spectrum" criterion for the seismic input was adopted, representative of two levels of intensity of earthquakes which might be expected in the Los Angeles area. The less intense of these was called the "damage threshold" earthquake; it was assumed to cause moderate nonstructural damage and stresses in the structural members approaching the yield level. The stronger input was called the "collapse threshold" earthquake, and was expected to produce significant structural damage but not to induce collapse. The damage threshold earthquake has about a 50 percent probability of being exceeded during the 70 year estimated life of the structure, while the collapse threshold earthquake has less than 10 percent probability of being exceeded during the same period.

The ground motions associated with these two earthquakes were represented by response spectra in the ATC-2 report. Also, because both earthquakes were expected to cause some damage in the structures, it was necessary to modify elastic response spectra to account for the inelastic behavior. For this purpose, the method developed by Newmark and Hall[11] to construct inelastic response spectra was adopted. The basic assumption of the ATC-2 procedure was that the forces developed in the structure are given by the root-sum-square superposition of the modal forces indicated by the inelastic acceleration response spectrum, while the displacements may be obtained similarly from the inelastic displacement response spectrum. Also, for simplicity it was assumed that the same damping ratio was applicable to each mode.

In order to develop inelastic response spectra by the Newmark and Hall method, elastic design spectra are first constructed. These are based on the expected maximum displacement, velocity and acceleration each amplified by an appropriate factor depending on the assumed structural damping ratio. Two different inelastic response spectra are then produced from these elastic response spectra. The inelastic displacement response spectrum is the same as the elastic spectrum in the amplified displacement and amplified velocity range; the inelastic acceleration response spectrum in these ranges is obtained by dividing the displacements by the ductility factor μ.

The amplified acceleration range of the elastic response spectrum curve is modified to obtain the inelastic acceleration response spectrum for this range by dividing by $\sqrt{2\mu - 1}$, which provides for the same energy absorption as contained in the elastic system. The high frequency (short period) range of the inelastic acceleration response spectrum is the same as the elastic response spectrum; in this range the response is governed by the maximum ground accelerations. Throughout the entire frequency range, the inelastic displacement response spectrum exceeds the inelastic acceleration response spectrum by the ductility factor μ.

The two inelastic response spectra developed in the ATC-2 project by this procedure are shown in Fig. 22. The curve labeled DTSS represents the Damage Threshold Spectrum for Strength determination and was constructed from the elastic design spectrum for the damage threshold earthquake using a damping ratio β = 5 percent and a ductility factor μ = 1.5 (representing minimal damage). The curve labeled CTSD is the Collapse Threshold Spectrum for Deformation analysis and was constructed from the collapse threshold earthquake elastic spectrum using 5% damping and a ductility factor of 2.

5.2 STRESSES RESULTING FROM THE DTSS SPECTRUM

As was stated above, the procedure proposed by the ATC-2 report to evaluate the inelastic response to an earthquake represented by the Damage Threshold Spectrum is to calculate the effective seismic forces developed in each response mode due to the DTSS by the standard elastic acceleration response spectrum method, and then to superpose the modal response quantities by taking the square root of the sum of their squares.

The total shear force calculated at each level of the building due to the DTSS applied in the Y-axis direction is shown in Figs. 23, 24, and 25 for the 3, 9, and 17 story buildings, respectively. In each figure, results are shown for the fixed base case and for the two different conditions of foundation flexibility. Also shown for comparison is the story shear force specified by the 72 Uniform Building

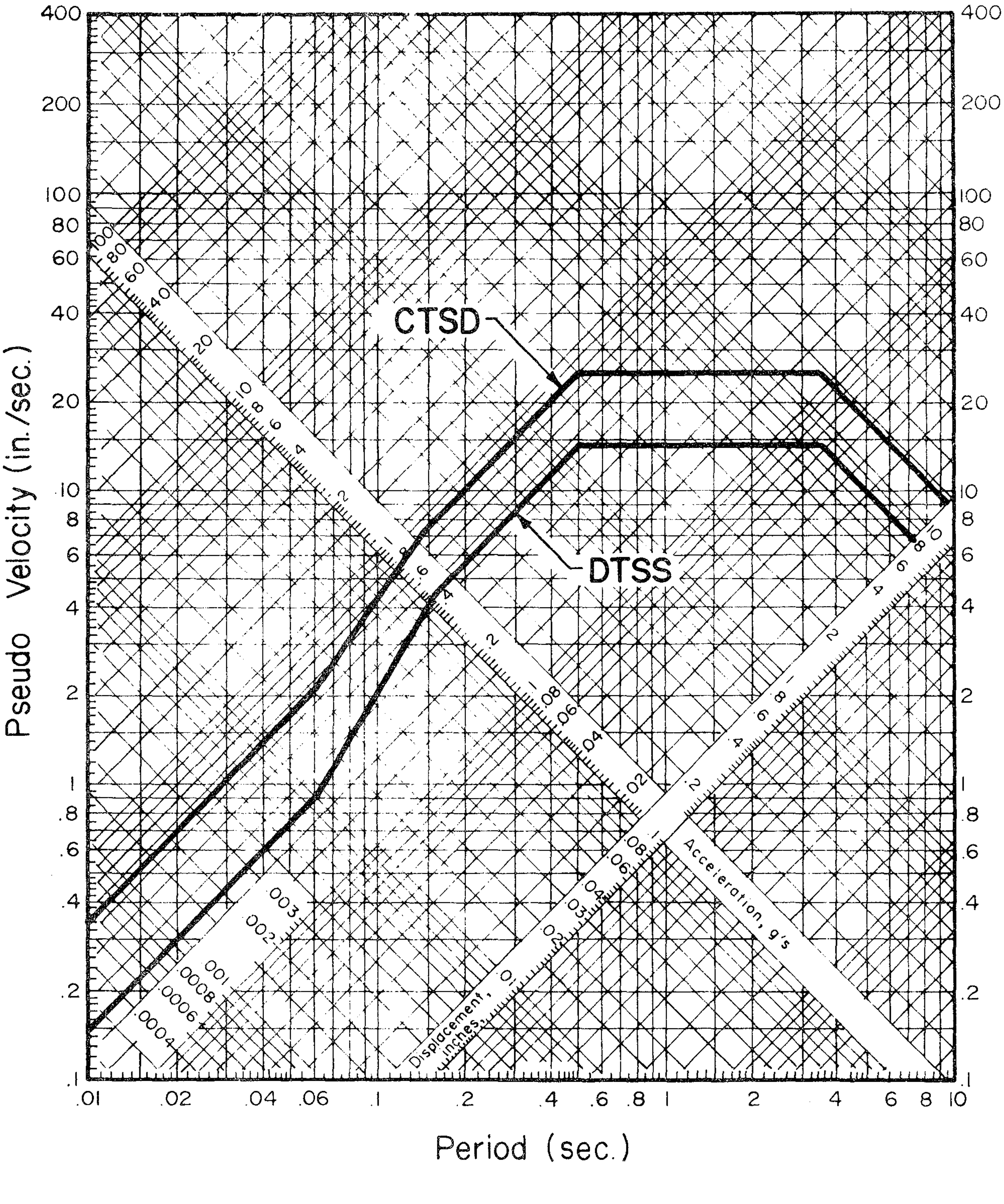


FIGURE 22 ATC-2 RESPONSE SPECTRUM

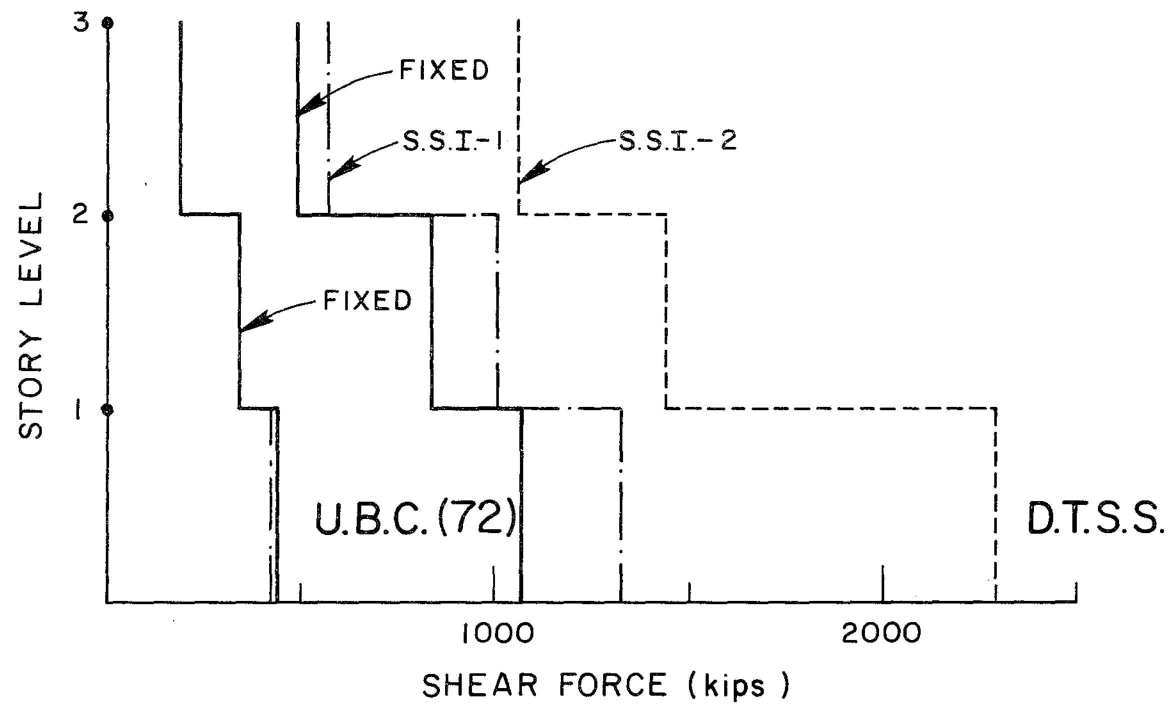


FIGURE 23 STORY SHEARS RESULTING FROM DTSS SPECTRUM FOR 3-STORY BUILDING

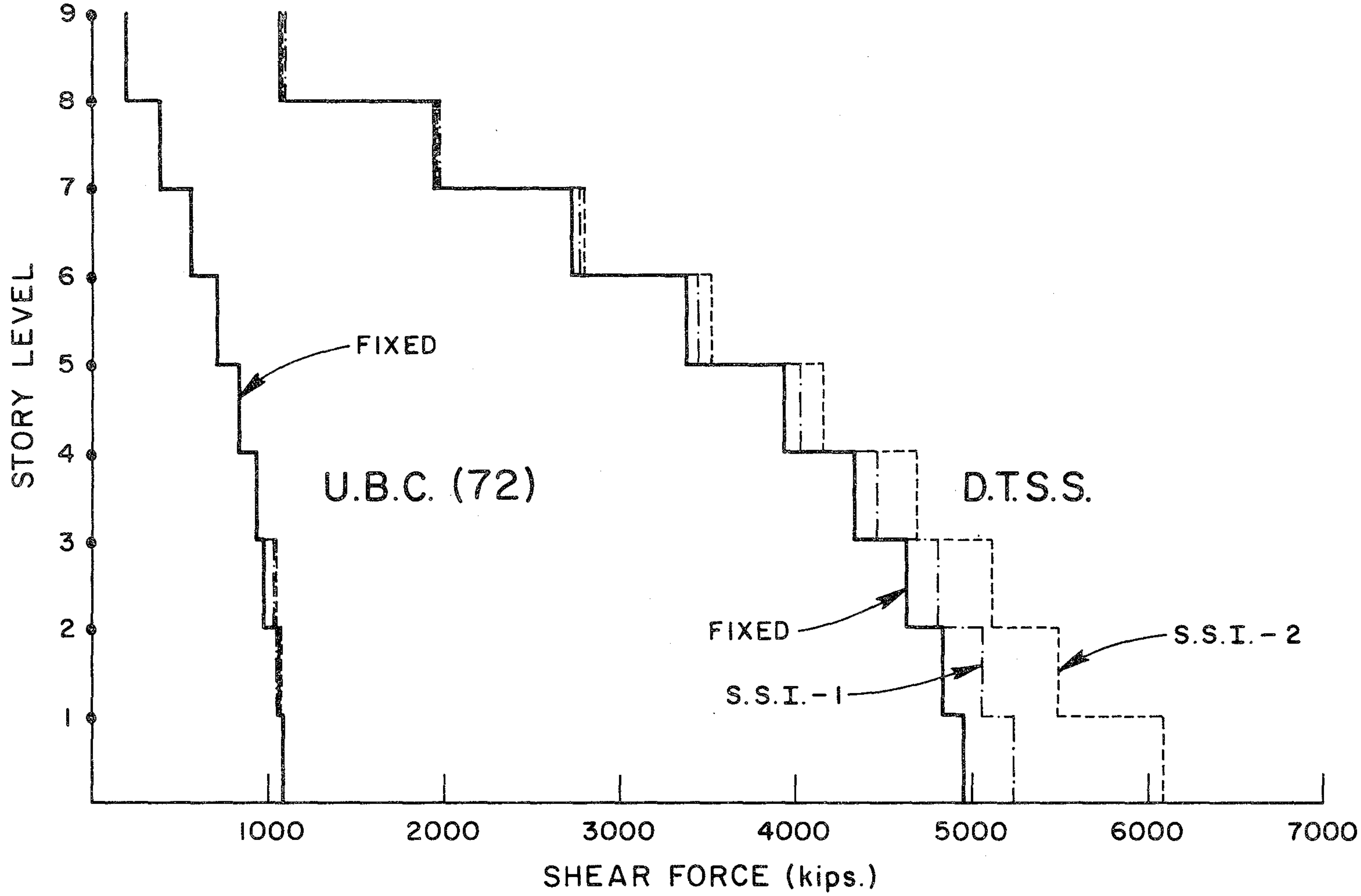


FIGURE 24 STORY SHEARS RESULTING FROM DTSS SPECTRUM FOR 9-STORY BUILDING

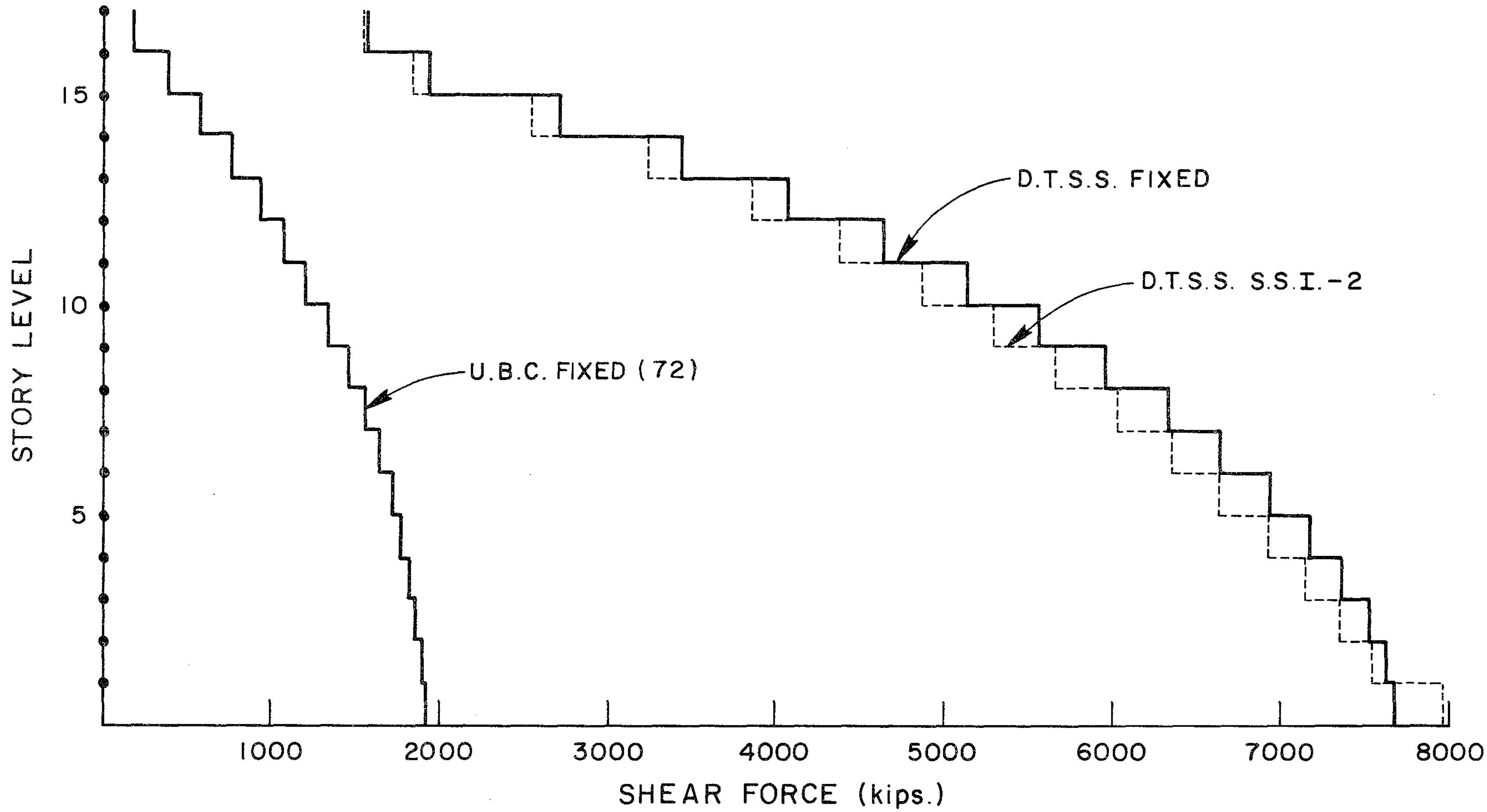


FIGURE 25 STORY SHEARS RESULTING FROM DTSS SPECTRUM FOR 17-STORY BUILDING

Code. Clearly, the DTSS represents a much more severe loading for each structure than is provided by the UBC. Moreover the foundation flexibility is seen to increase the seismic response, although its effect is greatest for the relatively stiff 3 story building, and has less effect with the taller, more flexible structures.

The total story overturning moments (OTM) calculated similarly for the three structures are shown in Figs. 26-28, together with the overturning moments resulting from the 72 and 76 UBC specified seismic loads. Again these results clearly show the greater intensity of the DTSS as compared with UBC; and also that the foundation flexibility effects cause an important increase of response for the 3 story building but not for the taller structures.

The calculated base shear forces, and the base shear coefficients derived from them for the three buildings, are listed in Table 4, to permit comparison of the UBC loads and the DTSS response. These results are used in Chapter 7 to evaluate the expected seismic performance of the buildings.

5.3 DEFORMATIONS RESULTING FROM THE CTSD SPECTRUM

The ATC-2 analysis procedure also permits evaluation of the inelastic displacements resulting from a specified earthquake. In order to evaluate the stability against collapse, the displacements resulting from the CTSD may be evaluated by the same procedure used in the analysis of response to the DTSS (see above). The effective seismic loads are calculated for each mode, and the resulting modal displacements are then superposed by the root-sum-square method.

Plots of the lateral story displacements and of the story-to-story drift calculated for the three buildings are shown in Figs. 29

to 33. Results due to the UBC loads and to the DTSS and CTSD forces are presented. Values of maximum story drift are presented in Table 5. Also, the fixed base as well as the two flexible foundation cases are included.

No failure criterion based on deformation has been defined in the present study, so the building performances have not been evaluated with respect to this factor. However, it is recognized that deformations are a most important aspect of the building behavior, and that some form of drift limit undoubtedly should be included in the design criteria. Further consideration will be given to this factor in the future.

TABLE 4. BASE SHEAR FORCES AND SHEAR COEFFICIENTS

Building	Case	UBC		DTSS	
		Base Shear (kips)	Shear Coefficient*	Base Shear (kips)	Shear Coefficient*
3-story	FIX	441.6	0.098	1058.2	0.235
	SSI-1			1323.6	0.294
	SSI-2			2282.5	0.507
9-story	FIX	1089.8	0.081	4959.1	0.367
	SSI-1			5299.1	0.393
	SSI-2			6092.4	0.451
17-story	FIX	1913.1	0.077	7683.3	0.311
	SSI-2			7989.6	0.323

*Total weights of buildings are 4,500 kips (3-story), 13,500 kips (9-story) and 24,720 kips (17-story)

TABLE 5. MAXIMUM STORY DRIFT

(unit: inch)

Building	Base Cond.	UBC(72)	DTSS	CTSD
3-story	FIX	0.0040	0.0072	0.0187
	SSI-1	0.0051	0.0119	———
	SSI-2	0.0064	0.0217	———
9-story	FIX	0.0174	0.0599	0.1038
	SSI-1	0.0181	0.0623	———
	SSI-2	0.0194	0.0748	———
17-story	FIX	0.0513	0.1662	0.2765
	SSI-1	———	———	———
	SSI-2	0.0532	0.1642	———

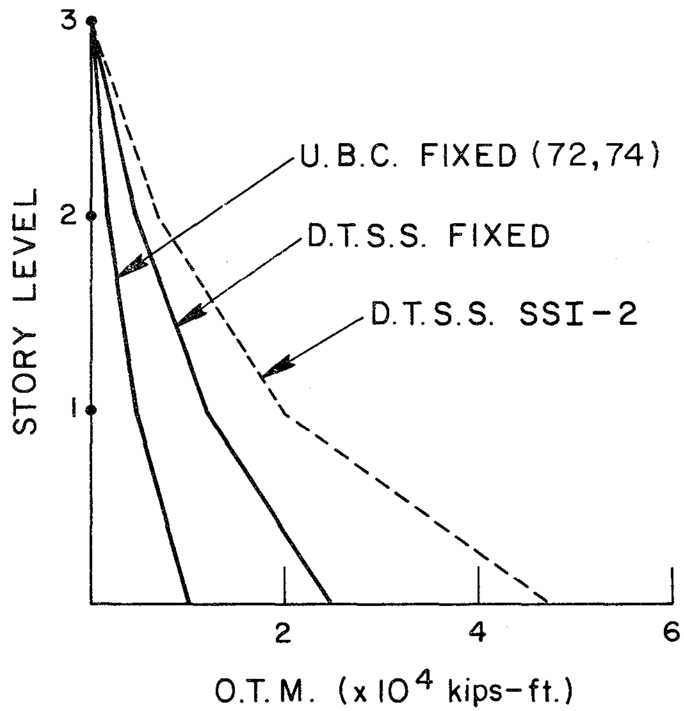


FIGURE 26 O.T.M. RESULTING FROM DTSS SPECTRUM FOR 3-STORY BUILDING

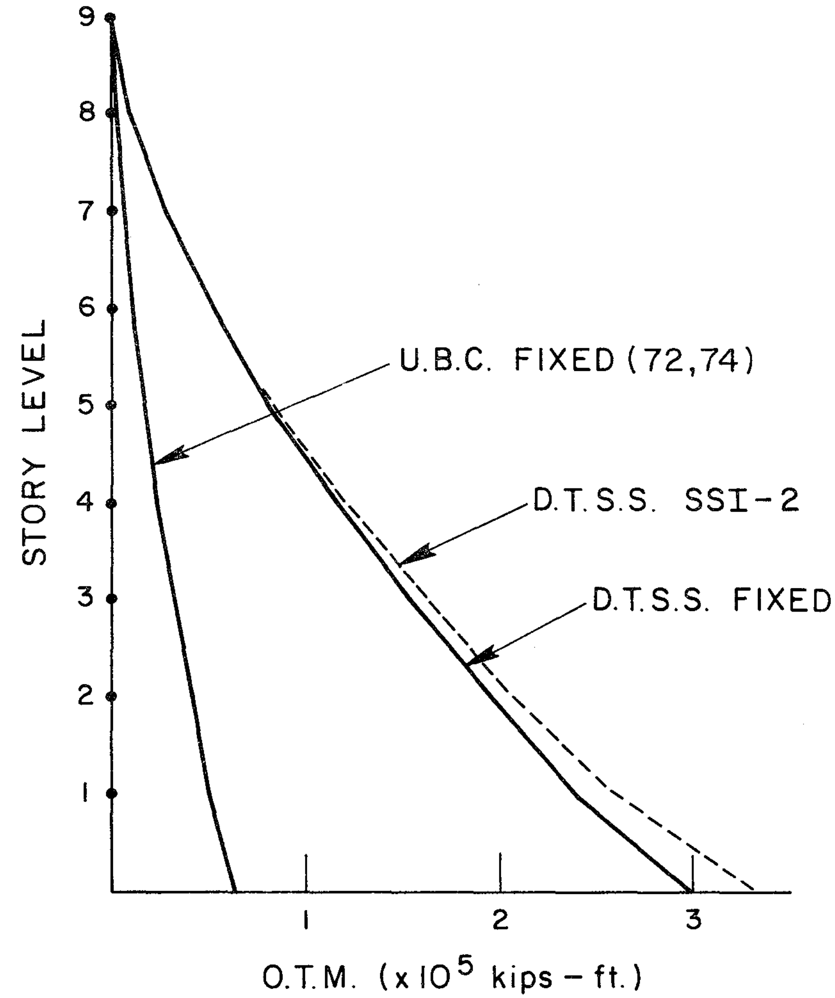


FIGURE 27 O.T.M. RESULTING FROM DTSS SPECTRUM FOR 9-STORY BUILDING

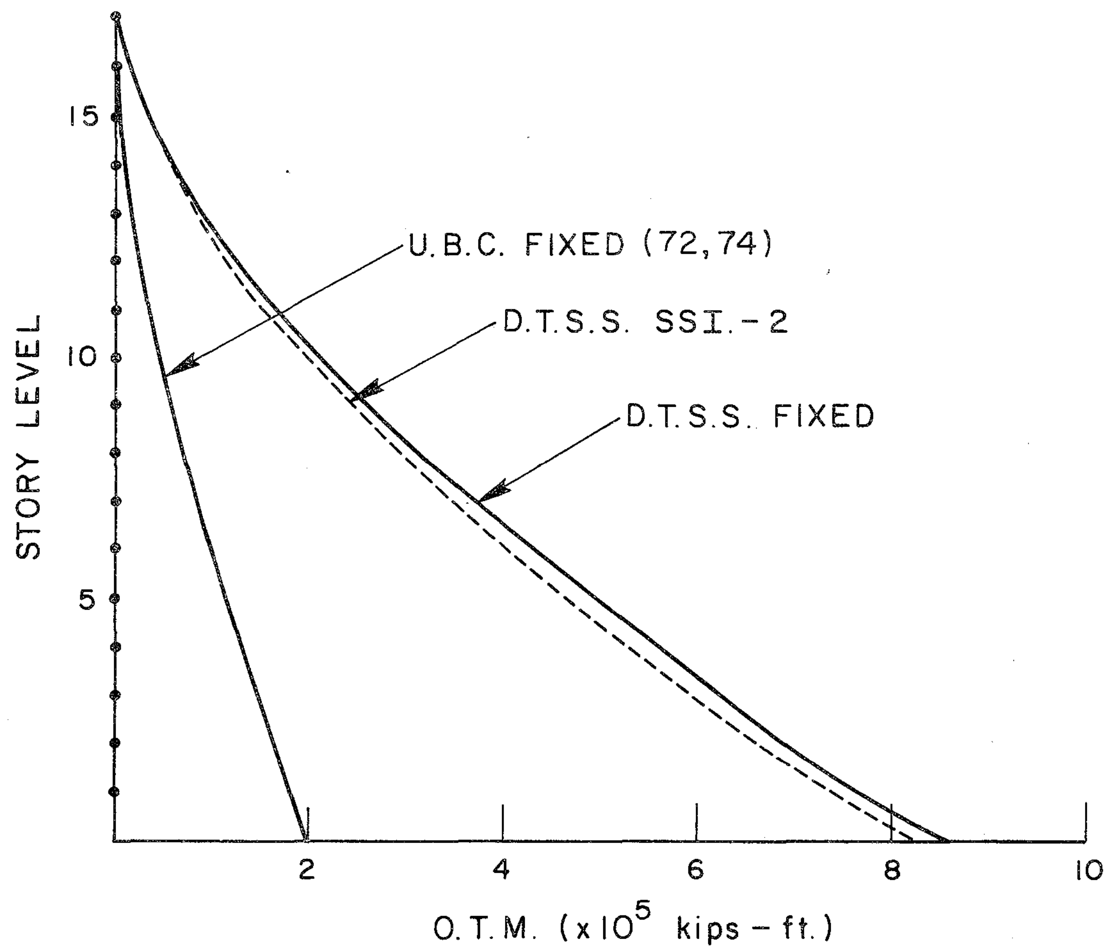


FIGURE 28 O.T.M. RESULTING FROM DTSS SPECTRUM FOR 17-STORY BUILDING

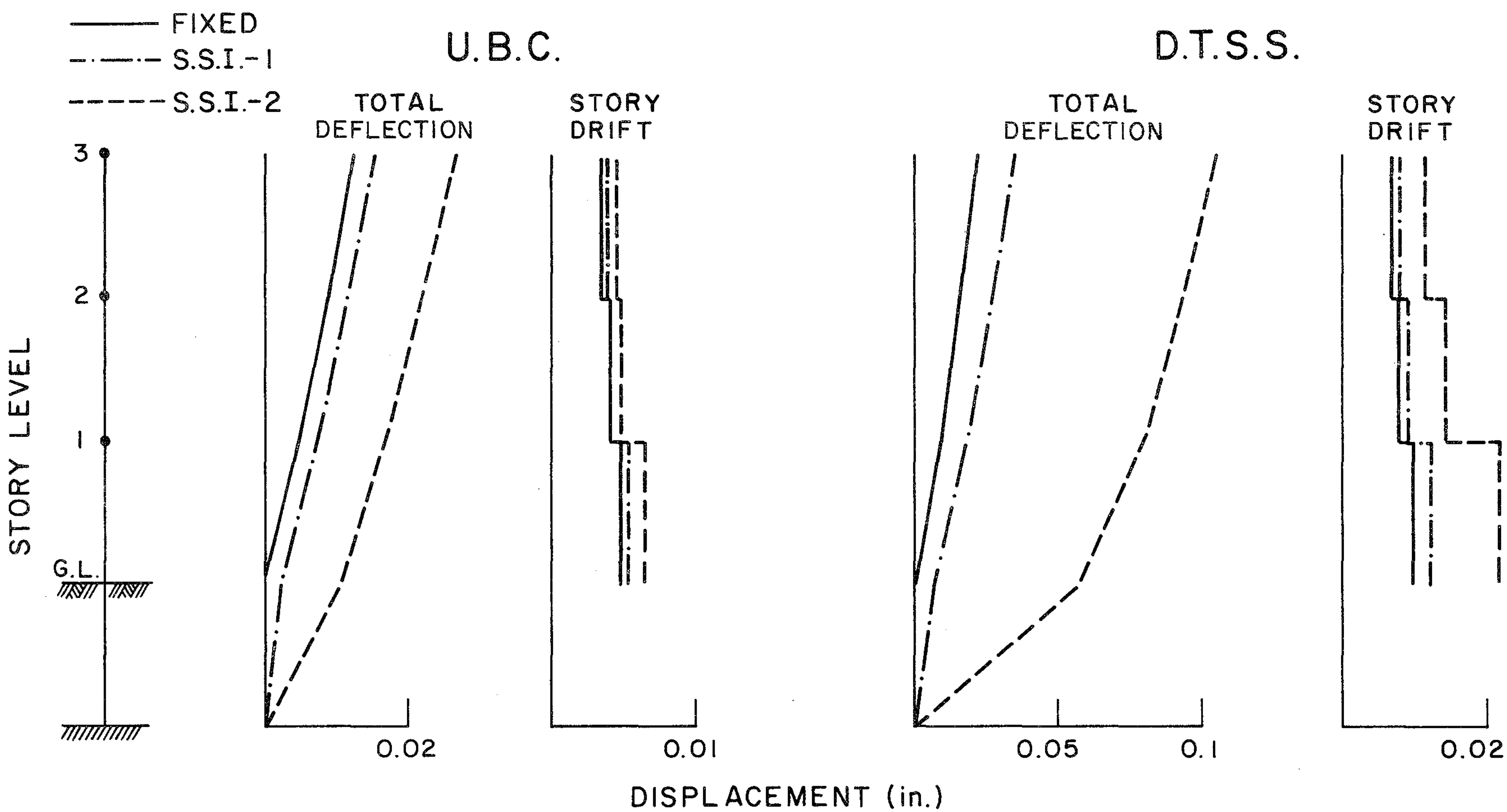


FIGURE 29 COMPARISON OF LATERAL DEFLECTIONS AND STORY DRIFTS RESULTING FROM THE 72 UBC AND DTSS SPECTRA FOR THE 3-STORY BUILDING

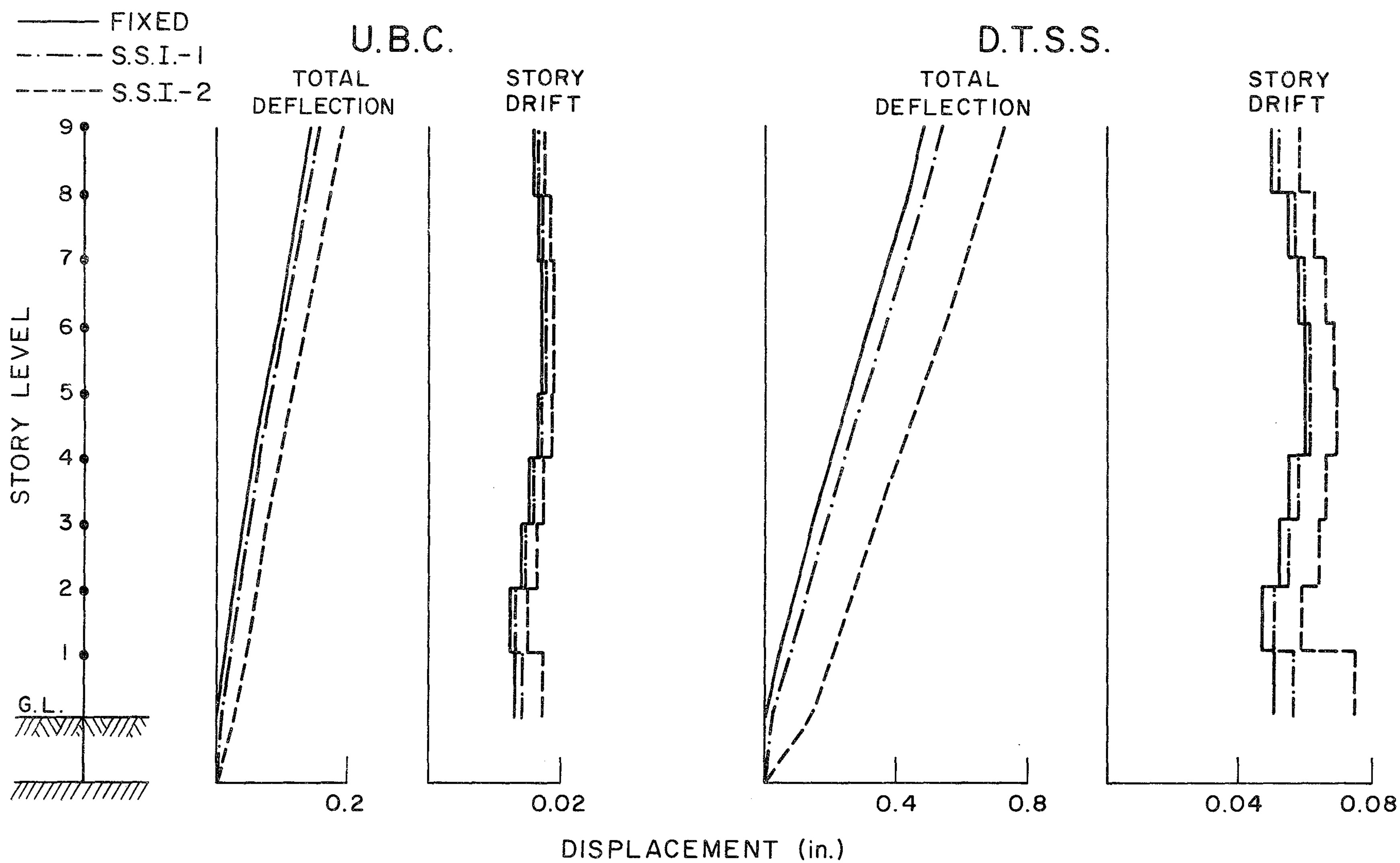


FIGURE 30 COMPARISON OF LATERAL DEFLECTIONS AND STORY DRIFTS RESULTING FROM THE 72 UBC AND DTSS SPECTRA FOR THE 9-STORY BUILDING

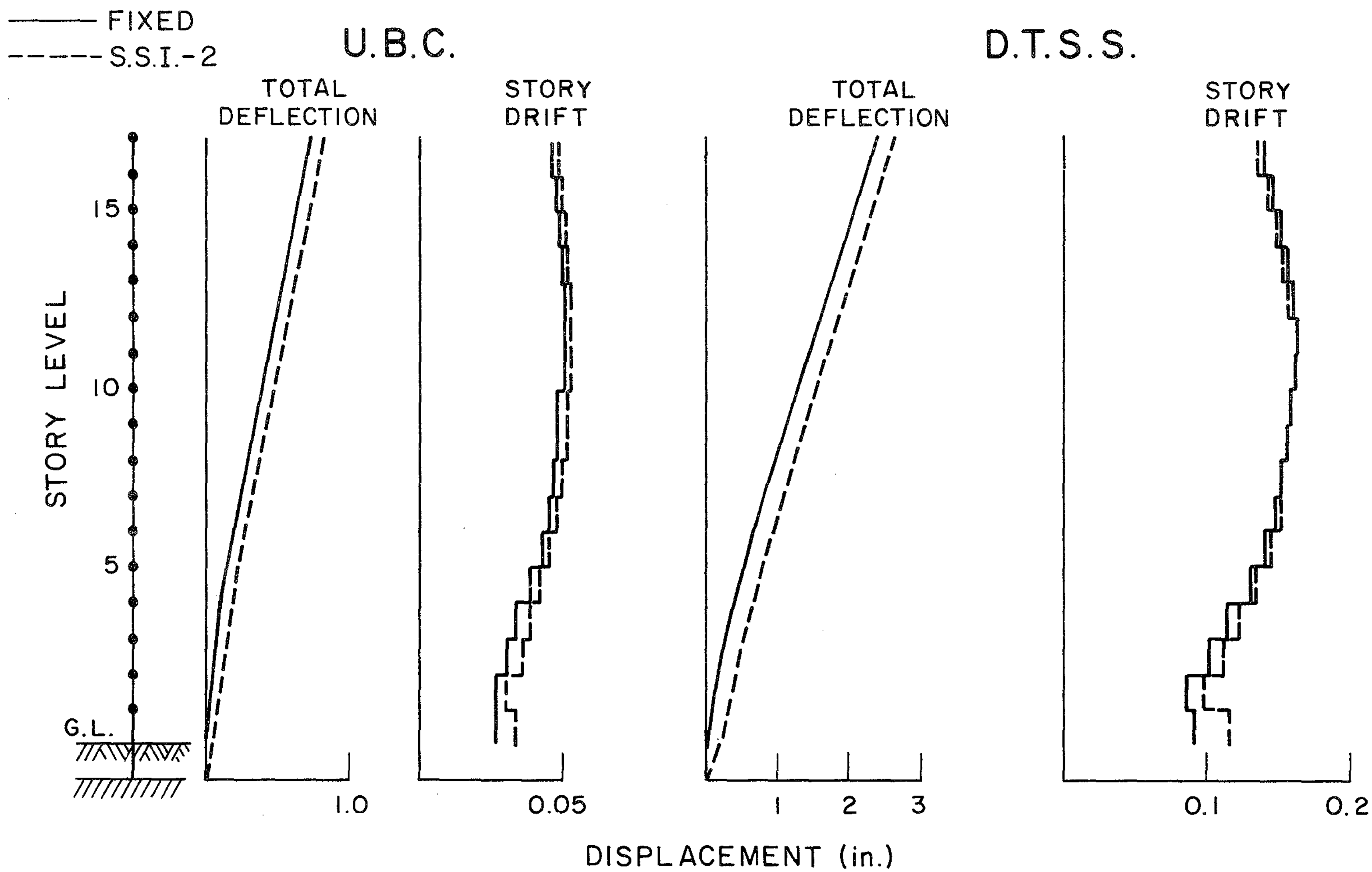


FIGURE 31 COMPARISON OF LATERAL DEFLECTIONS AND STORY DRIFTS RESULTING FROM THE 72 UBC AND DTSS SPECTRA FOR THE 17-STORY BUILDING

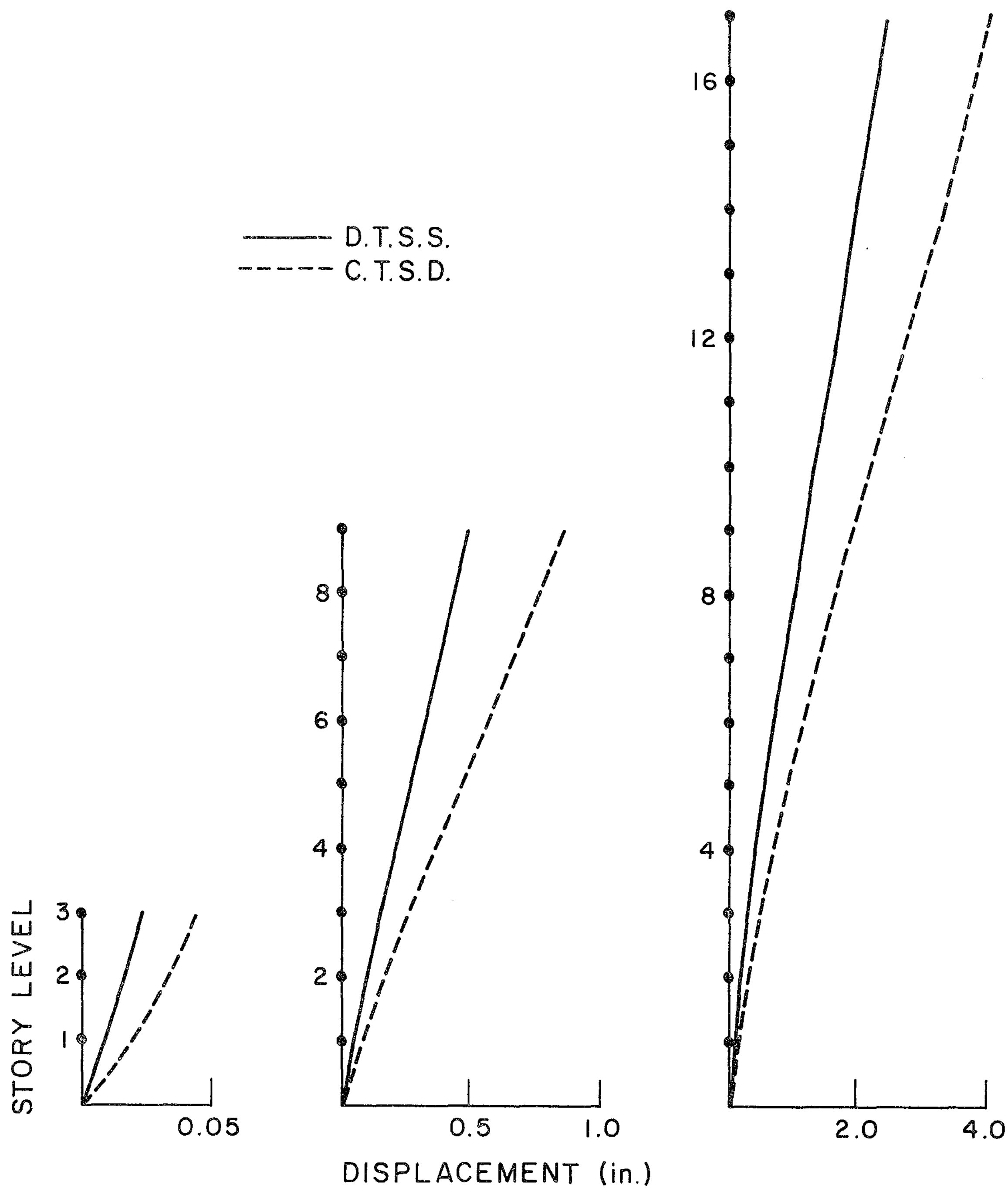


FIGURE 32 LATERAL DEFLECTIONS RESULTING FROM THE DTSS AND CTSD SPECTRA (FIXED CASE)

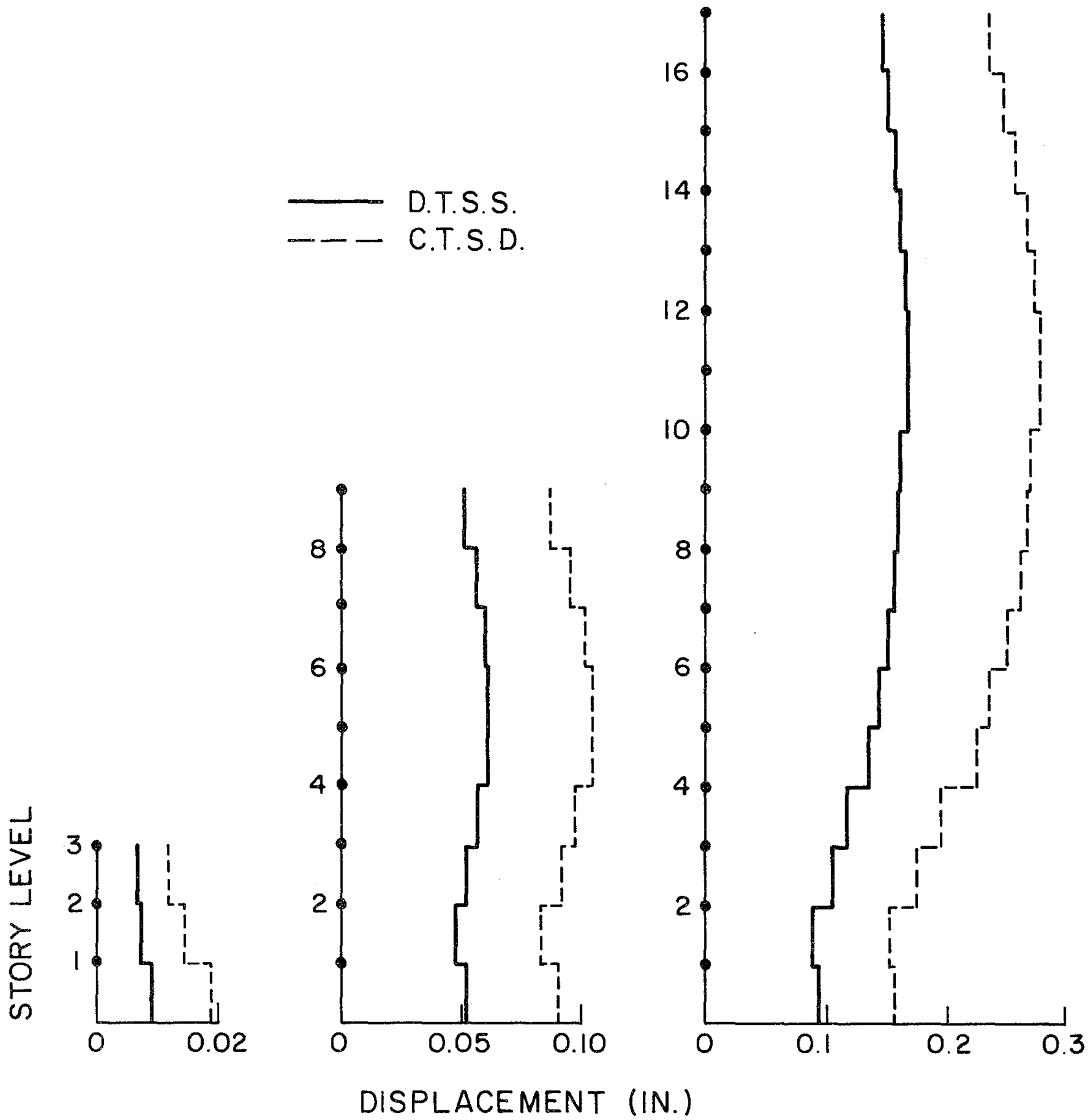


FIGURE 33 STORY DRIFTS RESULTING FROM THE DTSS AND CTSD SPECTRA (FIXED CASE)

6. SIMPLIFIED FAILURE CRITERIA AND TENSILE STRENGTH OF MASONRY PANELS

6.1 INTRODUCTION

Although investigations are currently in progress to determine a reliable failure criterion for masonry structural elements, it was necessary to assume a simplified criterion for the purpose of this study. Obviously the failure criterion was limited by the assumptions inherent in the analysis. If a detailed finite element analysis had been performed, the stress distribution in the lower level panels would have been obtained directly and no assumptions with regard to the shear and normal stress distributions would have been required. However, because an equivalent frame analysis was performed, only the typical beam theory stresses were available from the analysis. Therefore, it was necessary to define the failure criterion with respect to these stresses, as explained below.

6.2 SIMPLIFIED FAILURE CRITERION

The dynamic analysis of the buildings based on an equivalent frame model led to values of axial force, shear, and bending moment in each member. According to elementary beam theory, the shear forces are assumed to be associated with a parabolic variation of shear stress across the member sections, while the axial forces and moments are assumed to lead to a linear variation of normal stresses across the sections. These stress distributions were depicted in Fig. 2.

The failure mechanism assumed for this study is that failure will occur when the principal tensile stress at the most highly stressed point in a member exceeds the tensile strength of the material. In general, it may be assumed that the critical tensile stress occurs at the center of the shear wall panels, at which point the horizontal shear stress is 1.5 times the average shear stress τ and the normal stress σ_c is equal to the resultant vertical force divided by the cross-sectional area. Based on a Mohr's circle analysis, the principal tensile stress at this point is

$$\sigma_t = \sqrt{(1.5\tau)^2 + (\sigma_c/2)^2} - \sigma_c/2 .$$

Therefore, failure is assumed to occur when this stress exceeds the tensile strength of the shear wall masonry.

6.3 TENSILE STRENGTH OF PANELS OBTAINED FROM TEST DATA

The sizes of the interior panels in the buildings under study vary between 16 feet and 40 feet in length, and are 8 ft 6 in high. No laboratory tests have been performed on panels of this size, so an estimate of their strength can only be obtained from other types of test data. The significant experimental work has been discussed in Reference 5; a summary of this material follows.

6.3.1 Results from Diagonal Compression Tests

Probably the most convenient experimental procedure for evaluating the tensile strength of masonry is the diagonal compression test, shown in Fig. 34. This test has been used by a number of researchers, either with or without the supplementary vertical load which produces the compressive stresses σ_c. The tests are performed by applying the vertical load, if any, and then increasing the diagonal compression load P_d until failure occurs. The tensile strength is assumed to be given by the principal tensile stress existing when the failure occurred. However, the principal tensile stress resulting from these test conditions has been calculated on the basis of a variety of different assumptions.

The simplest assumption is that the test produces a uniform state of stress in the test specimen, in which case the principal tensile stress may be determined from a Mohr's circle analysis to be

$$\sigma_{t_{cr}} = \sqrt{\tau^2 + (\sigma_c/2)^2} - \sigma_c/2 \qquad \text{(I)}$$

where τ is the shear stress produced by the diagonal load (given by $P_d/\sqrt{2}\,A$) and σ_c is the vertically applied normal stress. A more thorough study of the stress distribution in the test panel, based largely on photoelastic analyses and assuming homogeneous isotropic material, leads to the following formula for principal tensile stress

$$\sigma_{t_{cr}} = \sqrt{2.424\ \tau^2 + (\sigma_c/2)^2} - \left(\frac{\sigma_c}{2} + 0.823\ \tau\right) \qquad \text{(II)}$$

where σ_c and τ are as defined above.

Blume(12) and Borchelt(13) carried out experimental studies of masonry by the diagonal compression approach. Blume tested both double wythe grouted brick and hollow clay brick specimens, 4 ft by 4 ft, but did not apply any vertical compressive load σ_c. Borchelt tested single wythe brick specimens using both normal and high strength mortar and applying a vertical compressive stress. Results of these tests are summarized in Table 6. It will be noted here that results have been obtained using both formulas I and II; actually Borchelt used I and Blume used II so it was necessary to make this type of adjustment to compare their results.

TABLE 6. RANGE OF CRITICAL TENSILE STRENGTHS

DIAGONAL COMPRESSIVE TESTS

	BORCHELT'S FORMULATION		BLUME'S FORMULATION	
	LOWER BOUND	UPPER BOUND	LOWER BOUND	UPPER BOUND
DOUBLE WYTHE GROUTED BLUME (12)	175	425	130	210
SINGLE WYTHE SOLID BRICK BORCHELT (13)	175	620	85	410
HOLLOW CLAY BRICK BLUME (12)	125	390	90	290

RACKING TESTS

	UNIFORM SHEAR DISTRIBUTION		PARABOLIC SHEAR DISTRIBUTION	
DOUBLE WYTHE GROUTED PRIESTLEY (15)	100	110	150	190
HOLLOW CLAY BRICK WILLIAMS (14)	70	140	130	250
CONCRETE BLOCK MAYES AND CLOUGH (16)	80	180	130	230

All values are based on the net area.

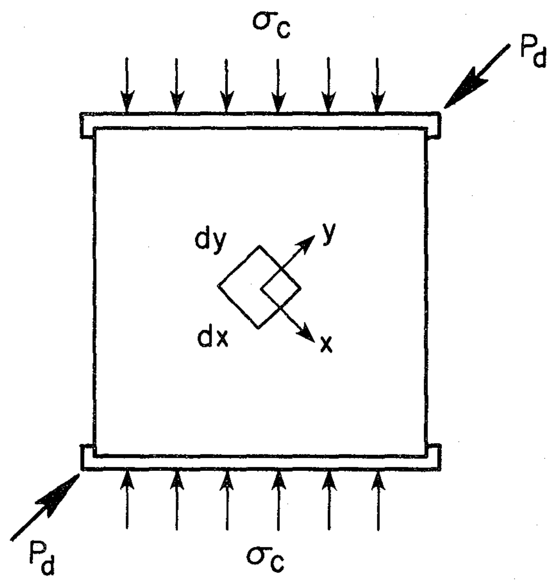


FIGURE 34 EDGE LOAD WITH RACKING TEST

6.3.2 Results from Cantilever and Fixed Ended Piers

The other principal experimental procedure used to evaluate the strength of masonry is the shear wall racking test, in which a wall panel is anchored at the base and subjected to a horizontal load at the top edge. Various systems have been employed to develop resistance to the overturning moments resulting from the racking loads--external straps, internal vertical reinforcing, or a double pier specimen in which the spandrel girders provide fixity at top and bottom of the shear panels. In order to compare the results of these shear panel tests with those obtained from diagonal compression tests, the principal tensile stress at the center of the panel was obtained from the Mohr's circle analysis mentioned earlier for analysis of complete buildings, as follows:

$$\sigma_{t_{cr}} = \sqrt{(1.5\ \tau)^2 + (\sigma_c/2)^2} - \sigma_c/2\ . \qquad \text{(III)}$$

The only difference between this formula and formula I above is that the horizontal shear stress at the center of the shear wall panel is assumed to be 1.5 times the average shear stress τ. Critical tensile stresses for shear wall panel tests were evaluated with both formulations (I and III) to indicate the significance of this assumption on the shear stress distribution.

Average critical tensile stresses obtained in three different test programs using shear wall type specimens also are tabulated in Table 7. The work by Williams[14] involved hollow clay brick walls, the work by Priestley and Bridgeman[15] was on double wythe grouted brick walls, and the Mayes and Clough tests[16] employed double fixed-ended piers.

TABLE 7. COMPARISON OF THE RATIO B FOR THE LOWEST LEVEL PANELS IN THE 3,9 AND 17 STORY BUILDINGS

$$B = \frac{\text{CRITICAL TENSILE STRENGTH (130 PSI)}}{\text{MAXIMUM TENSILE STRESS FROM REALISTIC EARTHQUAKE}}$$

BUILDING	RATIO B			
	P-2*	P-3	P-11	P-12
3	4.69	3.69	3.77	7.18
9	0.75	0.48	0.48	0.98
17	0.48	0.26	0.24	0.51

*P-2, P-3, P-11, P-12 are identified in Figure 19

7. EXPECTED PERFORMANCE OF THE BUILDINGS

The best evaluation of the expected performances of the selected buildings subjected to a realistic earthquake would be provided by an inelastic time history analysis, using a failure criterion that is related to both the ultimate strength and the deformation capacities of the panels. In this way, both failure (extensive cracking) and collapse of a structure could be predicted. Unfortunately such analyses cannot be carried out at present, although the development of these techniques is the basic objective of an extensive research project currently being performed at the University of California, San Diego. For the purpose of the present study, the simplified failure criterion described in Chapter 6 was utilized to evaluate the expected performances of the buildings. Provided that no panels in a particular building are predicted to fail according to this criterion, the performance of the building is considered adequate. If one or more panels fail, the expected performance is not adequate, although inadequate performance may not necessarily mean collapse or complete failure of the building. Interpretation of the failure mechanism requires judgment, and will depend upon the number of panels failing according to the criterion and the amount by which each panel exceeds the definition of failure.

As was noted earlier, the expected performance of the masonry building was determined by comparing the maximum tensile stress calculated in the lower level shear wall panels, due to the design earthquake (Chapter 5) with the critical tensile strength of the masonry determined by various experiments (Chapter 6 - Table 6). To ensure a conservative estimate of the performances of the buildings, the lower bound of the test data shown in this table was taken as the available tensile strength of masonry -- 130 psi. Then the factor of safety with respect to the material behavior was defined as the ratio B (Eq. 8), representing the ratio of available material strength to the maximum stress calculated from the realistic earthquake. A value of B greater than one indicates an adequate factor of safety of the material, a value less than one suggests that the material would be expected to fail.

The values of the ratio B computed for the lower level panels of each building are listed in Table 7. It is clear from this table that the three story building has adequate strength for the realistic earthquake loads, whereas the nine story building is developing slight overstresses in several locations. The seventeen story building clearly shows significant overstress, and would be expected to fail during this expected earthquake.

The fact that this building is expected to fail during the realistic earthquake would indicate that the design code is inadequate if the building had been designed carefully to the minimum standards permitted by the code. However, the building was not actually designed according to any of the codes considered in this study, so the value of the ratio B by itself cannot indicate the adequacy of these codes. Therefore it was necessary to devise a comprehensive evaluation procedure for the purpose of this study; that procedure is described in the following chapter.

8. EVALUATION OF RECENT AND CURRENT BUILDING CODES

As was indicated in the Introduction, the lack of relevant information on the dynamic strength of masonry structural assemblages and the associated uncertainty concerning the seismic behavior of masonry buildings has led to substantial changes in the Uniform Building Code in the past five years. An obvious method of evaluating the various changes that have been introduced would be to design a typical building optimally for each code and to determine the expected performance of each building during a realistic earthquake by the method used in the preceding section.

However, the design of different buildings for each of the various code requirements is a time consuming effort which was beyond the scope of the present study. Instead, it was decided to consider only a single set of building designs and to evaluate two factors of safety for each building: "A" representing the factor of safety relative to the strength prescribed by the various codes, and "B" indicating the expected factor of safety relative to the demand imposed by a realistic earthquake. Then the ratio of these factors of safety (B/A) is indicative of the adequacy of the code used in determining A; if B/A is less than one, the code is not adequate, if B/A exceeds one, the code is more than adequate.

The factors of safety A and B were defined by Eqs. 6 and 8 respectively, and their values computed for the various buildings and codes are listed in Tables 5 and 7, respectively. Values of the ratio B/A for each case, listed in Table 8, therefore provide an indication of the changing quality of the code requirements during recent years.

TABLE 8. CODE EVALUATION RATIO B/A (Based on the lowest level shear panels)

	B/A (3-story)				B/A (9-story)				B/A (17-story)			
CODE	P-2*	P-3	P-11	P-12	P-2	P-3	P-11	P-12	P-2	P-3	P-11	P-12
72 UBC	0.68	0.56	0.57	0.90	0.28	0.18	0.18	0.30	0.31	0.17	0.15	0.27
73 UBC	0.93	0.76	0.77	1.23	0.38	0.24	0.24	0.41	0.43	0.23	0.27	0.37
74 UBC	0.70	0.57	0.58	0.92	0.28	0.18	0.18	0.31	0.32	0.17	0.16	0.28
76 UBC	1.08	0.87	0.90	1.44	0.63	0.40	0.40	0.68	0.65	0.35	0.32	0.57
ATC-3	1.04	0.84	0.86	1.37	0.59	0.38	0.38	0.65	0.70	0.38	0.35	0.61

*P-2, P-3, P-11 and P-12 are identified in Figure 19

9. DISCUSSION OF RESULTS

Table 8 shows that the ratios B/A for all interior lower level panels of both the nine and seventeen story buildings are less than one for the 72, 73, 74 and 76 Uniform Building Codes and the proposed ATC-3 Code. This indicates that none of these codes provide adequate protection against damage to these buildings from the realistic earthquake considered in this study.

The values of B/A for the three story building in the 76 UBC and proposed ATC-3 code are greater than one for three of the six interior lower level panels and less than one for the other three. This suggests that within the limits of this study the 76 UBC and proposed ATC-3 code provide reasonable protection against severe damage in low masonry buildings.

In the proposed ATC-3 code the code allowable stresses of Table 9A-5[4] are multiplied by 2.5ϕ, where ϕ is 0.6 if horizontal reinforcement carries all the shear. If ϕ were reduced to 0.3 the values of B/A for three of the six interior lower level panels would be greater than one and less than one for the other three. This suggests that if this change were made the proposed ATC-3 code would provide reasonable protection against severe damage in masonry buildings of moderate height.

Although the study considered the effect of a realistic earthquake occurring in a highly active seismic region only, inferences for less seismically active regions can be drawn from these results. For example, in seismic zone 2 of the Uniform Building Code the zone factor Z equals 0.5. The consequence of this reduction in design loads is an increase in the ratio A by a factor of 2, because the strength of the structure is not changed. Although a representative response spectrum for a typical site in seismic Zone 2 was not considered in this study, it is expected that it would define significantly lower seismic loads than those from the spectrum obtained from the ATC-2 study which was used here to calculate the ratio B. If a representative spectrum for Zone 2 gave seismic coefficients approximately half of the ATC-2 spectrum, then the ratio B would be increased by a factor of approximately 2.0. In this case, the increase in both A and B, would cancel when the ratio B/A is calculated leaving the end results of this study unchanged.

Consequently, the values of B/A which are presented in Table 8 for the most active seismic regions in the codes also may be indicative of results that would be obtained for regions of lower seismic activity. To obtain a more refined calculation of B/A for other seismic zones a representative spectrum for a typical site would be required but the preceding discussion shows qualitatively the general applicability of the inferences drawn in this study.

In conclusion, it is apparent that the trend towards increasing conservatism which is evidenced in recent code changes concerning masonry structures is justified. Moreover the study suggests that the codes should be more conservative for masonry buildings of moderate height.

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