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EVALUATION OF SEISMIC DESIGN PROVISIONS FOR MASONRY IN THE UNITED STATES

by

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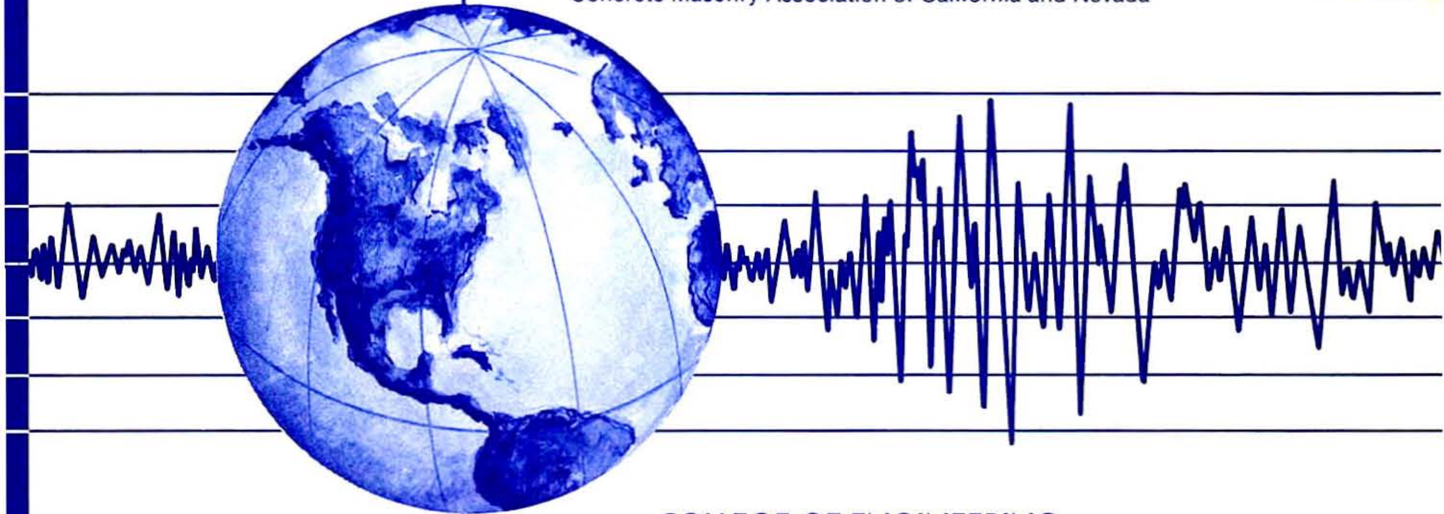

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ABSTRACT

This report presents an evaluation of two sets of seismic design provisions for masonry construction in the United States - namely, the 1979 Uniform Building Code and the ATC-3-06 "Tentative Provisions for the Development of Seismic Design Regulations for Buildings".

The method of evaluation is based on an Over-Design Ratio which compares the shear wall area required to resist code loads with that required to resist realistic earthquake loads. The latter area is determined from test results from the continuing masonry research program at the Earthquake Engineering Research Center, University of California, Berkeley. A summary of the test results is included in the report.

The report also contains a comparison of the shear wall areas required by the two sets of seismic provisions, and changes to both sets of provisions are suggested.

ACKNOWLEDGEMENTS

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1. INTRODUCTION

Masonry is the oldest and most traditional of all the construction materials currently in use. The type of masonry unit has changed substantially over the centuries, but the fundamental concept of a masonry unit joined by a bonding material is still the basic form of masonry construction. There are numerous examples of buildings in Europe that attest to the longevity of masonry buildings; and because of this long history of design and construction it would seem logical to assume that design codes for masonry buildings would be well established and widely accepted. Unfortunately, this is not the case.

All of the early European masonry buildings were based on trial and error methods of construction. As engineering insight developed, engineers could show explicitly why the methods of construction used in the past worked. Then the conservative methods of trial and error construction were refined and less massive forms of masonry construction resulted.

During the last three decades our knowledge of earthquake engineering has increased significantly, primarily as a result of increased research activity in areas such as geology, seismology, soil dynamics, analytical techniques, material behavior and structural performance. A major part of the research in materials has been concentrated on steel and reinforced concrete building components. Research on the dynamic characteristics of masonry structural components has significantly lagged behind that of other construction materials. However, in the past eight years masonry research activity has increased substantially and if this increased effort continues,the seismic performance of masonry buildings and structural components will be more thoroughly understood in the coming decade.

Despite lack of knowledge, building codes in seismic areas must address the design of masonry buildings. The question that has to be considered is "Are seismic design provisions adequate, and what margin of safety is inherent in them?" An attempt to address this question was carried out as part of a continuing masonry research program at the Earthquake Engineering Research Center, University of California, Berkeley, and is described in this report. A previous attempt in 1976 based on the limited data available at that time, is presented in an earlier report entitled "Expected Performance of Uniform Building Code Designed Masonry Structures". In the intervening five years much has been learned, and the objective of this report is to summarize this information and use it to evaluate seismic design provisions, both current and proposed, for masonry buildings in the United States.

Two sets of seismic design provisions are evaluated here - the 1979 Uniform Building Code (UBC) and the ATC-3-06 "Tentative Provisions for the Development of Seismic Design Regulations for Buildings". The method of evaluation of a set of provisions (or code) for load bearing masonry buildings is to compare the required area for shear resistance of the code design with that derived from the state-of-the-art. (The ratio of these required areas is called the Over-Design Ratio.) The code required shear area is taken to be the ratio of the code design seismic force to the code recommended masonry unit stress. The area derived from the state-of-the-art is the ratio of a "realistic" earthquake force, obtained from the response spectrum of earthquake ground motion studies, to the recommended stress determined from the Berkeley test program.

In Chapter 2 the method used to evaluate the design provisions is discussed. The results of the Berkeley test program are summarized, and then used to determine ultimate shear strengths for masonry piers. In Chapter 3 various comparisons are made between the two sets of provisions, and the Over-Design Ratios are determined. Story shears, overturning moments and Over-Design Ratios are given in Chapter 4 for each of a 3, 9 and 17-story building. Conclusions from the evaluation are presented in Chapter 5.

2. METHOD OF EVALUATION AND SUMMARY OF RESEARCH RESULTS

2.1 Introduction

At the present time there is one code governing seismic design of masonry, the Uniform Building Code, and another set of provisions, ATC-3-06 "Tentative Provisions for the Development of Seismic Regulations for Buildings", which is tentative in that it has not yet been adopted. Both provisions are of necessity somewhat empirical. This circumstance raises questions about their adequacy for safe earthquake design and makes an appraisal of their provisions desirable.

It is not immediately apparent what should be the basis for such an appraisal. We were guided in our choice by significant advances in knowledge in two areas; namely, the force at each floor that must be resisted in shear in a multi-story masonry building, and the ability of different types of masonry to resist these forces. The first comes from earthquake response spectra which reflect the state-of-the-art in ascertaining the horizontal force imposed by an earthquake, and analysis programs which indicate how this force should be distributed floor-by-floor. The second is the result of an extensive experimental program on the seismic resistance of masonry conducted at the Earthquake Engineering Research Center of the University of California.

Accordingly, the appraisal of a code is made by comparing the area of masonry required at a particular floor of a building, using a particular kind of masonry, as ascertained from the particular code provision, with the area as ascertained using the state-of-the-art (or realistic) force and stress capability of the masonry from the experimental program. The ratio of the code area to the "realistic" area we term the Over-Design Ratio (ODR).

The ODR is described more fully in Section 2.2. The way in which the floor forces are established using each of the two design provisions is described in Section 2.3 and the provisions for allowable shear stresses for seismic loads appear in Section 2.4.

The "realistic" earthquake floor force is explained in Section 2.5, and Section 2.6 gives a summary of the results of the Berkeley test program. In Section 2.7 the ultimate shear strengths and associated ductility factors, used in the study, are developed.

2.2 Over-Design Ratio

The area of masonry wall at a particular floor required to resist a given load is the load divided by the shear stress capacity of the masonry.

$$\text{Area Required} = \frac{\text{Load}}{\text{Shear Stress Capacity}} . \tag{2.1}$$

In what follows we calculate this area three times for each floor of each building. The first time using the provisions in ATC-3-06, the second using the Uniform Building Code, and the third time using a "realistic" horizontal load and shear stress capacity obtained from the Berkeley test program. So that

$$\text{A code} = \frac{\text{F code}}{\sigma \text{ code}} \qquad \text{and} \qquad \text{A real} = \frac{\text{F real}}{\sigma \text{ exp.}} .$$

To make a code assessment we form the ratio

$$\frac{\text{A code}}{\text{A real}} ,$$

and call it the Over-Design Ratio (ODR). It follows then, that

$$\text{ODR} = \frac{\text{F code}}{\sigma \text{ code}} \Big/ \frac{\text{F real}}{\sigma \text{ exp}} = (L_c/R_c)/(L_{eq}/R_{eq}) \tag{2.2}$$

where

L_c = F code - force specified by a code

R_c = σ code - shear stress allowed by a code

L_{eq} = F real - force resulting from realistic earthquake

R_{eq} = σ exp. - ultimate shear stress evaluated from the test program.

If the ODR is greater than one, the code under study is conservative, if it is less than one, its provisions are inadequate for safety.

2.3 Loads from Seismic Design Provisions

In the following subsections loads and stresses are evaluated for both a reinforced masonry building where masonry takes all the shear, and a more heavily reinforced masonry building where the reinforcement takes all the shear, in accordance with ATC-3-06 [1] and the 1979 UBC [2].

2.3.1 ATC-3-06 Tentative Provisions

The equivalent lateral force procedure given in ATC-3-06 for the seismic base shear, V, in the direction under consideration is defined by the formula

$$V = C_S W \tag{2.3}$$

where

W = the total gravity load of the building

C_S = the seismic design coefficient.

The seismic design coefficient is determined in accordance with the following formula:

$$C_S = \frac{1.2\ A_v S}{R\ T^{2/3}} \leq \frac{2.5\ A_a}{R} \tag{2.4}$$

where

A_v = a coefficient representing the effective peak velocity-related acceleration

A_a = a seismic coefficient representing the effective peak acceleration

S = a coefficient for the soil profile characteristics of the site, and S equals 1.2 for unknown soil properties

R = a response modification factor

T = the fundamental period of the building.

The fundamental period of the building is either determined by a dynamic analysis or by the formula:

$$T = \frac{0.05\ h_n}{\sqrt{L}} \tag{2.5}$$

where

h_n = height (in feet) above the base to the highest level of the building

L = overall length (in feet) of the building at the base in the direction under consideration.

The seismic base shear, as determined by Eq. 2.3, is distributed vertically in accordance with the following formula:

$$F_x = C_{vx}\ V \tag{2.6}$$

where

$$C_{vx} = \frac{W_x\ h_x^k}{\sum_{i=1}^{n} W_i\ h_i^k} \tag{2.6a}$$

$$k = \begin{cases} 1.0 & T \leq 0.5 \\ \frac{1}{2}(T + 1.5) & 0.5 < T < 2.5 \\ 2.0 & T \geq 2.5 \end{cases}$$

W_i, W_x = the portion of W located at or assigned to level i, x

h_i, h_x = the height above the base to level i, x.

2.3.2 1979 Uniform Building Code

The seismic base shear determined by the 1979 UBC and acting in the direction under consideration shall be determined in accordance with the following formula:

$$V = Z\ I\ K\ C\ S\ W \tag{2.7}$$

where

Z = numerical coefficient dependent on the seismic zone

I = Occupancy Importance Factor

K = horizontal force factor (Table 23-I in 1979 UBC)

C = numerical coefficient determined in accordance with Eq. 2.9

S = numerical coefficient for site structure resonance in accordance with Eq. 2.8.

For Eq. 2.7 to be comparable with the corresponding Eq. 2.3 of ATC-3-06, Eq. 2.7 is written as follows:

$$V = C_s' W \tag{2.7a}$$

where

$$C_s' = Z\ I\ K\ C\ S. \tag{2.7b}$$

The value of S shall be determined by the following formulas, but shall be not less than 1.0:

$$S = 1.0 + \frac{T}{T_s} - 0.5\left(\frac{T}{T_s}\right)^2, \qquad \frac{T}{T_s} \leq 1.0, \tag{2.8a}$$

$$S = 1.2 + 0.6\ \frac{T}{T_s} - 0.3\left(\frac{T}{T_s}\right)^2, \qquad \frac{T}{T_s} > 1.0, \tag{2.8b}$$

where

T shall be established by a properly substantiated analysis, but shall be not less than 0.3 sec.

T_s shall be established in accordance with UBC standard No. 23.1, except that the following shall hold:

$$0.5 \leq T_s \leq 2.5 \text{ sec}$$

and T_s shall be as near to T as possible within the range of site periods.

Where T_s is not properly established, the value of S shall be 1.5.

The value of C shall be determined in accordance with the following formula:

$$C = \frac{1}{15\sqrt{T}} \leq 0.12 \; . \tag{2.9}$$

Furthermore, the product CS need not exceed 0.14. This will be the limiting factor when S = 1.5 represents unknown soil properties. For an Importance Factor equal to 1, and a K factor of 1.33 for load-bearing shear wall type buildings, C_s' reduces to

$$C_s' = \frac{1.33\ Z\ S}{15\sqrt{T}} \leq 0.14 \cdot 1.33\ Z \; . \tag{2.10}$$

The fundamental period, T, of the building can be determined either by dynamic analysis or by the formula

$$T = \frac{0.05\ h_n}{\sqrt{D}} \tag{2.11}$$

where

D = dimension of the structure (in feet) in a direction parallel to the applied forces.

Equation 2.11 is the same as Eq. 2.5 for D = L.

The seismic base shear force determined from Eq. 2.7 is distributed vertically in accordance with the formula

$$F_x = C_{vx} (V - F_t) \tag{2.12}$$

where

$$C_{vx} = \frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i} \tag{2.13}$$

W_i, W_x = the portion of W located at or assigned to level i, x

h_i, h_x = the height above the base to level i, x

and

$$F_t = \begin{cases} 0 & T \leq 0.7 \text{ S} \\ 0.07 \text{ T V} \leq 0.25 \text{ V} & T > 0.7 \text{ S}. \end{cases} \tag{2.14}$$

F_t is the portion of V considered concentrated at the top of the structure in addition to F_n.

2.4 Allowable Stresses for Seismic Design Provisions

Both seismic design provisions tabulate allowable stresses for masonry. Both sets of provisions adjust the allowable stresses when seismic design is performed to account for both the cyclic nature of seismic loads and the past performance of masonry buildings in earthquakes.

2.4.1 ATC-3-06 Tentative Provisions

The ATC-3-06 Tentative Provisions require "the strength of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads to be determined using a capacity reduction factor, ϕ, and 2.5 times the allowable working stresses of Chapter 12A. The value of ϕ shall be as follows:

When considering shear carried by shear reinforcement and bolts . . . . ϕ = 0.6

When considering shear carried by the masonry . ϕ = 0.4."

From the stress tables of Chapter 12A of ATC-3-06 and the use of the 2.5 multiplier and strength reduction factor, the allowable stresses for seismic loads are those shown in Table 2.1.

TABLE 2.1

ATC-3-06 ALLOWABLE STRESSES FOR SEISMIC LOADS

	$\frac{M}{Vd} \geq 1$	$\frac{M}{Vd} = 0$
Unreinforced Masonry:		
Grouted	25	25
Hollow Unit	12	12
Reinforced Masonry:		
a) Masonry takes all the shear	$0.9 \sqrt{f'_m} < 40$	$2.0 \sqrt{f'_m} \leq 50$
b) Reinforcement takes all the shear	$2.25 \sqrt{f'_m} \leq 112.5$	$3.0 \sqrt{f'_m} \leq 180$

All values are in PSI and special inspection is required.

For values of M/Vd between 0 and 1 a straight line interpolation should be used.

2.4.2 1979 Uniform Building Code

The 1979 UBC requires forces for masonry shear walls to be increased by 50% (footnote in Table 24-H) if they are seismic. In addition, the allowable stresses for seismic loads are permitted to increase by one-third over the maximum allowable working stresses of Table 24-H. When these two factors are considered, the following Table is obtained for effective allowable shear stresses for masonry shear walls when considering seismic loads. The values given in Table 2.2 are obtained by multiplying the allowable stresses of Table 24-H of the UBC by 1.33/1.5.

TABLE 2.2

1979 UBC ALLOWABLE STRESSES FOR SEISMIC LOADS

	$\frac{M}{Vd} \geq 1$	$\frac{M}{Vd} = 0$
Unreinforced and Partially Reinforced:		
Grouted	25	25
Hollow Unit	12	12
Reinforced Masonry:		
a) Masonry takes all the shear	$0.8 \sqrt{f'_m} \leq 30.2$	$1.78 \sqrt{f'_m} \leq 44.4$
b) Reinforcement takes all the shear	$1.33 \sqrt{f'_m} \leq 66.5$	$1.78 \sqrt{f'_m} \leq 106.7$

All values are PSI and for inspected masonry.

For values of M/Vd between 0 and 1 a straight line interpolation should be used.

2.5 Loads Resulting from a "Realistic" Earthquake

The most difficult aspect of a study of this kind is to define in simple terms a "realistic" earthquake force. Two methods can be used to investigate the dynamic response of a structure to a strong motion earthquake. One of the methods requires the formulation of an inelastic model of the structure. The model is then subjected to a known ground motion and the inelastic dynamic response is determined. The results of a study of this type depend on how accurately the structure is represented by the inelastic model. This approach, although rather time consuming and costly, is sometimes used to check the final design of important structures. The major deficiency of this method for masonry buildings is that the properties of masonry structural elements in the inelastic range have not yet been incorporated in an inelastic computer program, but are still under investigation. The other method, which is the one used here, separates the properties of the structure from those of the earthquake. The earthquake is represented by a response spectrum which is then modified to accommodate the inelastic or ductile response of the building. The building is modeled elastically and the forces resulting from the reduced response spectrum are determined.

2.5.1 Inelastic Response Spectrum

In the development of the ATC-3-06 Tentative Provisions a distinguished group of experts including geologists, seismologists, soils engineers and structural engineers defined ground motion response spectra to represent realistic ground shaking in all regions of the United States. These spectra are normalized and combined; the resulting spectrum is shown in Fig. 2.1. ATC-3-06 states that this spectrum has an 85% - 90% probability of not being exceeded in 50 years.

To define this spectrum the ATC-3-06 provisions introduce two parameters - effective peak acceleration (EPA) and effective peak velocity (EPV). The EPA is, by definition, proportional to the spectral ordinate for periods in the range 0.1 to 0.5 sec; the EPV is proportional to the spectral ordinate at a period of about 1 sec. The constant of proportionality (for a 5% damped spectrum) is set at a standard value of 2.5 in both cases.

The following relationship exists between EPA and EPV and the coefficients A_a and A_v of Eq. 2.4.

$$\begin{aligned} EPA &= A_a \\ EPV &= 30\ S\ A_v \end{aligned} \tag{2.15}$$

where EPA is expressed as a fraction of gravity and the units of EPV are in./sec.

S = soil profile coefficient of Eq. 2.4.

In this study, for simplicity and design purposes, the flat portion of the spectrum of Fig. 2.1 is extended in the low period range. The resulting normalized design ground motion response spectrum is shown in Fig. 2.2. The elastic design response spectrum for 5% damping is then defined to be

$$\begin{aligned} S_a^{max} &= 2.5\ EPA = 2.5\ A_a \\ S_v^{max} &= 2.5\ EPV = 75\ S\ A_v . \end{aligned} \tag{2.16}$$

If the soil profile is unknown, let

$$S = 1.2,$$

then

$$S_v^{max} = 90\ A_v .$$

When the standard approximate relationship between velocity and acceleration, namely

$$S_a = \omega S_v = \frac{2\pi S_v}{T}, \tag{2.17}$$

is adopted,the elastic design response spectrum

$$C^{eq} = \frac{S_a}{g} \leq \frac{S_a^{max}}{g} \tag{2.18}$$

can be expressed as

$$C^{eq} = \frac{2\pi S_v^{max}}{gT} \leq \frac{S_a^{max}}{g}, \tag{2.19}$$

where

C^{eq} = acceleration spectral ordinate

g = acceleration of gravity

T = period of vibration.

To construct inelastic acceleration response spectra from the elastic response spectrum we adopt the method of Newmark and Hall([3], [4],[5]) which is explained as follows:

For small excursions into the inelastic range, when the resistance of the structure is idealized as an elasto-plastic function, the total displacement of the structure is assumed to remain unchanged, but is divided by the ductility factor, μ, to obtain yield displacement or acceleration. This is assumed to be valid for periods of vibration greater than about 0.5 sec. For stiffer structures with a lower period of vibration and also an elasto-plastic resistance function, a new level of acceleration is reached by equating the energy absorption of the elasto-plastic system to the energy absorption of the elastic system. Hence, the accelerations are divided by a factor of $\sqrt{2\mu-1}$. This is explained in Fig. 2.3 which is taken from reference 6.

The resulting expression is then valid for the inelastic response spectra for all ductilities:

$$C_\mu^{eq} = \frac{2\pi\, S_v^{max}}{\mu\, g\, T} \leq \frac{S_a^{max}}{\sqrt{2\mu-1}\; g} \tag{2.20}$$

Plots of C_μ^{eq}, presented in Fig. 3.20, are discussed in Section 3.1.3.

To evaluate the base shear for a single degree-of-freedom system exhibiting a ductile response, Eq. 2.20 is used as the base shear coefficient in the same manner as Eqs. 2.3 and 2.7; i.e.,

$$V = C_\mu^{eq}\, W. \tag{2.21}$$

For a multi degree-of-freedom system, some form of modal combination must be used to define the base shear force. The individual modal responses are given by

$$V_i = C_{\mu,i}^{eq}\, W_i \tag{2.22}$$

where

V_i = base shear determined for the ith mode of vibration for ductility μ,

$C_{\mu,i}^{eq}$ = corresponding spectral value, and

W_i = effective weight responding in mode i.

The most commonly accepted method of combining forces of different modes is the square root of the sum of the squares method. This is a statistical approximation and its validity for use with an inelastic response spectrum has not been established. Nevertheless, for the purposes of this study, it will be used in Chapter 4 where three individual buildings are studied, to determine the base shears from the inelastic spectrum. In Chapter 3 where the ODR is evaluated

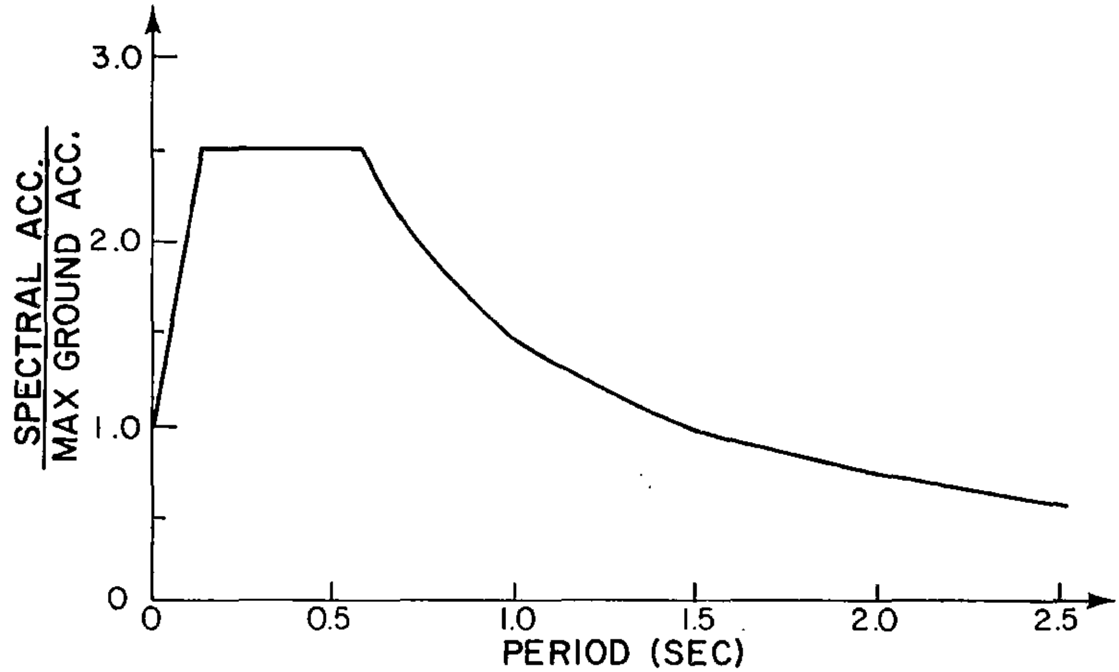


FIG. 2.1 NORMALIZED RESPONSE SPECTRUM FROM ATC-3-06

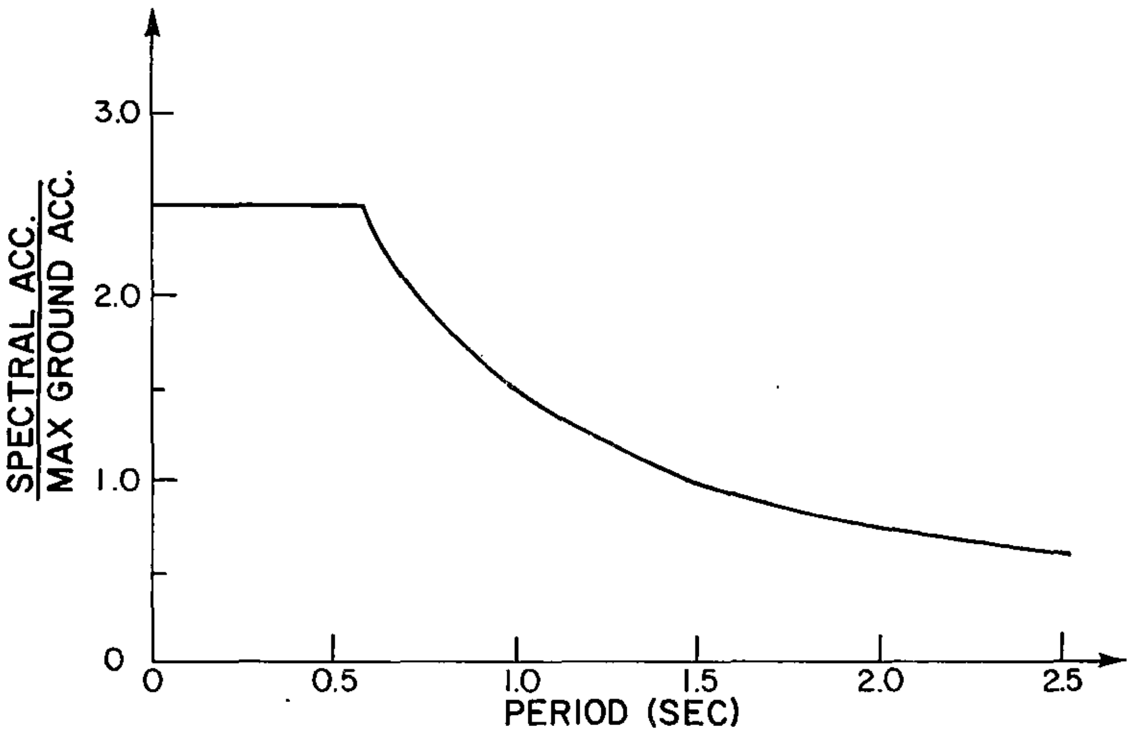


FIG. 2.2 MODIFIED NORMALIZED RESPONSE SPECTRUM

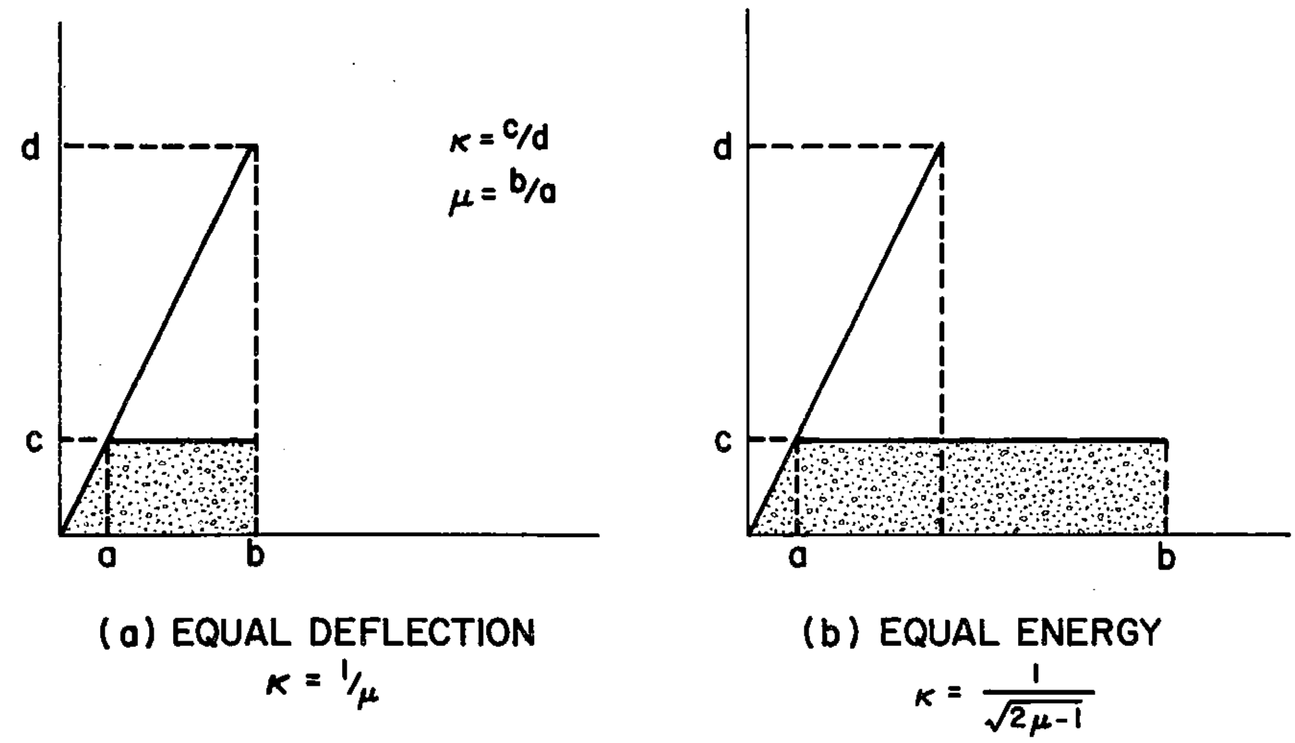


FIG. 2.3 REDUCTION FACTORS FOR SEISMIC LOADING EQUATING ELASTIC AND INELASTIC RESPONSE IN TERMS OF DEFLECTION AND ENERGY

from base shear forces, Eq. 2.22 is simplified by incorporating a modal-participation factor, α, such that only the first mode response of the building needs to be considered. Then

$$V = C_{\mu}^{eq} \alpha W \tag{2.23}$$

where

V = total base shear

α = modal-participation factor (see Appendix A)

W = total gravity load

C_{μ}^{eq} = design response spectra defined by Eq. 2.20.

The only problem now remaining is to get a reasonable estimate of the modal-participation factor, α. Using the results of the analyses of the three buildings α was determined (see Appendix A) to be

$$\alpha = 0.017/T + 0.686 \leq 1.0. \tag{2.24}$$

This is assumed to be valid for stiff structures with three or more degrees of freedom in the direction under consideration.

This means that by using a fraction α of the total weight with the first mode only, an estimate of the "realistic" earthquake forces can be made.

2.6 Summary of Test Results

This section provides a review and analysis of the test results obtained in the University of California, Berkeley test program to date. Tables 2.3 to 2.5, obtained from references 7 to 11, summarize the results of the tests and indicate that the average ultimate shear stress is affected by the height-to-width ratio of the piers, the type of masonry material, the amount of reinforcement and the type of grouting - either full or partial.

A comparison of the effect of the variables on the ultimate strengths and the inelastic behavior of the piers is shown graphically in the hysteresis envelopes presented in Figs. 2.6 through 2.17.

The first two subsections below discuss the inelastic behavior of the piers and the effects of various parameters on the ultimate strength; the third subsection discusses methods for predicting the ultimate strength. Then based on this discussion, the ultimate stresses and ductility factors used in this study are established and justified in Section 2.7.

2.6.1 Inelastic Behavior of Piers

Recently the single pier test setup (Fig. 2.4) used for the tests reported here, has been modified (Fig. 2.5). This modification was made because in these tests the value of the compressive vertical load acting on the pier increased as the in-plane horizontal displacement of the test specimen increased. This increase was due to the natural tendency of constraining steel columns to maintain a constant length, and distorted the results by changing the mode of failure of some of the piers from flexure to shear. Thus, the inelastic behavior of a pier after a major diagonal crack occurs, may be different from the behavior observed in the tests reported here. These potential distortions of the test results have been validated by preliminary tests using a modified single pier test setup that eliminated the additional compressive load on the piers. The modification consisted of replacing the steel columns by vertical actuators; these actuators impose forces of equal value but

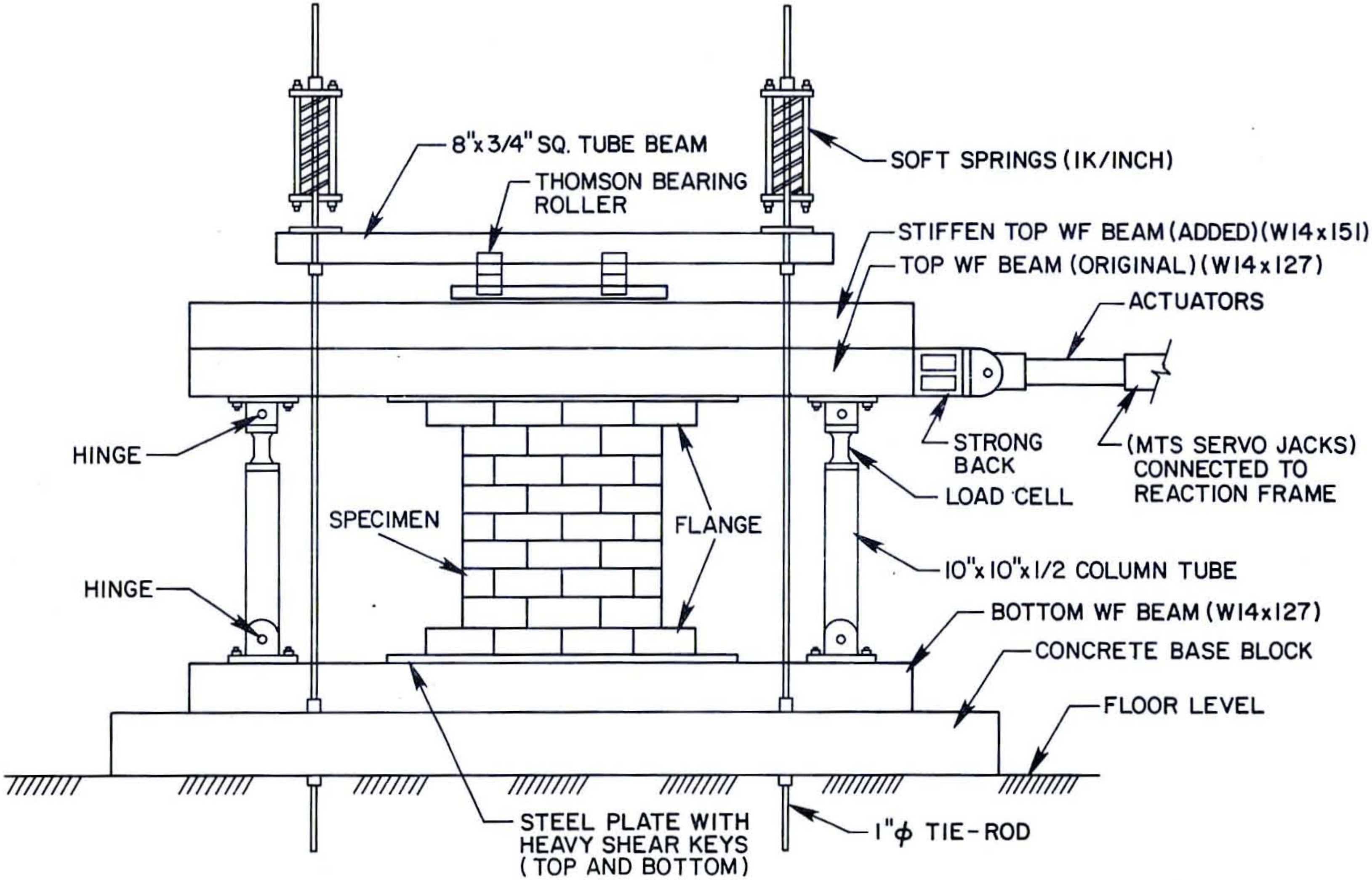


FIG. 2.4 SINGLE PIER TEST SETUP

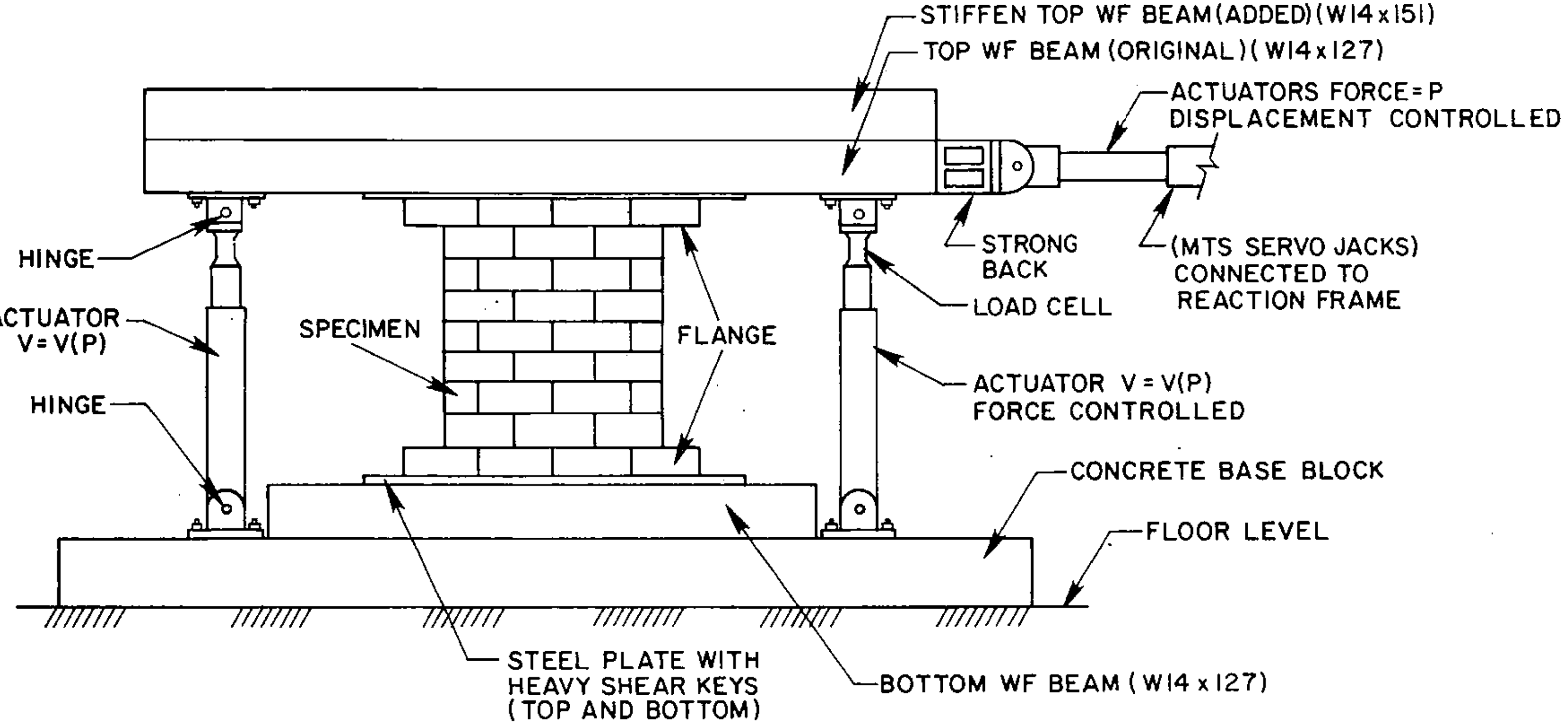


FIG. 2.5 MODIFIED SINGLE PIER TEST SETUP

TABLE 2.3

GENERAL TEST RESULTS - HCBL

(Gross cross sections: HCBL-21 = 180 in^2, HCBL-11 = 366 in^2, HCBL-12 = 610 in^2. Net cross section HCBL-11 = 220 in^2)

SPECIMEN	TEST FRE-QUENCY	GROUTING Full (F) Partial (P)	INITIAL* BEARING STRESS	VERTICAL REINFORCEMENT			HORIZONTAL REINFORCEMENT				RATIO OF TOTAL AREA OF STEEL TO GROSS AREA OF WALL	AVERAGE ULTIMATE SHEAR		PEAK ULTIMATE SHEAR		AXIAL VALUE AT ULTIMATE**	
				No. Bars	Yield Strength	$p_v = \frac{A_{vs}}{A_g}$	No. Bars	Yield Strength	$p_h = \frac{A_{hs}}{A_g}$	$A_{hs}\ fy$		FORCE	STRESS*	FORCE	STRESS*	FORCE	STRESS*
	(cps)		(psi)		(ksi)			(ksi)		(kip)	$p_v + p_h$	(kip)	(psi)	(kip)	(psi)	(kip)	(psi)
HCBL-21- 1	0.2	F	250	4#6	79.0	0.0098	-	-	-	-	0.0098	24.0	133	26.0	144	-12.0	-67
- 3	0.02	F	125	4#4	54.1	0.0044	-	-	-	-	0.0044	26.0	144	27.3	152	+12.2	+68
- 5	0.02	F	0	4#6	78.1	0.0098	-	-	-	-	0.0098	18.5	103	20.5	114	+26.1	+145
- 7	0.02	F	250	4#6	78.1	0.0098	3#5	67.8	0.0024	63.1	0.0116	39.0	217	40.7	226	+ 6.7	+37
- 9	0.02	F	500	4#6	78.5	0.0098	-	-	-	-	0.0098	28.7	159	29.5	164	-52.6	-292
-13	0.02	F	125	4#4	50.8	0.0044	3#7 2#5	62.9	0.0063	152.2	0.0105	26.0	144	29.1	162	+14.4	+80
-15	0.02	F	125	4#4	51.8	0.0044	3#7 PL 2#5	64.0	0.0063	154.9	0.0105	33.6	187	35.2	196	+22.2	+123
HCBL-11- 1	1.5	F	55	-	-	-	-	-	-	-	-	45.2	123	49.5	135	-44.0	-120
- 2	1.5	P	91(55)	-	-	-	-	-	-	-	-	25.2	115(69)	26.3	120(72)	-42.2	-192(-115)
- 3	1.5	F	55	2#5	70.8	0.0017	-	-	-	-	0.0017	46.3	127	49.1	134	-25.1	-69
- 4	1.5	F	55	2#5	70.8	0.0017	1#5	47.9	0.0007	14.8	0.0024	60.3	165	62.7	171	-39.1	-107
- 5	1.5	P	91(55)	2#5	70.8	0.0017	1#5	47.9	0.0007	14.8	0.0024	46.8	213(128)	49.6	226(136)	-30.2	-137(-83)
- 6	1.5	F	55	2#5	70.8	0.0017	4#5	47.9	0.0029	59.4	0.0046	72.8	199	82.7	226	-52.7	-144
- 7	1.5	F	55	2#8	69.2	0.0043	-	-	-	-	0.0043	53.6	146	65.8	180	-33.3	-91
- 8	1.5	P	91(55)	2#8	69.2	0.0043	-	-	-	-	0.0043	36.8	167(101)	37.9	172(104)	-29.2	-133(-80)
- 9	1.5	F	55	2#8	69.2	0.0043	2#5	47.9	0.0015	29.7	0.0058	53.6	146	56.9	155	-41.9	-114
-10	1.5	P	91(55)	2#8	69.2	0.0043	2#5	47.9	0.0015	29.7	0.0058	48.7	222(133)	50.2	228(137)	-31.2	-142(-85)
-11	1.5	F	55	2#8	69.2	0.0043	4#6	73.9	0.0041	130.1	0.0084	84.5	231	87.7	240	-50.8	-139
HCBL-12- 1	0.02	F	52	3#7	80.3	0.0030	-	-	-	-	0.0030	189.1	310	200.3	328	-118.5	-194
- 2	0.02	F	52	3#7	80.3	0.0030	1#5	69.6	0.0010	21.6	0.0040	201.5	330	211.7	347	-122.0	-200
- 3	0.02	F	52	3#7	80.3	0.0030	2#5	69.6	0.0020	43.2	0.0050	242.5	398	251.4	412	-148.5	-243
- 4	0.02	F	52	3#7	80.3	0.0030	3#5	69.6	0.0030	64.7	0.0060	209.9	344	218.6	358	-129.4	-212
- 5	0.02	F	52	3#7	80.3	0.0030	4#5	69.6	0.0040	86.3	0.0070	220.2	361	228.0	374	-130.9	-215
- 6	0.02	F	52	3#7	80.3	0.0030	4#6	67.3	0.0058	118.4	0.0088	252.0	413	261.7	429	-143.0	-234

* Partially grouted pier stresses computed using net areas. Values in parenthesis indicate gross area stresses.

** Positive values indicate tension; negative values indicate compression. For the double pier tests (HCBL-21) these values correspond to the pier where a tensile axial force is imposed by the overturning moment effect.

TABLE 2.4

GENERAL TEST RESULTS - HCBR

Gross Cross Sections: HCBR-21 = 310 in^2, HCBR-11 = 354 in^2, HCBR-12 = 575 in^2, Net Cross Sections: HCBR-21 = 171 in^2
CBRC-21 = 420 in^2, CBRC-11 = 480 in^2, CBRC-12 = 780 in^2 HCBR-11 = 189 in^2

Specimen	Grouting Full (F) Partial (P) Solid (S)	Vertical Reinforcement		Horizontal Reinforcement				Ratio Of Total Area Of Steel To Gross Area Of Wall	Average Ultimate Shear		Peak Ultimate Shear		Axial Compression At Ultimate		Shear Crack Strength*	Compressive Stress At Shear Crack*
		No. Bars	$P_v = \frac{A_{vs}}{A_g}$	No. Bars	Yield Strength (ksi)	$P_h = \frac{A_{hs}}{A_g}$	$A_{hs} f_y$ (kip)	$P_v + P_h$	Force (kip)	Stress* (psi)	Force (kip)	Stress* (psi)	Force (kip)	Stress* (psi)	(psi)	(psi)
HCBR-21-1	F	No	--	No	---	--	--	---	75.4	244	82.6	267	179.5	580	267	580
-2	F	2#8	0.0051	No	---	--	--	0.0051	63.7	206	73.7	238	113.9	368	238	368
-3	P	2#8	0.0051	No	---	--	--	0.0051	27.1	159	31.0	181	33.0	193	181	193
-4	F	2#8	0.0051	2#5	49.7	0.0011	30.8	0.0062	84.6	273	95.4	308	128.6	415	308	415
-5	P	2#8	0.0051	2#5	49.7	0.0011	30.8	0.0062	47.6	279	51.8	303	53.6	314	303	314
-6	F	2#8	0.0051	3#5	49.7	0.0016	46.2	0.0067	98.2	317	106.3	343	152.4	492	343	492
-7	P	2#8	0.0051	3#5	49.7	0.0016	46.2	0.0067	47.5	278	51.9	304	52.3	306	304	306
-8	F	2#8	0.0051	4#5	49.7	0.0021	61.6	0.0072	99.3	321	107.2	346	150.2	485	346	485
-9	F	2#8	0.0051	5#5	49.7	0.0026	77.0	0.0077	95.1	307	107.9	348	147.5	476	348	476
HCBR-11-1	F	No	--	No	---	--	--	---	90.1	255	98.5	278	116.1	328	278	328
-2	P	No	--	No	---	--	--	---	---	---	26.6	141	76.5	405	141	405
-3	F	2#5	0.0018	No	---	--	--	0.0018	94.4	267	98.9	279	52.3	148	279	148
-4	F	2#5	0.0018	1#5	70.0	0.0008	21.7	0.0026	119.3	337	124.8	353	114.3	323	353	323
-5	P	2#5	0.0018	1#5	70.0	0.0008	21.7	0.0026	45.4	240	52.4	278	53.7	284	278	284
-6	F	2#5	0.0018	5#5	64.2	0.0038	99.5	0.0056	116.2	328	122.4	346	61.9	175	346	175
-7	F	2#5	0.0018	5#5	72.6	0.0038	112.5	0.0056	94.6	267	99.2	280	85.3	241	280	241
-8	F	2#8	0.0045	No	---	--	--	0.0045	80.4	227	85.6	242	43.4	123	242	123
-9	P	2#8	0.0045	No	---	--	--	0.0045	43.0	228	49.1	260	37.3	198	260	198
-10	F	2#8	0.0045	2#5	68.7	0.0015	42.6	0.0060	101.6	287	104.8	296	54.2	153	296	153
-11	P	2#8	0.0045	2#5	68.7	0.0015	42.6	0.0060	46.0	244	51.9	275	26.7	141	275	141
-12	F	2#8	0.0045	5#6	73.9	0.0053	162.6	0.0098	94.3	266	97.2	275	85.0	240	275	240
-13	F	2#8	0.0045	5#6	74.7	0.0053	164.3	0.0098	113.3	320	116.3	329	110.6	312	329	312
HCBR-12-1	F	3#7	0.0031	No	---	--	--	0.0031	208.7	363	220.8	384	101.2	176	--	--
-2	F	3#7	0.0031	1#6	67.3	0.0015	29.6	0.0046	182.7	318	191.0	332	86.0	149	319	125
-3	F	3#7	0.0031	2#6	67.3	0.0030	59.2	0.0061	211.8	368	220.8	384	114.1	198	351	150
-4	F	3#7	0.0031	3#6	67.3	0.0045	88.8	0.0076	245.8	427	255.3	444	142.4	248	356	143
-5	F	3#7	0.0031	4#6	67.3	0.0060	118.4	0.0091	223.8	389	232.7	404	100.7	175	394	154
-6	F	3#7	0.0031	5#7	80.3	0.0102	240.9	0.0133	251.4	437	259.0	450	128.0	223	392	153

TABLE 2.5

GENERAL TEST RESULTS - CBRC

Gross Cross Sections: HCBR-21 = 310 in^2, HCBR-11 = 354 in^2, HCBR-12 = 575 in^2, Net Cross Sections: HCBR-21 = 171 in^2
CBRC-21 = 420 in^2, CBRC-11 = 480 in^2, CBRC-12 = 780 in^2 HCBR-11 = 189 in^2

Specimen	Grouting Full (F) Partial (P) Solid (S)	Vertical Reinforcement		Horizontal Reinforcement				Ratio Of Total Area Of Steel To Gross Area Of Wall $P_v + P_h$	Average Ultimate Shear		Peak Ultimate Shear		Axial Compression At Ultimate		Shear Crack Strength*	Compressive Stress At Shear Crack*
		No. Bars	$P_v = \frac{A_{vs}}{A_g}$	No. Bars	Yield Strength (ksi)	$P_h = \frac{A_{hs}}{A_g}$	$A_{hs}f_y$ (kip)		Force (kip)	Stress* (psi)	Force (kip)	Stress* (psi)	Force (kip)	Stress* (psi)	(psi)	(psi)
CBRC-21-1	S	No	--	No	---	--	--	---	92.7	221	100.3	239	221.7	528	--	--
-2	S	2#8	0.0038	No	---	--	--	0.0038	114.2	272	123.8	295	200.5	477	295	477
-3	S	2#8	0.0038	2#5	49.7	0.0008	30.8	0.0046	106.0	252	110.8	264	192.5	458	264	458
-4	S	2#8	0.0038	3#5	49.7	0.0012	46.2	0.0050	104.2	248	112.3	267	175.2	417	267	417
-5	S	2#8	0.0038	5#5	49.7	0.0019	77.0	0.0057	105.0	250	110.0	262	158.5	377	262	377
CBRC-11-1	S	No	--	No	---	--	--	---	114.9	239	118.6	247	141.9	296	247	296
-2	S	2#5	0.0013	No	---	--	--	0.0013	106.0	221	117.0	244	92.7	193	244	193
-3	S	2#5	0.0013	1#5	68.3	0.0006	21.2	0.0019	106.7	222	114.5	239	89.5	186	239	186
-4	S	2#5	0.0013	5#5	68.3	0.0028	105.9	0.0041	124.4	259	128.6	268	132.5	276	268	276
-5	S	2#8	0.0033	No	---	--	--	0.0033	102.0	213	104.3	217	76.4	159	217	159
-6	S	2#8	0.0033	2#5	73.9	0.0011	45.8	0.0044	128.3	267	130.4	272	100.3	209	272	209
-7	S	2#8	0.0033	5#6	74.7	0.0039	164.3	0.0072	115.7	241	123.3	257	80.9	169	257	169
CBRC-12-1	S	3#7	0.0023	No	---	--	--	0.0023	190.4	244	197.2	253	83.9	108	253	108
-2	S	3#7	0.0023	1#6	67.3	0.0011	29.6	0.0034	186.3	239	194.8	250	98.9	127	250	127
-3	S	3#7	0.0023	2#6	67.3	0.0022	59.2	0.0045	207.9	267	217.3	279	117.1	150	275	138
-4	S	3#7	0.0023	3#6	67.3	0.0033	88.8	0.0056	227.1	291	235.0	301	96.1	123	--	--
-5	S	3#7	0.0023	4#6	67.3	0.0044	118.4	0.0067	183.0	235	192.3	247	109.8	141	231	116
-6	S	3#7	0.0023	5#7	80.3	0.0075	240.9	0.0098	207.3	266	216.1	277	110.7	142	240	94

* Partially grouted pier stresses computed using net areas.

opposite sign at two sides of the pier and the magnitude of the forces is selected to maintain the point of inflection of the deformed shape at the mid-height of the pier. The modified single pier test setup permits the test to be developed under any desired constant bearing load, and a series of tests is presently underway to ratify or modify the previous results concerning the inelastic behavior of the piers after major diagonal cracks have occurred. Consequently, a detailed discussion of the characteristics of the shear mode of failure will not be presented here but will await ratification or modification of the results obtained to date.

2.6.1.1 Flexural Mode of Failure

The inelastic characteristics obtained for the four double piers displaying a flexural mode of failure, are quite desirable (see Fig. 2.6) in that they are similar to those displayed by elastic-plastic materials. Furthermore, the use of plates in mortar joints in Tests HCBL-21-15 and 16 [7] significantly improved the inelastic performance of the piers. These results are similar to those obtained by Priestley ([12],[13]) in his extensive cantilever pier tests on the flexural mode of failure, from which he reports ductility factors ranging from 4 to 8. Thus, it is clear that if a pier can be designed to fail in flexure then desirable inelastic performance can be anticipated.

The vertical compressive load is an important parameter in determining the inelastic performance of the piers since it can change the mode of failure of the piers and thereby significantly affect the inelastic characteristics. The effect of an increasing compressive load can be removed from the test results for a single pier as discussed in reference 9. The flexural hysteresis envelope thus derived from the experimentally determined envelope for the pier failing in shear (i.e., assuming the compressive load is constant) is similar to the hysteresis envelope for the double pier; it is also very desirable as shown in Fig. 2.17 for Tests HCBR-21-8 and 9.

2.6.1.2 Shear Mode of Failure

i) Effect of partial grouting: From the hysteresis envelopes presented in Figs. 2.13 and 2.15, it is clear that there is no significant difference in the inelastic characteristics of partially and fully grouted hollow concrete block piers. However, for hollow clay brick piers there is a significant difference. As seen in Figs. 2.14 and 2.16 the inelastic behavior of the partially grouted HCBR piers is significantly less desirable than that of the fully grouted piers; the deformation capability of the partially grouted piers is less, the strength degradation is much sharper, and the ultimate strength based on net area stresses is always smaller than that of the corresponding fully grouted piers.

ii) Effect of horizontal reinforcement: In general, the test results of Figs. 2.7 through 2.12 show that horizontal reinforcement increases the ductility of the pier and hence the energy that the pier is able to absorb. An increase in the amount of horizontal reinforcement improves the crack pattern and increases the pier's deformation capacity. However, there is not a linear relationship between the amount of reinforcement and the amount of improvement obtained. Furthermore, the horizontal reinforcement does not appear to influence the rate of strength degradation of the pier after the ultimate strength has been attained, although this will be studied more extensively with the new test setup. This favorable influence of the reinforcement on the pier behavior holds for the HCBL and HCBR piers, but is quite minimal for the double wythe, grouted core, clay brick (CBRC) piers.

2.6.2 Effect of Various Parameters on the Ultimate Strength

2.6.2.1 Effect of Type of Material and Height-to-Width Ratio

The three different types of material had different effects on the average ultimate stresses associated with the shear mode of failure. The trends in the results for the hollow concrete block (HCBL) and hollow clay brick piers (HCBR) were similar and, in general, the hollow clay brick piers had higher ultimate stresses than the corresponding hollow concrete block piers, except for piers with a height-to-width ratio of 0.5. The values for the grouted core clay brick piers (CBRC) were different from both the HCBR and HCBL piers in that they did not increase as the height-to-width ratio decreased. Whereas for both the HCBL and HCBR piers there was an increase in the ultimate stress as the height-to-width ratio decreased.

For the HCBL piers the range of the average ultimate shear stress was 106 - 212 psi for piers with a height-to-width ratio of 2, 123 - 231 psi for a height-to-width ratio of 1 and 310 - 413 psi for a height-to-width ratio of 0.5. The corresponding ranges for the ratios of average ultimate shear stress to $\sqrt{f'_m}$ were 2.1 - 4; 3.0 - 6.3 and 5.2 - 7.6, respectively.

For the HCBR piers the range of the average ultimate shear stress was 206 - 321 psi for a height-to-width ratio of 2, 225 - 337 psi for a height-to-width ratio of 1 and 318 - 437 psi for a height-to-width ratio of 0.5. The corresponding ranges of the ratios of average ultimate shear stress to $\sqrt{f'_m}$ were 3.1 - 4.8, 4.2 - 6.5 and 6.0 - 8.2, respectively.

For the CBRC piers the range of the average ultimate shear stress was similar for piers of all three height-to-width ratios and was 213 - 272 psi. The corresponding range for the ratio of average ultimate shear stress to $\sqrt{f'_m}$ was 4.3 - 5.3.

The above values are listed in Tables 2.6 through 2.9 and illustrated graphically in Fig. 2.18.

2.6.2.2 Effect of Horizontal Reinforcement

For the hollow concrete block piers the effect of varying the amount of horizontal reinforcement was included in only one set of the 2 to 1 double piers. The 0.25% horizontal reinforcement (area of steel to gross vertical area of pier) increased the ultimate strength by approximately 40%. For the HCBL-11 piers, horizontal reinforcement increased the ultimate shear stress only when significant amounts (0.34% and 0.48%) were added to the piers. The increase in strength was of the order of 30%. For the HCBL-12 piers, the effect of increasing the amount of horizontal reinforcement was less clear because of the variations in the results. However, there was a trend of increasing strength with increasing amounts of reinforcement (see Fig. 2.19a).

For the hollow clay brick piers increasing the amount of horizontal reinforcement appeared to increase the ultimate stress of the HCBR-21 piers and, to a lesser extent, that of the HCBR-12 piers, although it had little effect on the HCBR-11 piers (see Fig. 2.19b).

For the grouted core clay brick piers increasing the amount of horizontal reinforcement had little or no effect on the ultimate strength for all three height-to-width ratios (see Fig. 2.19c).

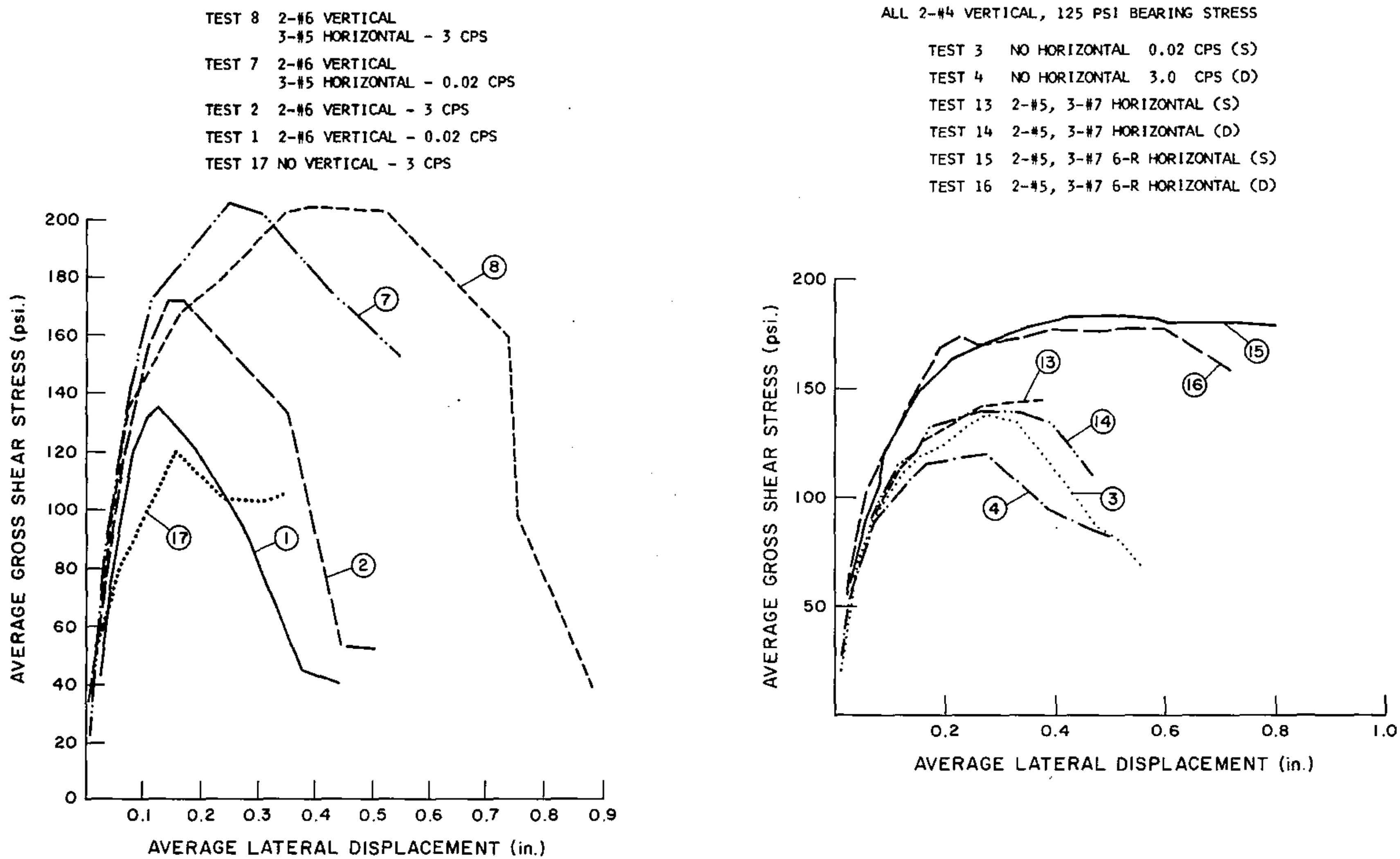


FIG. 2.6 EFFECT OF HORIZONTAL REINFORCEMENT ON HYSTERESIS ENVELOPE (HCBL-21)

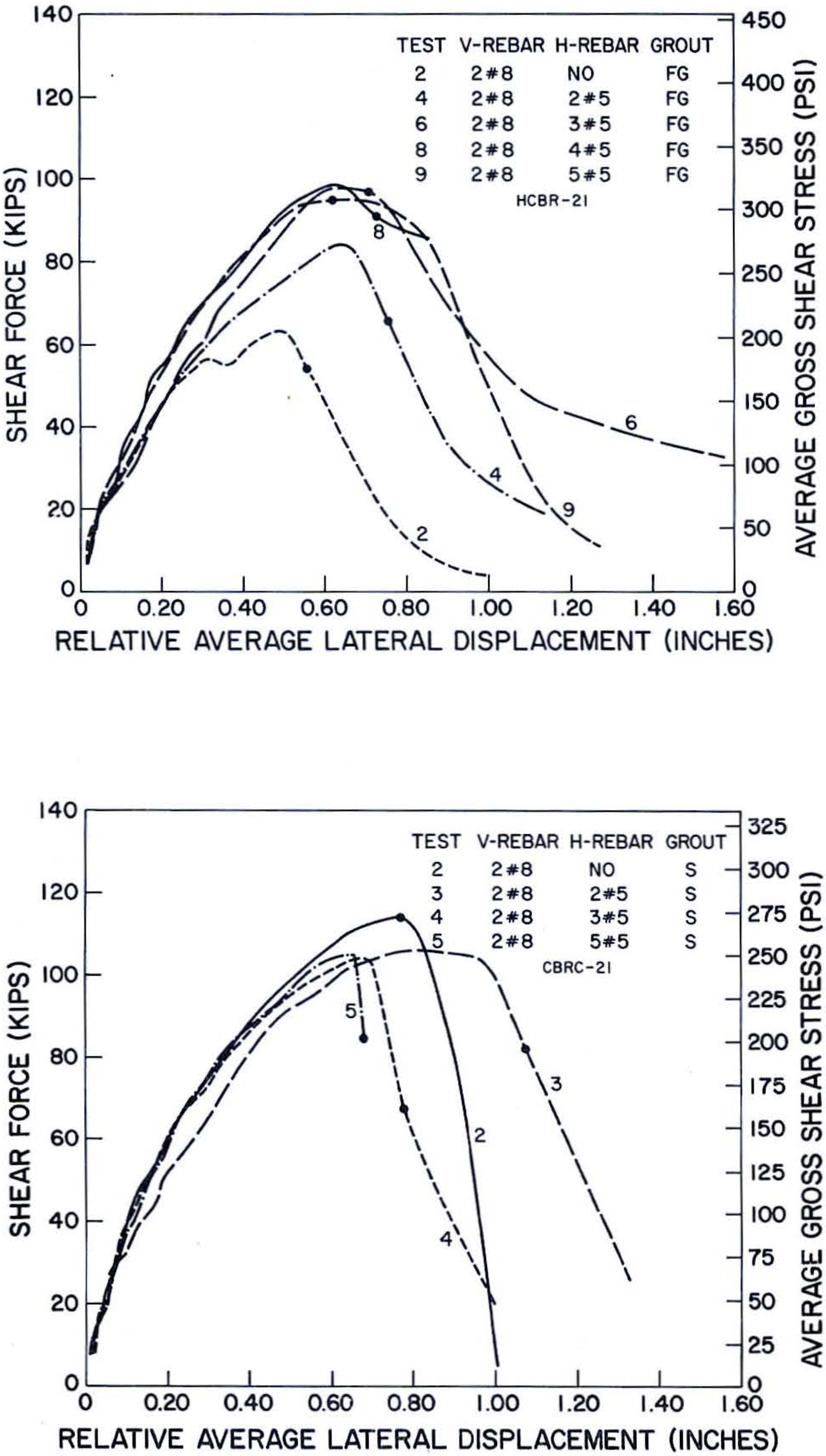


FIG. 2.7 EFFECT OF HORIZONTAL REINFORCEMENT ON HYSTERESIS ENVELOPE (HCBR/CBRC-21)

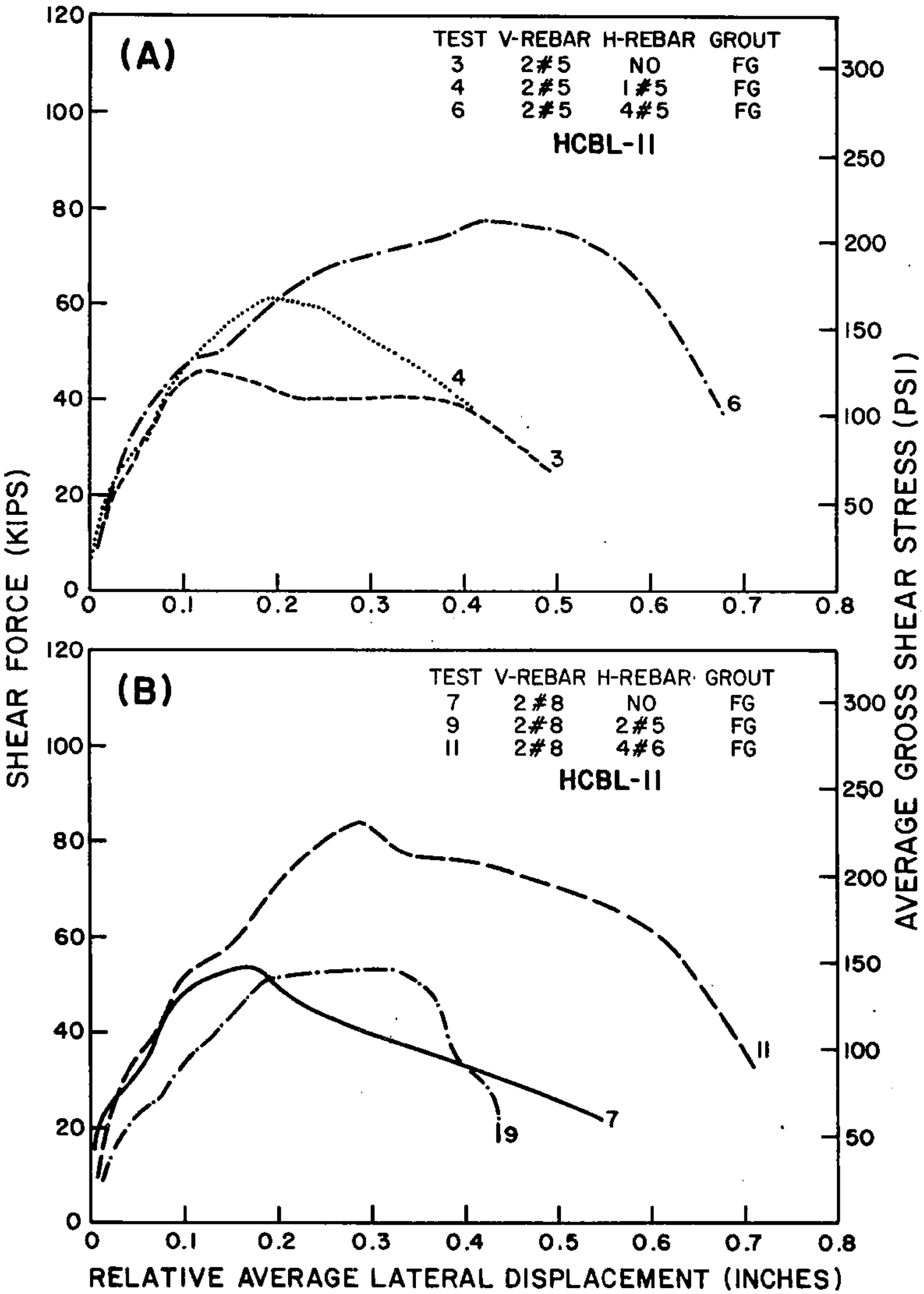


FIG. 2.8 EFFECT OF HORIZONTAL REINFORCEMENT ON HYSTERESIS ENVELOPE (HCBL-11)

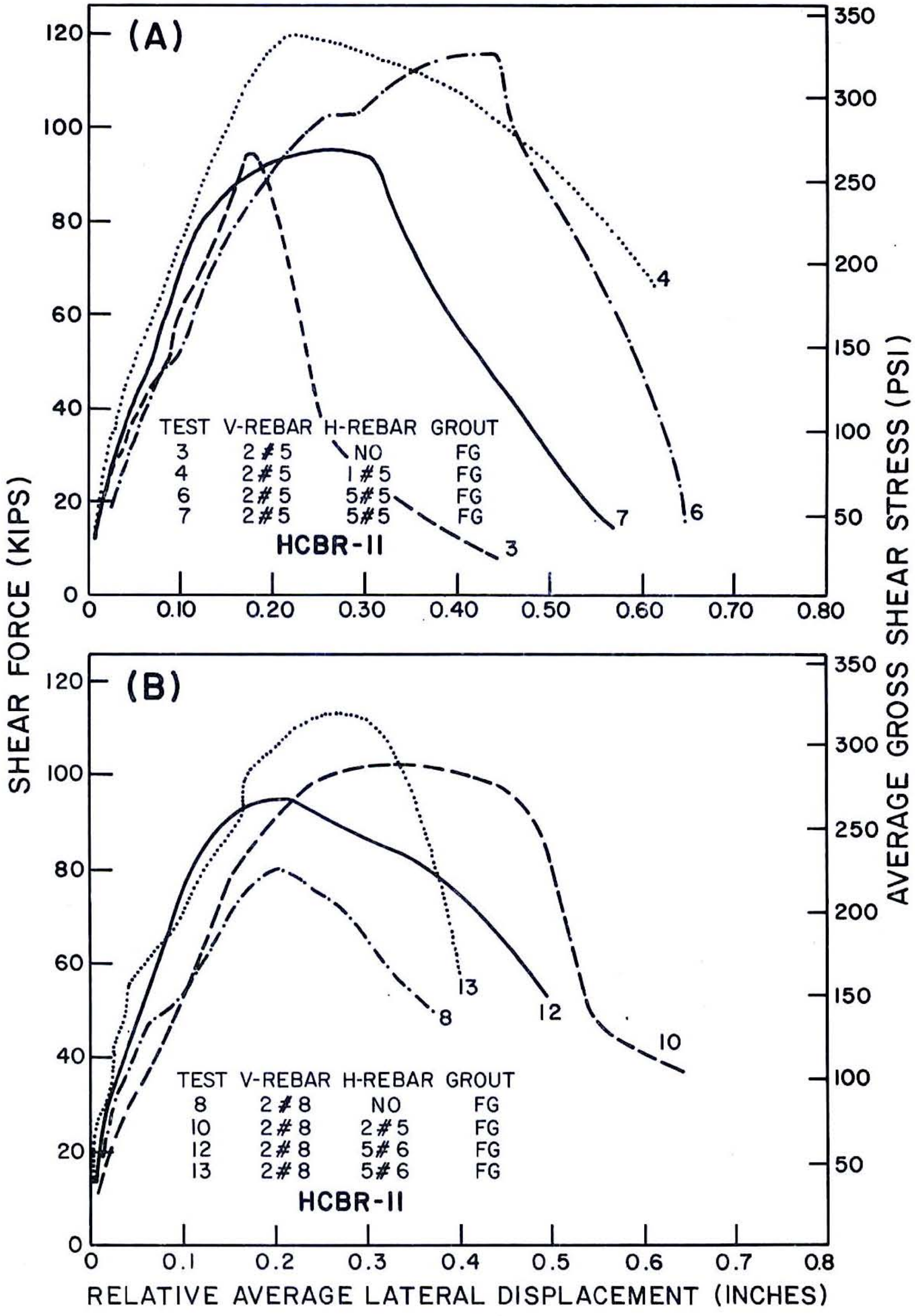


FIG. 2.9 EFFECT OF HORIZONTAL REINFORCEMENT ON HYSTERESIS ENVELOPE (HCBR-11)

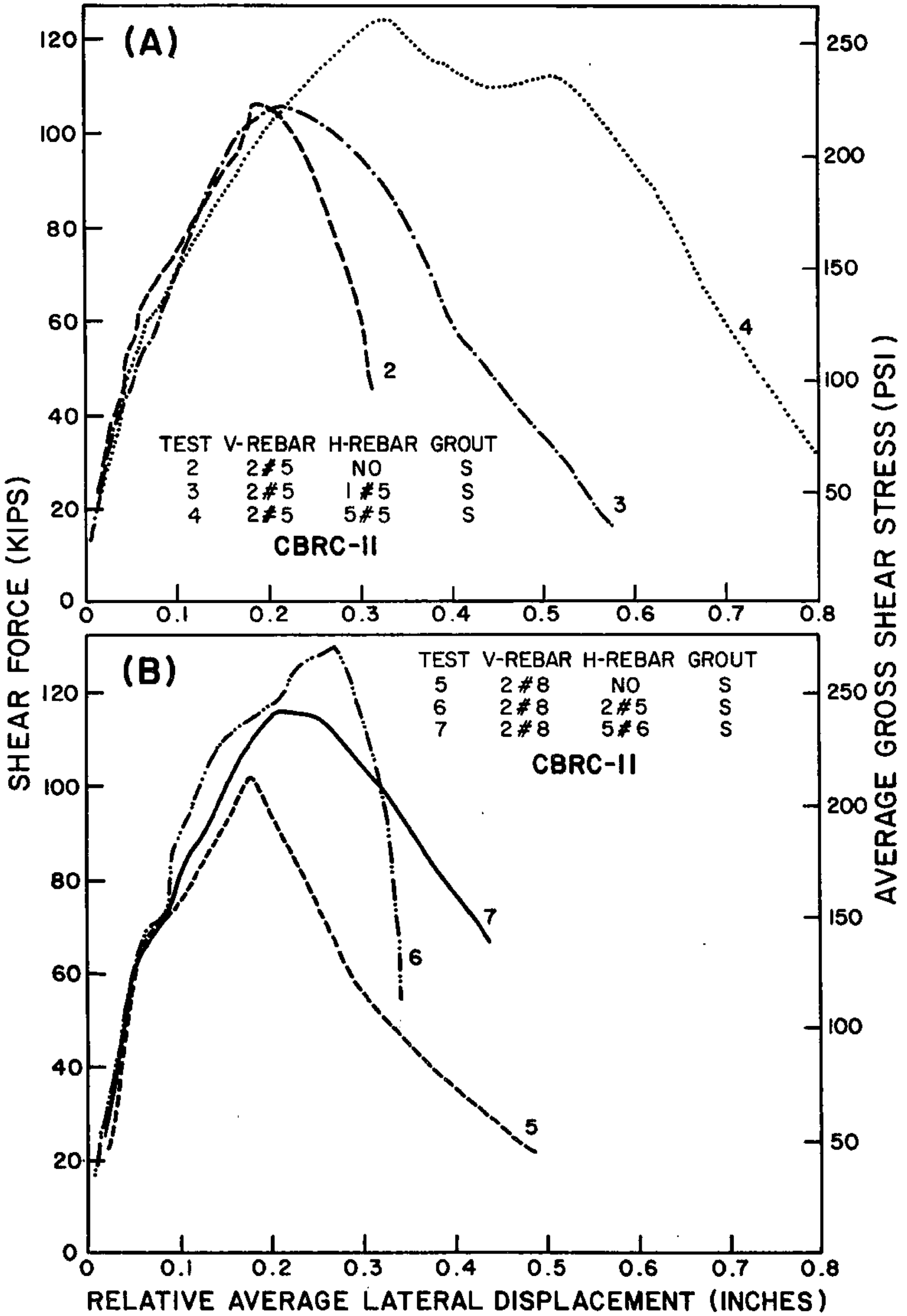


FIG. 2.10 EFFECT OF HORIZONTAL REINFORCEMENT ON HYSTERESIS ENVELOPE (CBRC-11)

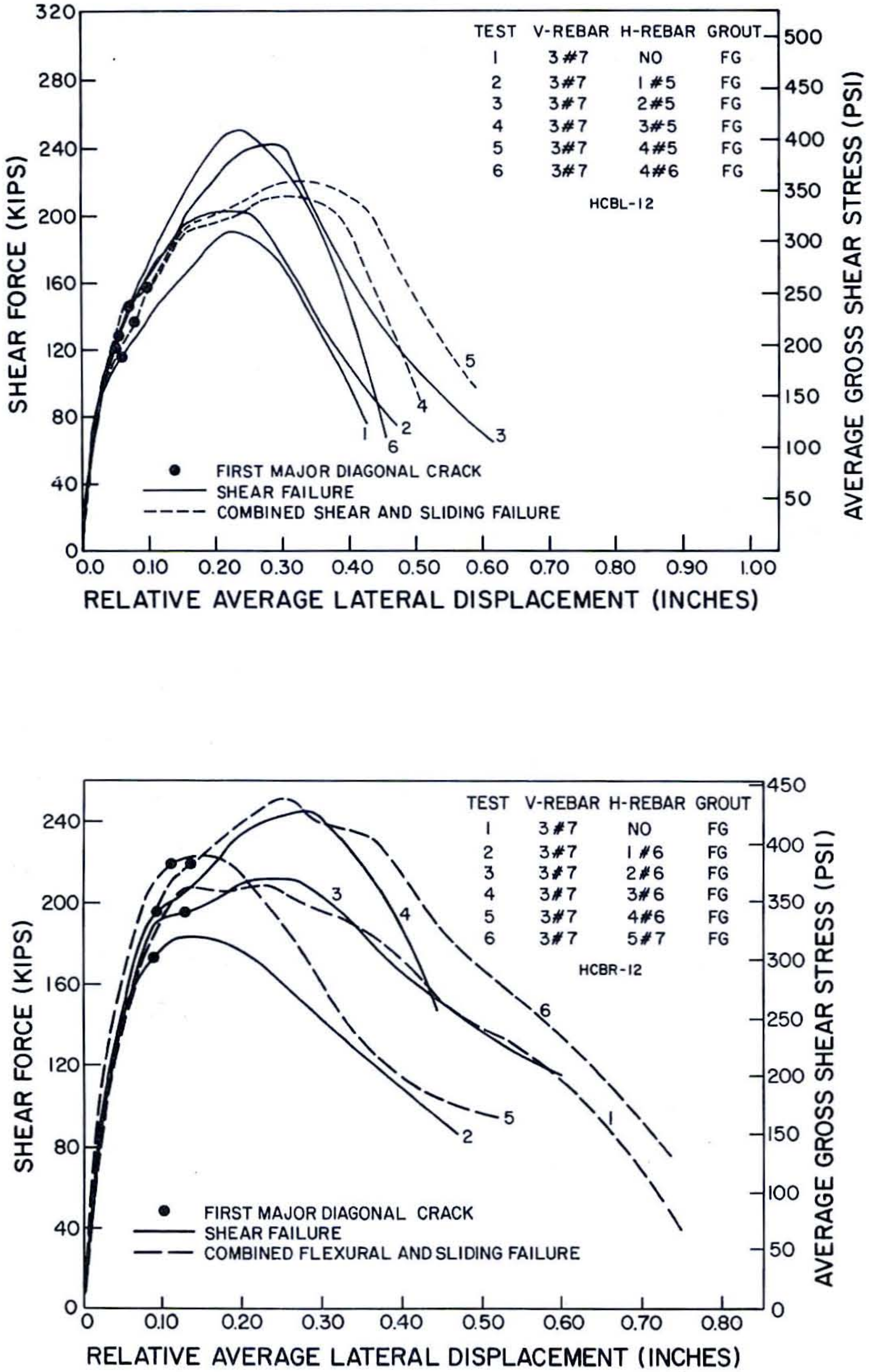


FIG. 2.11 EFFECT OF HORIZONTAL REINFORCEMENT ON HYSTERESIS ENVELOPE (HCBL/HCBR-12)

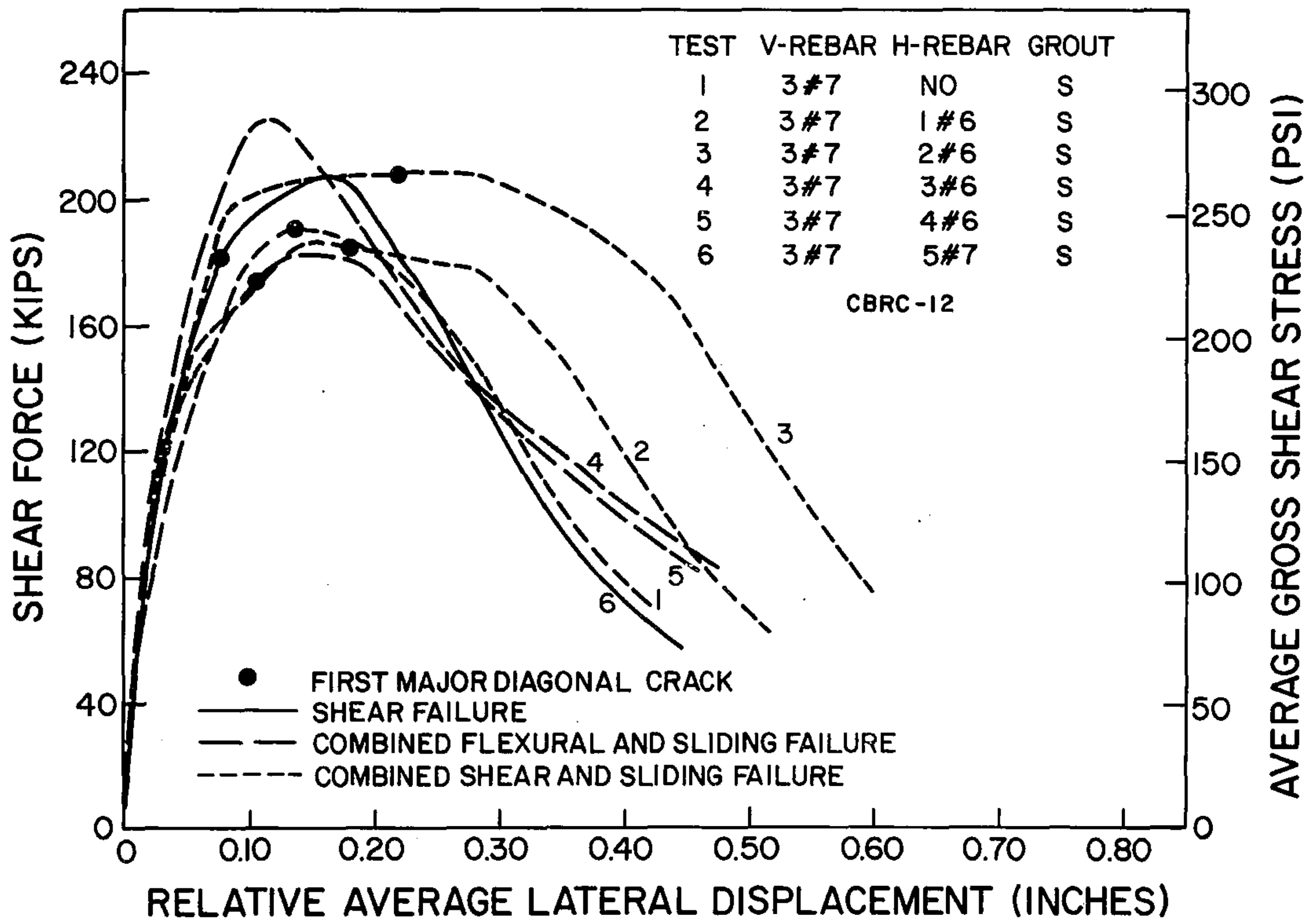


FIG. 2.12 EFFECT OF HORIZONTAL REINFORCEMENT ON HYSTERESIS ENVELOPE (CBRC-12)

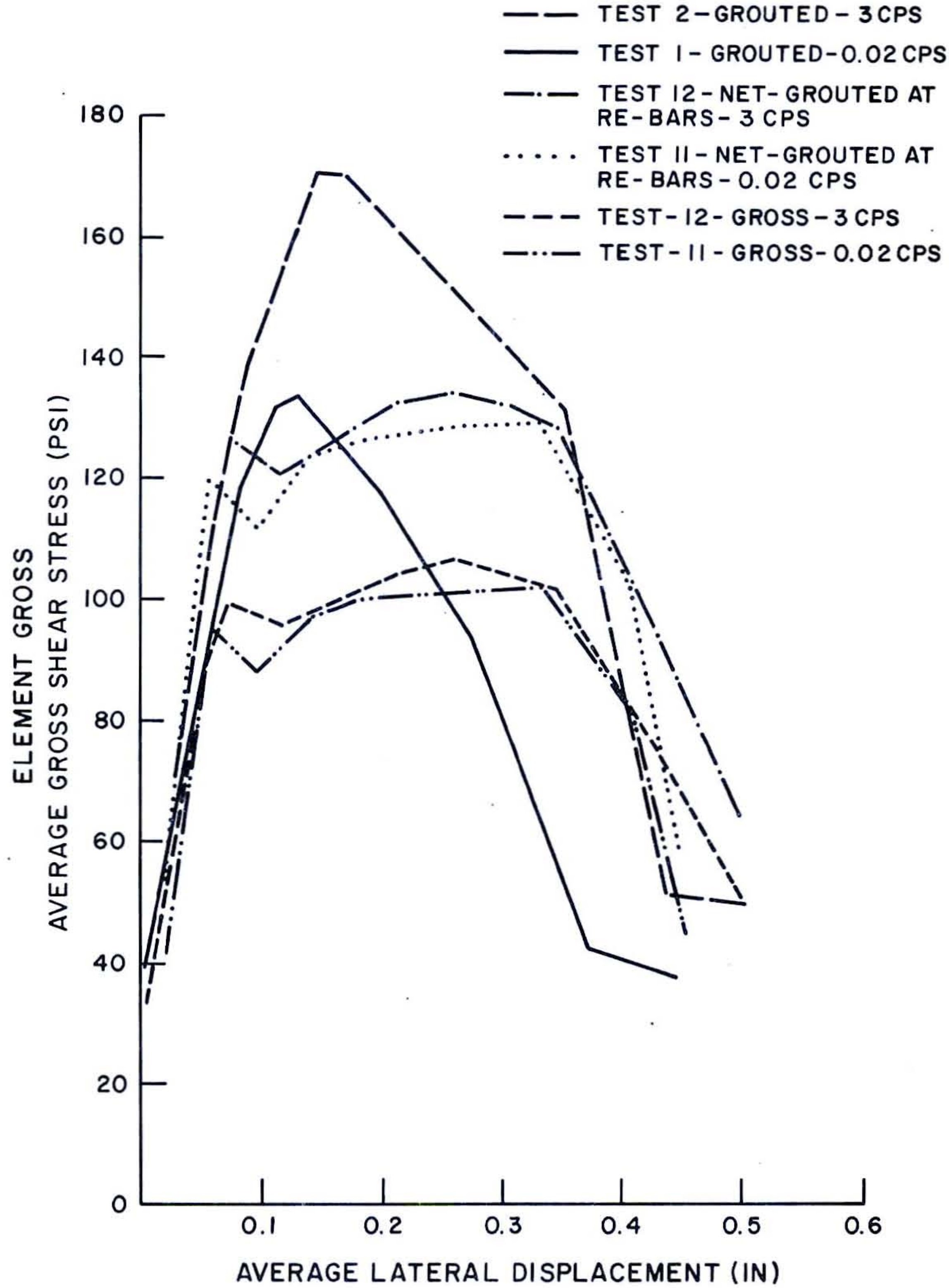
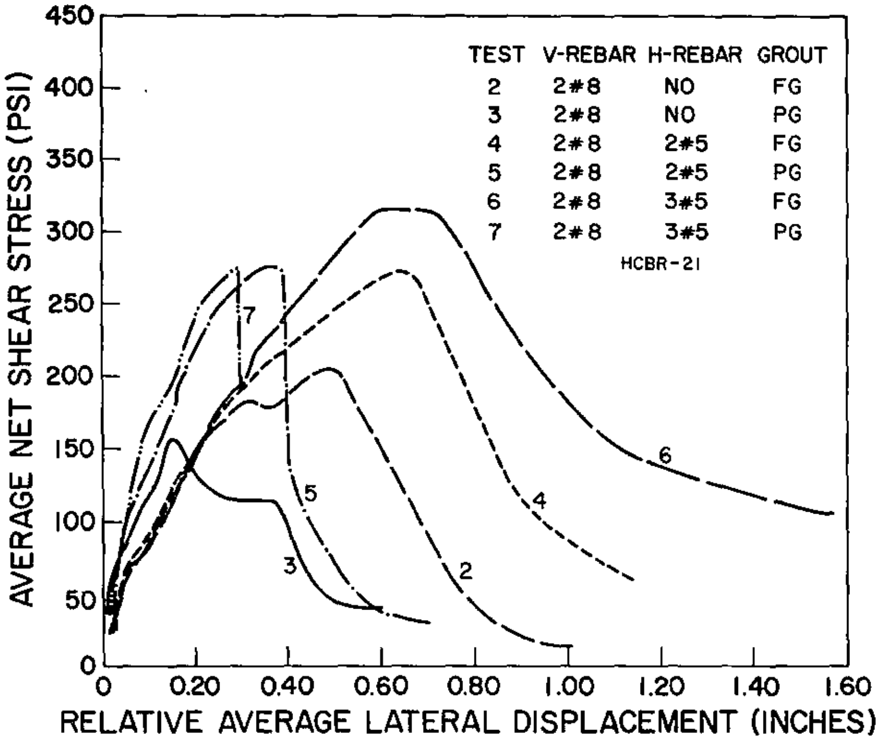


FIG. 2.13 EFFECT OF PARTIAL GROUTING ON HYSTERESIS ENVELOPE (HCBL-21)



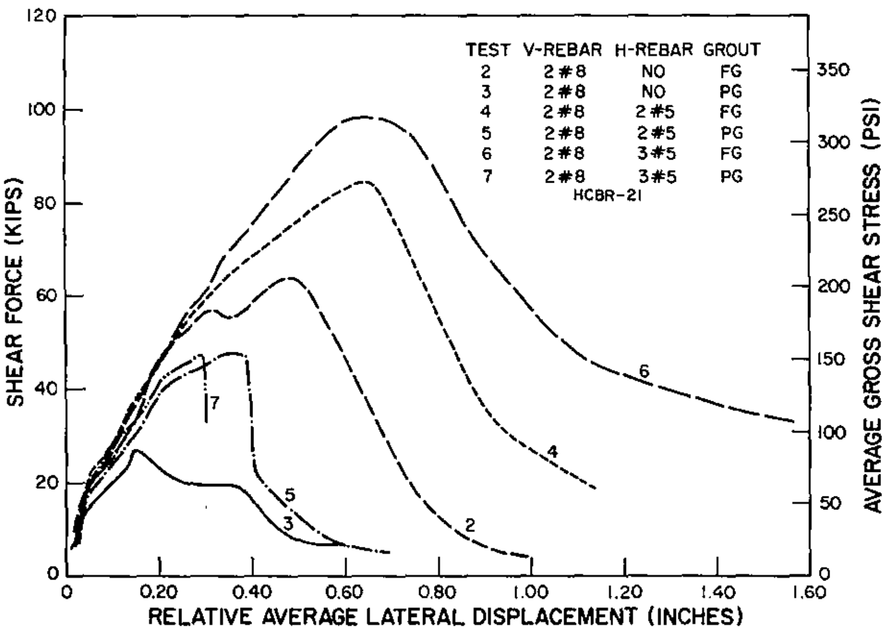


FIG. 2.14 EFFECT OF PARTIAL GROUTING ON HYSTERESIS ENVELOPE (HCBR-21)

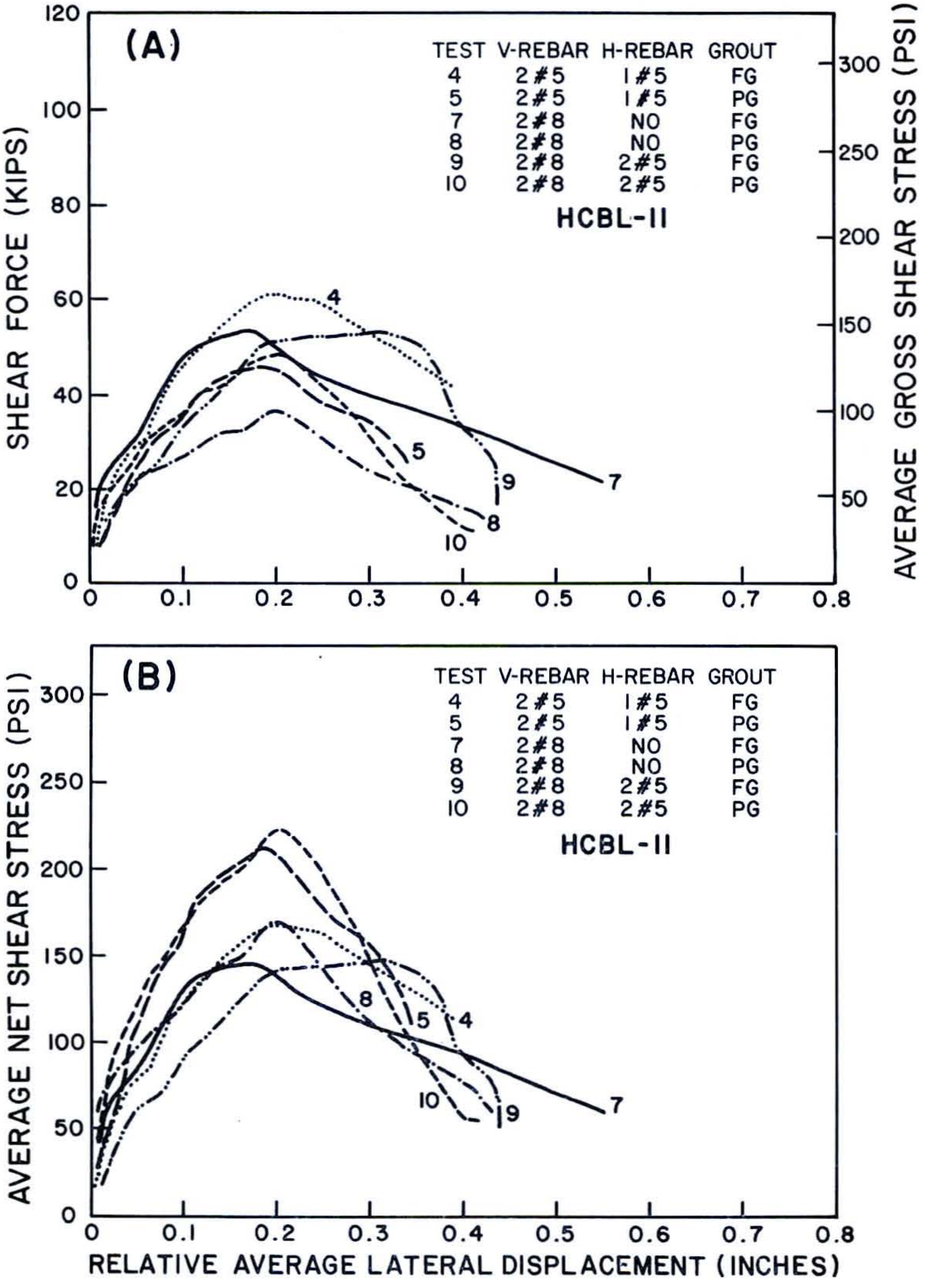


FIG. 2.15 EFFECT OF PARTIAL GROUTING ON HYSTERESIS ENVELOPE (HCBL-11)

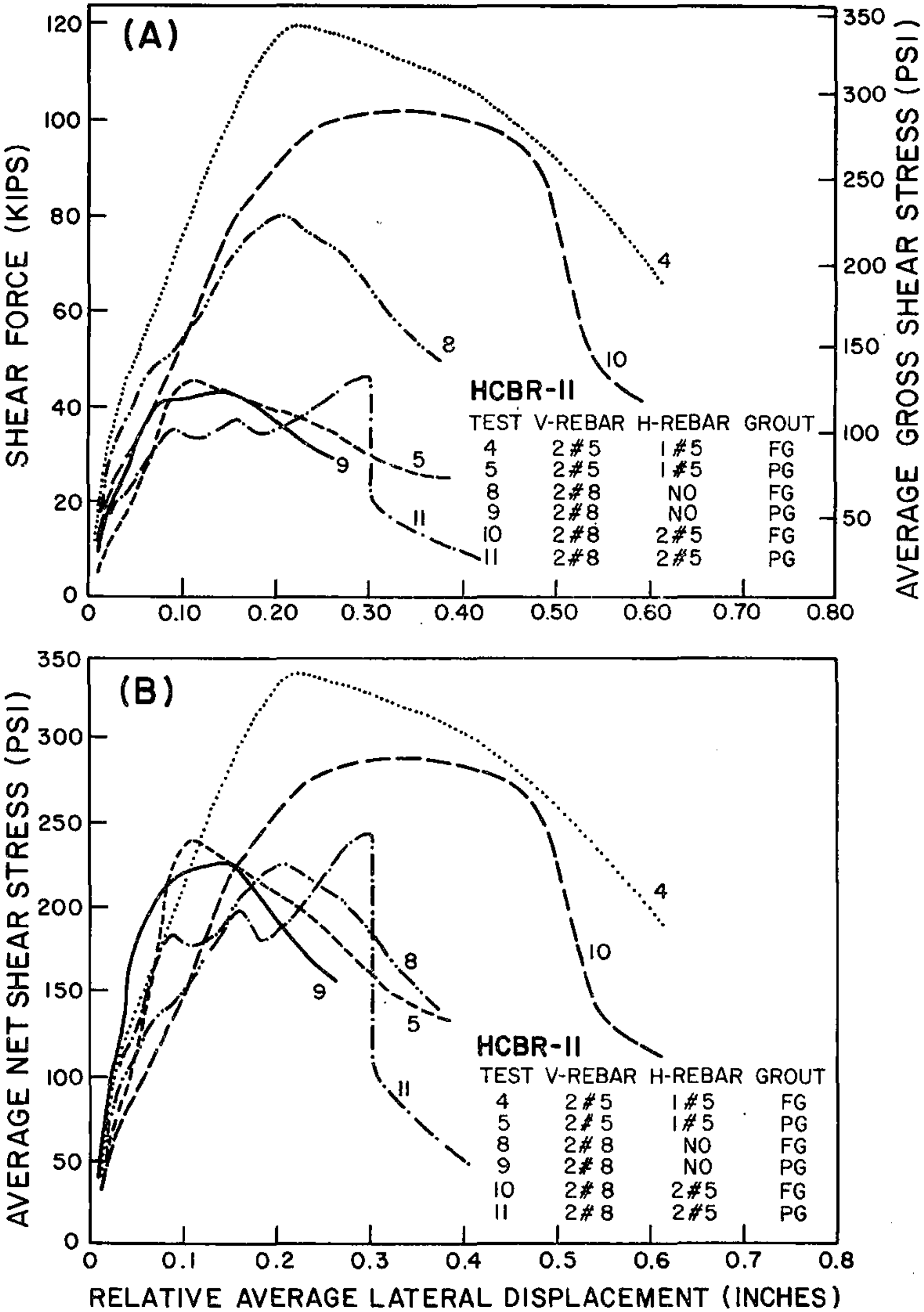


FIG. 2.16 EFFECT OF PARTIAL GROUTING ON HYSTERESIS ENVELOPE (HCBR-11)

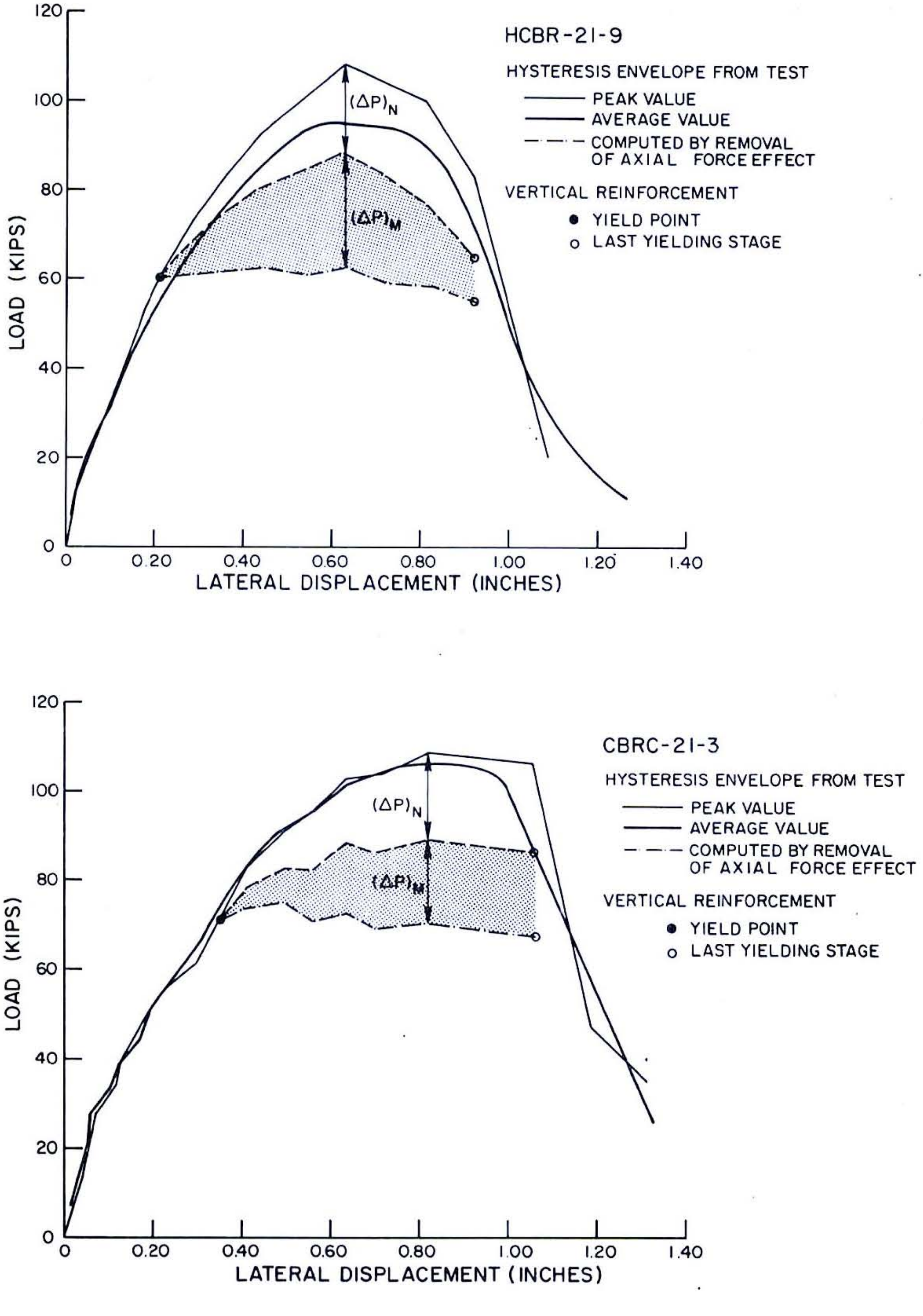


FIG. 2.17 EFFECT OF INCREASING AXIAL FORCE ON HYSTERESIS ENVELOPE

2.6.2.3 Effect of Type of Grouting

Partial grouting was included as a variable in a limited number of tests on both the hollow concrete block and hollow clay brick piers with height-to-width ratios of 2 and 1. It was not included in the tests on piers with a height-to-width ratio of 0.5.

For the hollow concrete block piers with a height-to-width ratio of 2 partial grouting had no significant effect on the net ultimate shear stress (see Table 2.3). In the 1 to 1 piers the net ultimate shear stress of the partially grouted piers was 20 to 30% greater than that of the fully grouted piers.

For the hollow clay brick piers partial grouting caused a reduction of 0 to 30% in the net ultimate shear stress of the piers (see Table 2.4). Furthermore, partial grouting significantly decreased the desirability of the inelastic performance of the piers as discussed in the following subsection.

2.6.3 Prediction of Ultimate Strength

The ultimate lateral load strength of each pier is determined by the lesser of the lateral load capacities associated with each of the two modes of failure. The ultimate strength associated with the sliding modes of failure has not yet been fully investigated and, therefore, the following discussion will be restricted to the flexural and the shear modes of failure.

The "flexural lateral load capacity" (lateral load capacity associated with the flexural mode of failure) is a function of the tensile yield strength of the vertical reinforcement, the applied axial load and the dimensions of the pier [8]. The methods suggested to predict the flexural lateral load capacity of a pier are similar and reasonably accurate, and are based on methods commonly used for reinforced concrete flexural elements. If all of the tension steel is assumed to be yielding, and the moment of the resultant of compressive forces around the extreme compression fiber is neglected, the moment capacity of a section under an axial compressive force N is given by

$$M = \sum A_{si} f_y d_i + N \frac{d}{2} \tag{2.25}$$

where d_i is the distance between the vertical reinforcing bar with area A_{si} and the extreme compressive fiber, d is the width of the pier and f_y is the yield strength of the vertical reinforcement (Fig. 2.21). If M_b and M_t denote the moment capacity of the bottom and top sections, respectively, of a pier of height h, the flexural lateral load capacity of a pier fixed against rotation at both top and bottom sections is

$$P = \frac{1}{h} (M_t + M_b). \tag{2.26}$$

If special devices such as those described in references 7 and 13 are used to increase the compressive strength of the masonry, the ultimate strength of the vertical steel f_u should be used in Eq. 2.25 instead of yield strength f_y to give an upper bound on the ultimate strength.

For the small number of piers that failed in the flexural mode of failure, Eq. 2.25 predicted the ultimate strength reasonably accurately. Furthermore, in Priestley's test program in which he extensively studied the flexural mode of failure, Eq. 2.25, using both f_y and f_u, gave the bounds for all the cantilever piers he tested [12], [13].

The "shear lateral load capacity" (lateral load capacity associated with the shear mode of failure) may be defined at two levels. The "shear crack strength" is defined as the lateral load required to produce the first major diagonal crack; the "ultimate shear strength" is the maximum lateral load resisted by the piers. In the case of the piers with height-to-width ratios of 2, both quantities are the same ([7], [9], [10]). In the case of the squat piers, (height-to-width ratio of 0.5), the lateral load continued to increase after the occurrence of the first major diagonal crack because the compression toe of the pier was wide enough to carry a significant shear. Increased amounts of cracking finally produced the failure of the pier at ultimate loads that exceeded the shear crack strength by percentages varying from 5% (CBRC piers), to 11% (HCBR piers), to 67% (HCBL piers).

Concurrent with the erection of the piers, prisms and square panels were constructed using the same mortar, grout and masonry units. The prisms were one block or brick wide, had the same thickness as the piers and a height five times the thickness. The square panels had the same thickness as the piers and the panel dimension was either 32 in. (HCBL) or 36 in. (HCBR and CBRC). The prisms were tested in uniaxial compression, the panels in diagonal compression (see Figs. 2.22 and 2.23). Tables 2.6 to 2.8 present the prism compressive strength f'_m, the panel critical tensile strength σ^o_{tcr}, as formulated by Blume [8], the pier strength associated with the occurrence of the first major diagonal crack τ_s, the average ultimate shear stress τ_u, and the pier critical tensile strength σ_{tcr}. The pier critical tensile stress was computed at the neutral axis of the pier sections, following the simple beam theory for a section under combined flexure, shear and axial force; a parabolic distribution of shear stress over the cross section was assumed. Tables 2.6 to 2.8 also present a comparison of the ratios $\sigma_{tcr}/\sigma^{o}_{tcr}$, $\tau_s/\sqrt{f'_m}$, and $\tau_u/\sqrt{f'_m}$ for all tests that failed in the shear mode of failure. Figure 2.18 is a plot of the average ultimate stress τ_u expressed in terms of $\sqrt{f'_m}$ versus the moment to shear ratio of the piers. Figures 2.19 and 2.20 are similar plots with the amount of horizontal reinforcement and axial stress, respectively, as the abscissa.

From Tables 2.6 to 2.8 and Figs. 2.24 and 2.25, it is clear that there is a very wide scatter both above and below 1 in the ratio $\sigma_{tcr}/\sigma^{o}_{tcr}$. This is somewhat surprising in that the square panel test induces a diagonal tension failure similar to that observed in the piers. However, it indicates that a prediction of the shear crack strength based on the critical tensile strength measured from a diagonal compression test on a square panel test must account for the scatter and lower bound values obtained in this program. Because of the need for conservatism in utilizing this test data some other method of predicting the shear crack strength may be more appropriate.

The ratios of τ_s and τ_u to $\sqrt{f'_m}$ shown in Figs. 2.18 through 2.20 and Tables 2.6 to 2.8, also contain a significant amount of scatter, although at this time, prediction of the shear crack strength or ultimate strength based on f'_m and height-to-width ratio appears to be a reasonable approach.

This statement should be qualified at this time because piers of similar dimensions and reinforcement ratios have not been tested where f'_m varies significantly, and piers with height-to-width ratios greater than 2 have not been tested.

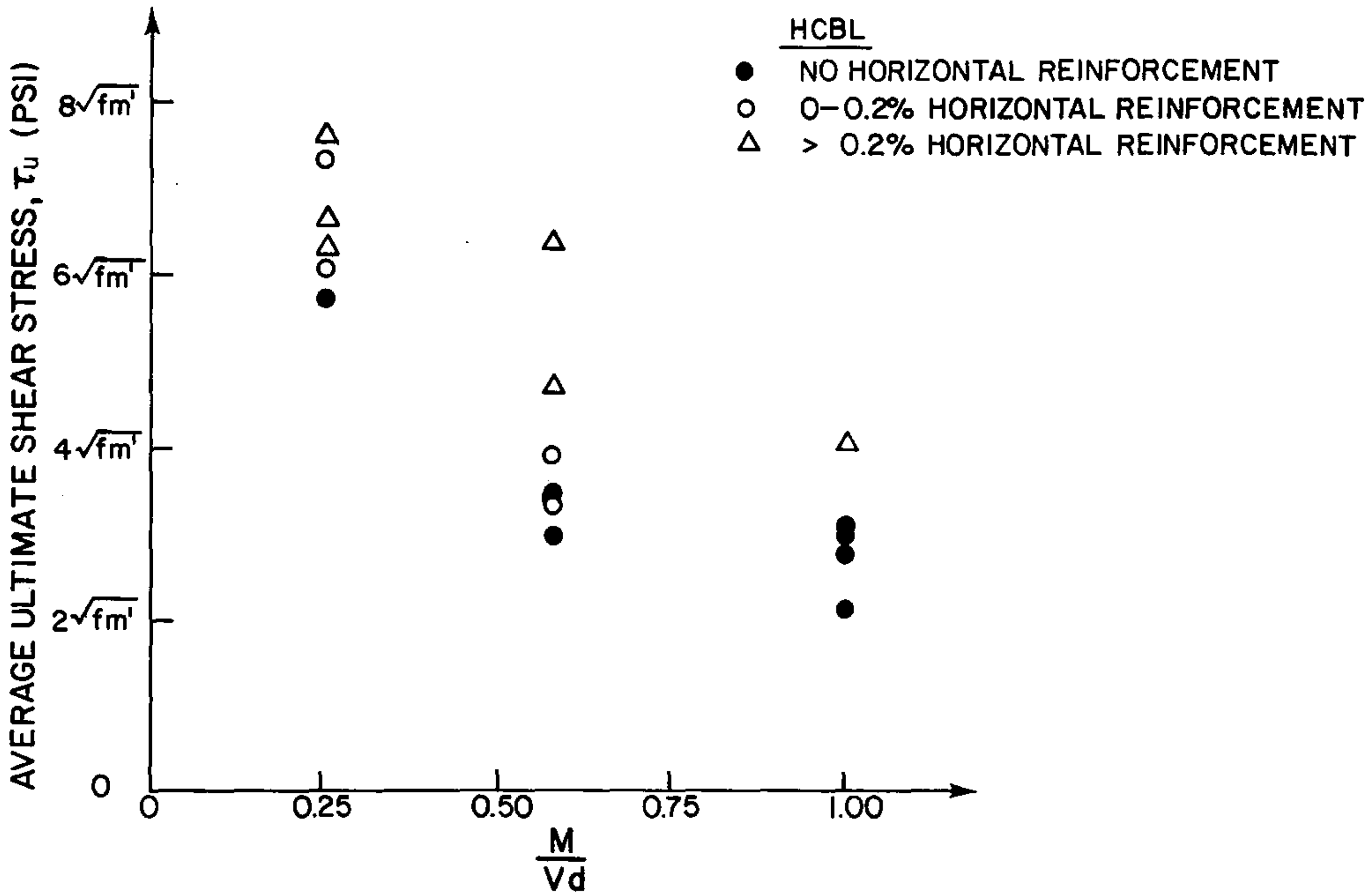


FIG. 2.18a ULTIMATE SHEAR STRESS VS. MOMENT-TO-SHEAR RATIO (HCBL)

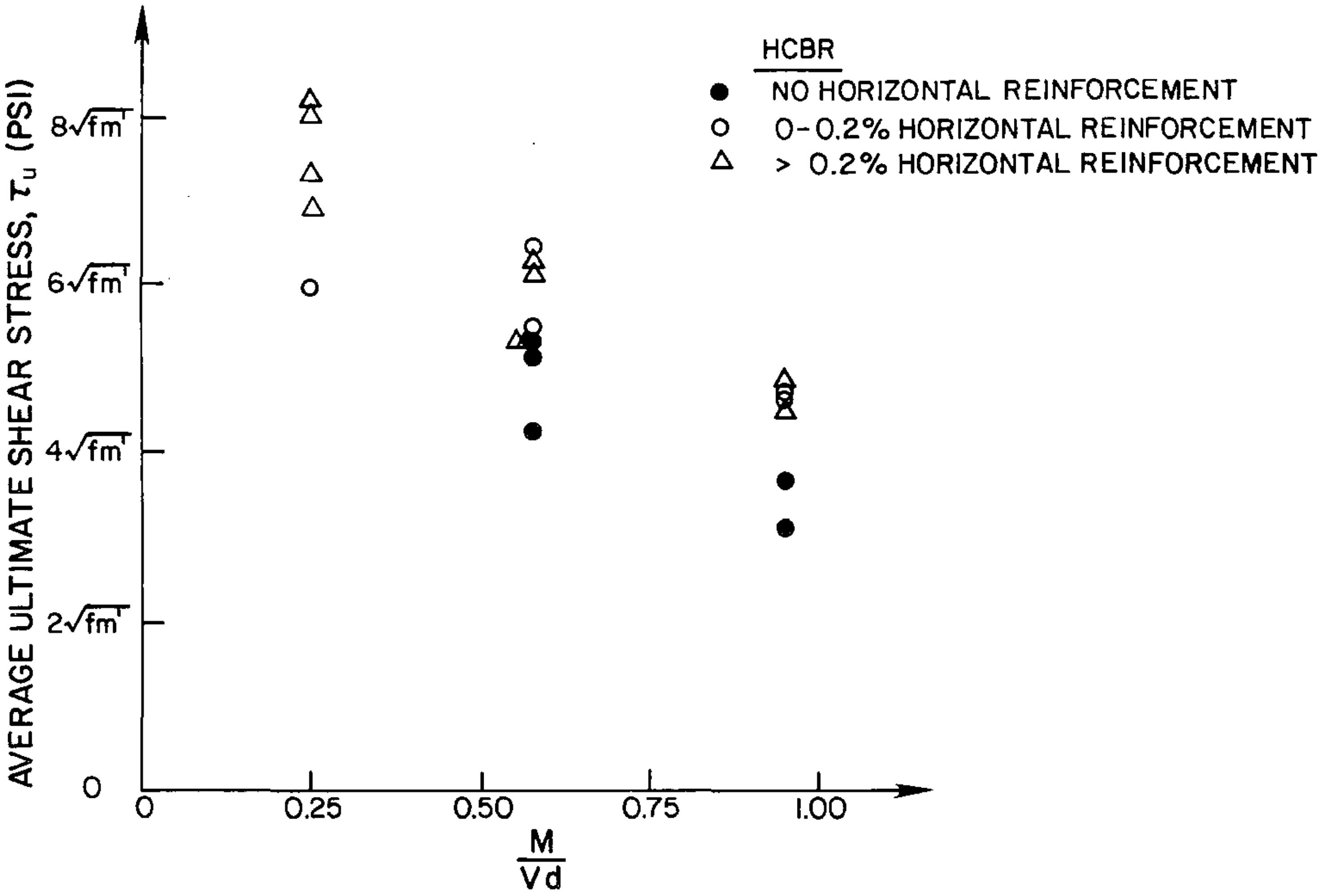


FIG. 2.18b ULTIMATE SHEAR STRESS VS. MOMENT-TO-SHEAR RATIO (HCBR)

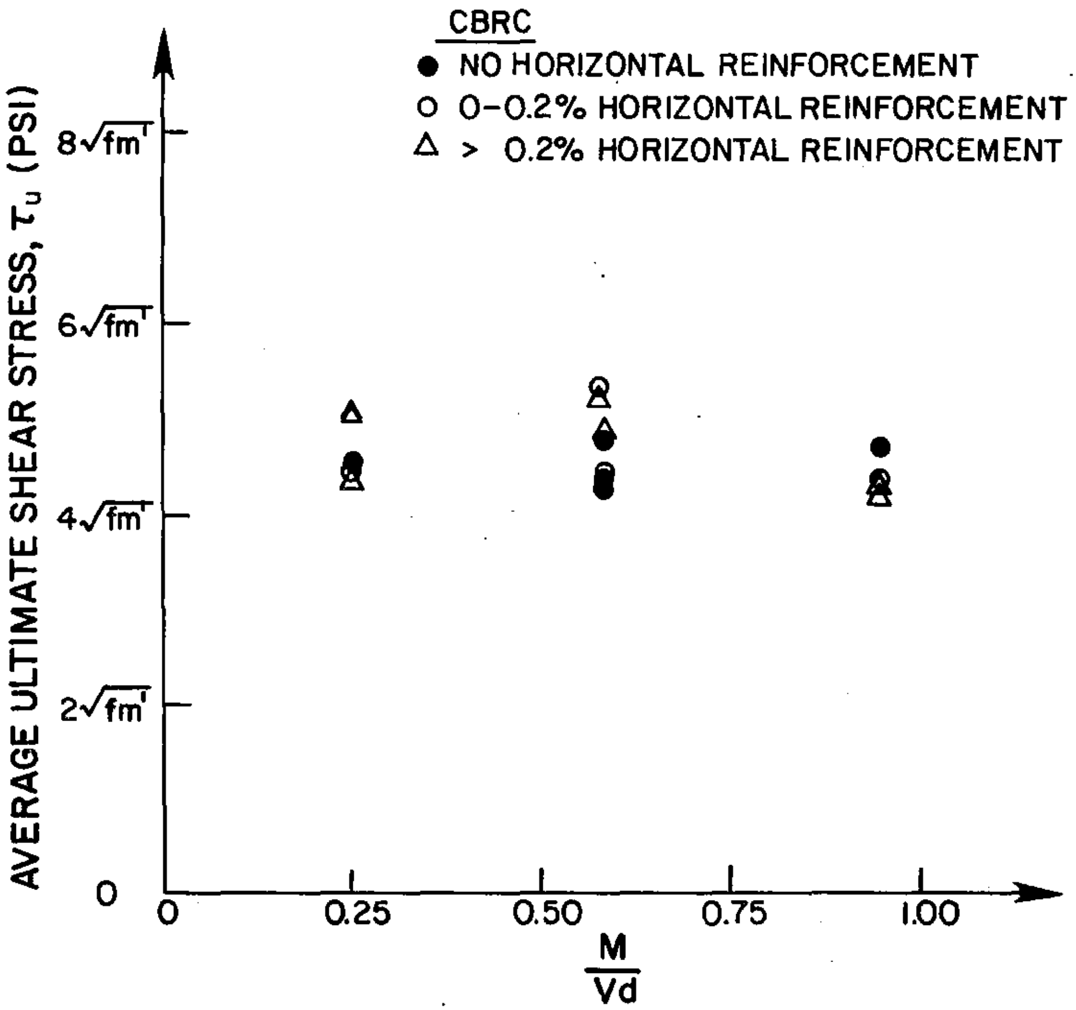


FIG. 2.18c ULTIMATE SHEAR STRESS VS. MOMENT-TO-SHEAR RATIO (CBRC)

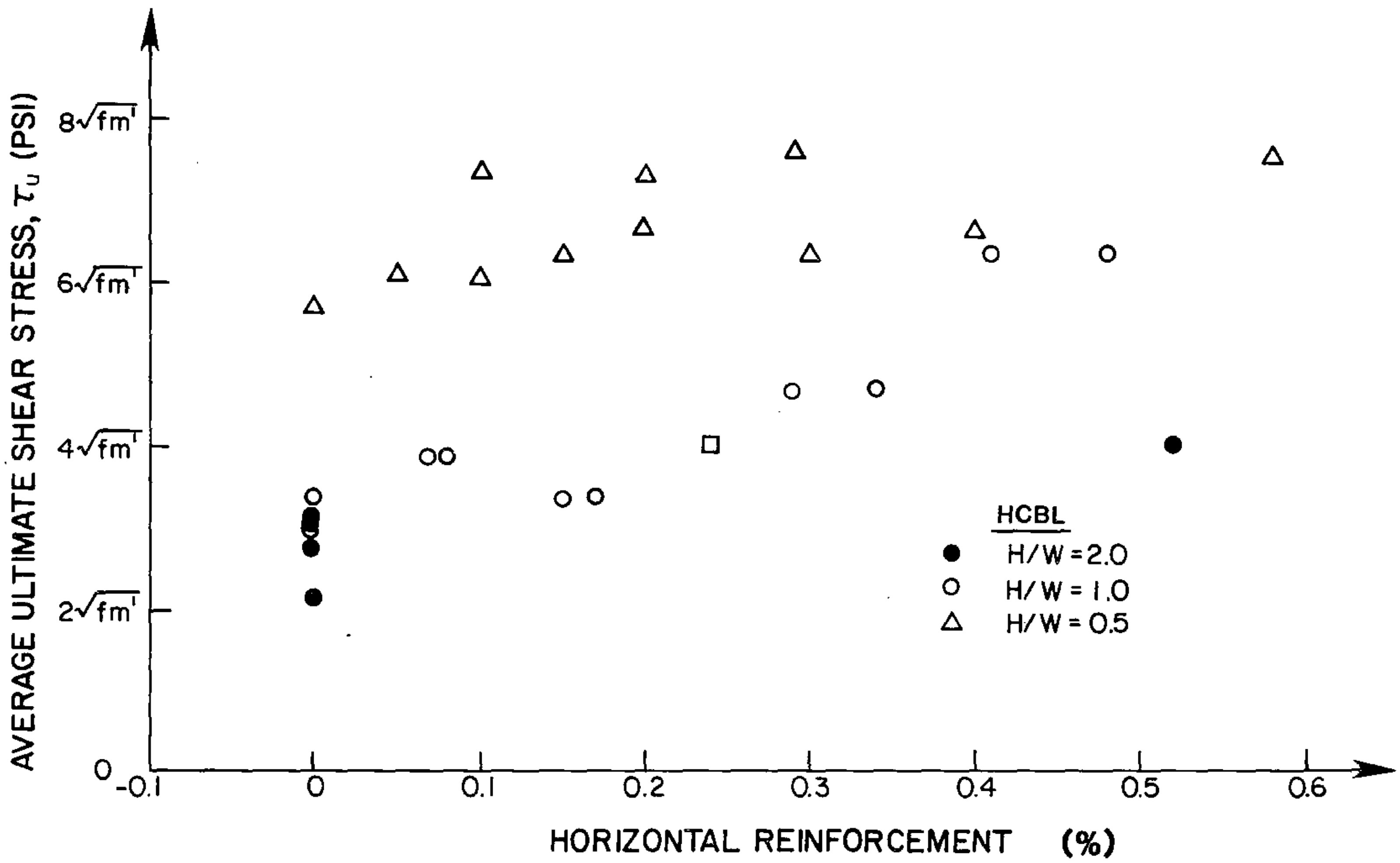


FIG. 2.19a EFFECT OF HORIZONTAL REINFORCEMENT ON ULTIMATE SHEAR STRESS (HCBL)

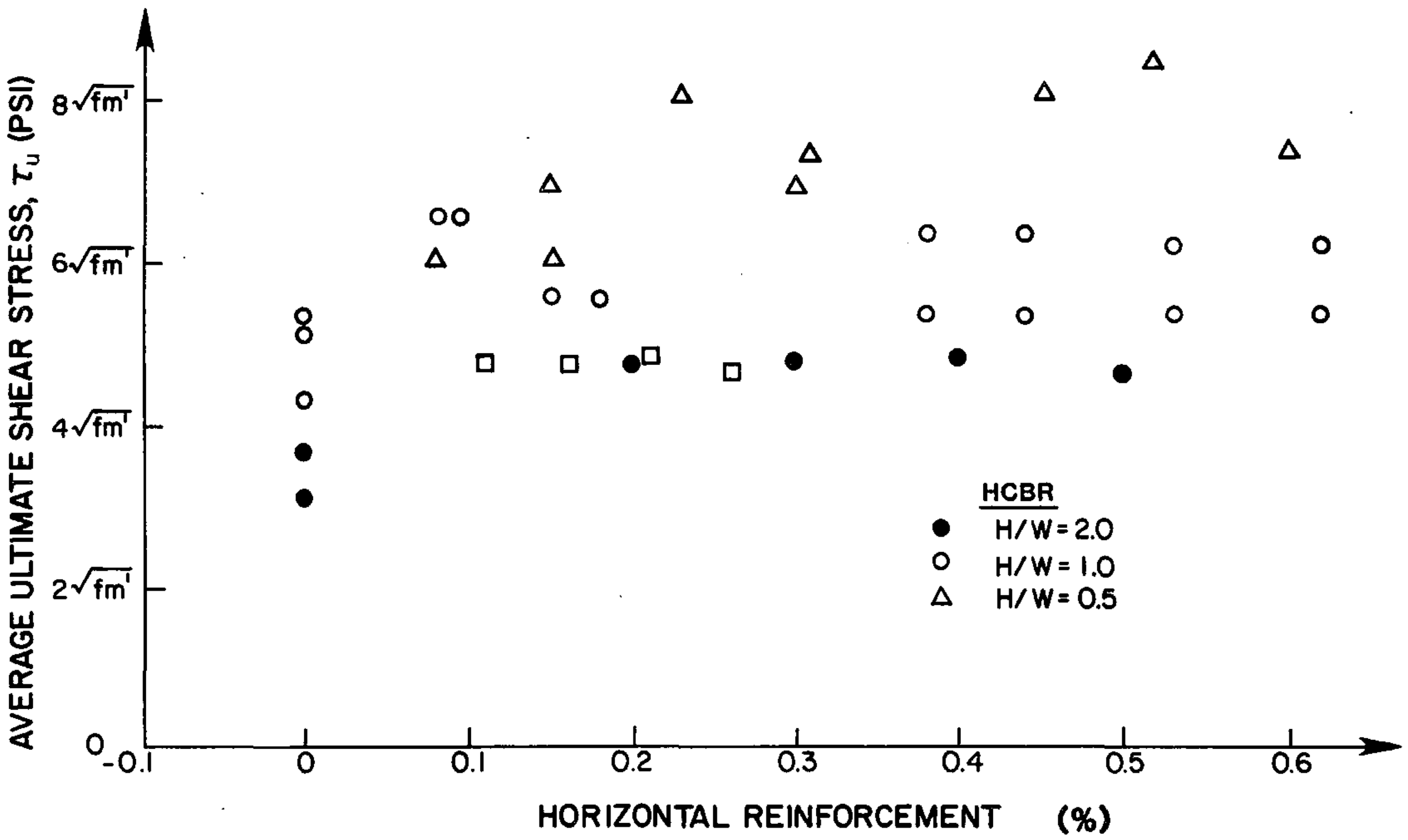


FIG. 2.19b EFFECT OF HORIZONTAL REINFORCEMENT ON ULTIMATE SHEAR STRESS (HCBR)

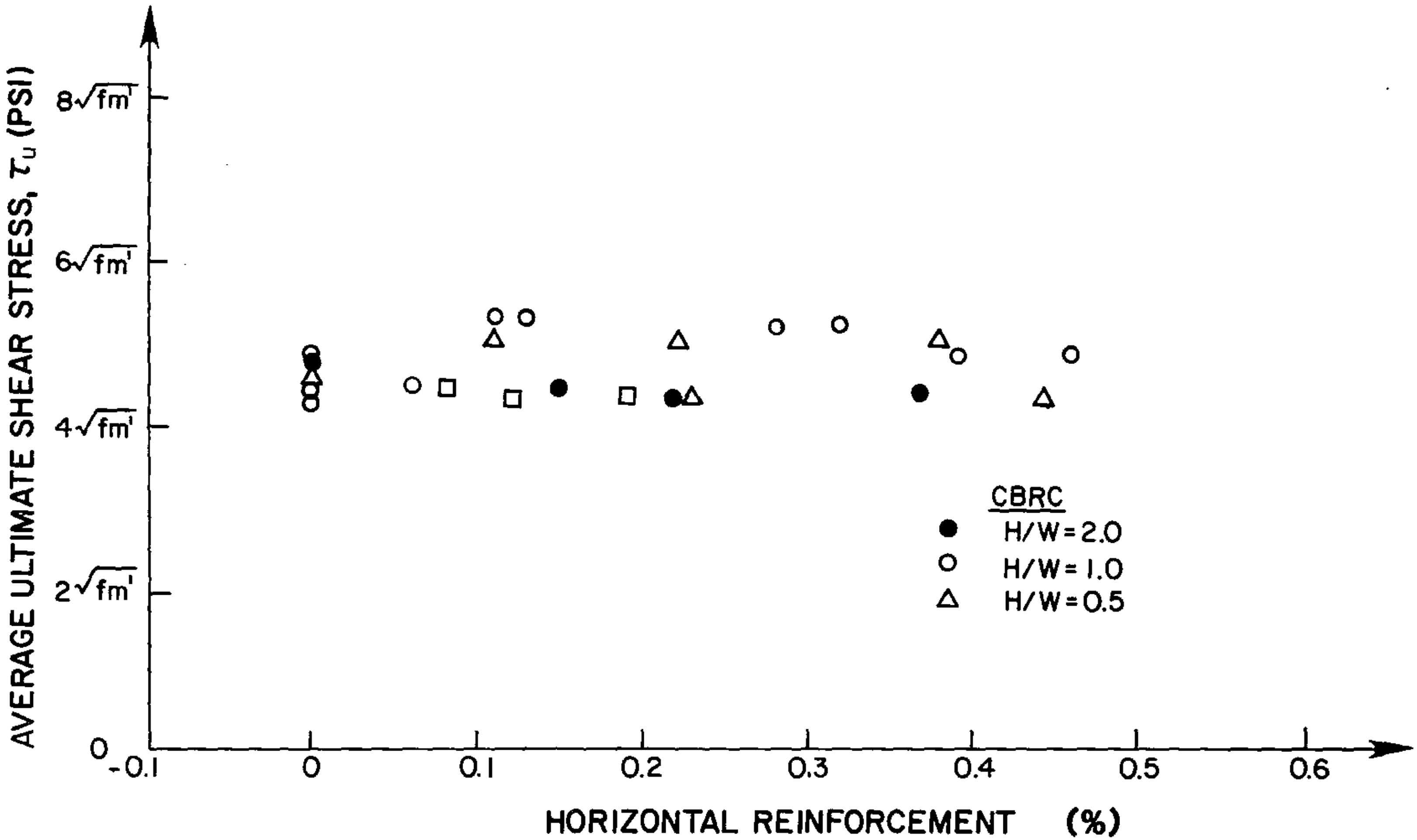


FIG. 2.19c EFFECT OF HORIZONTAL REINFORCEMENT ON ULTIMATE SHEAR STRESS (CBRC)

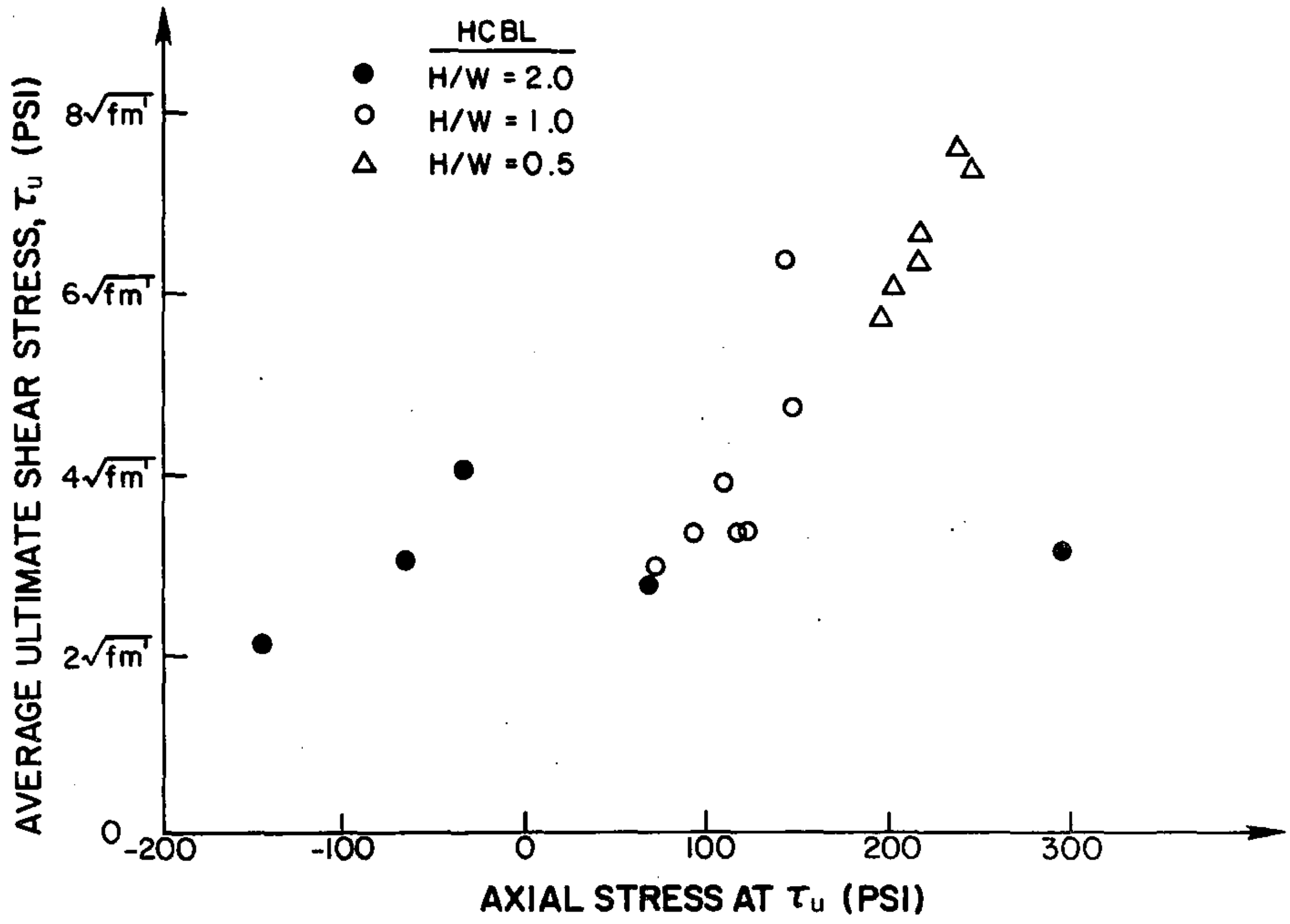


FIG. 2.20a EFFECT OF AXIAL STRESS ON ULTIMATE SHEAR STRESS (HCBL)

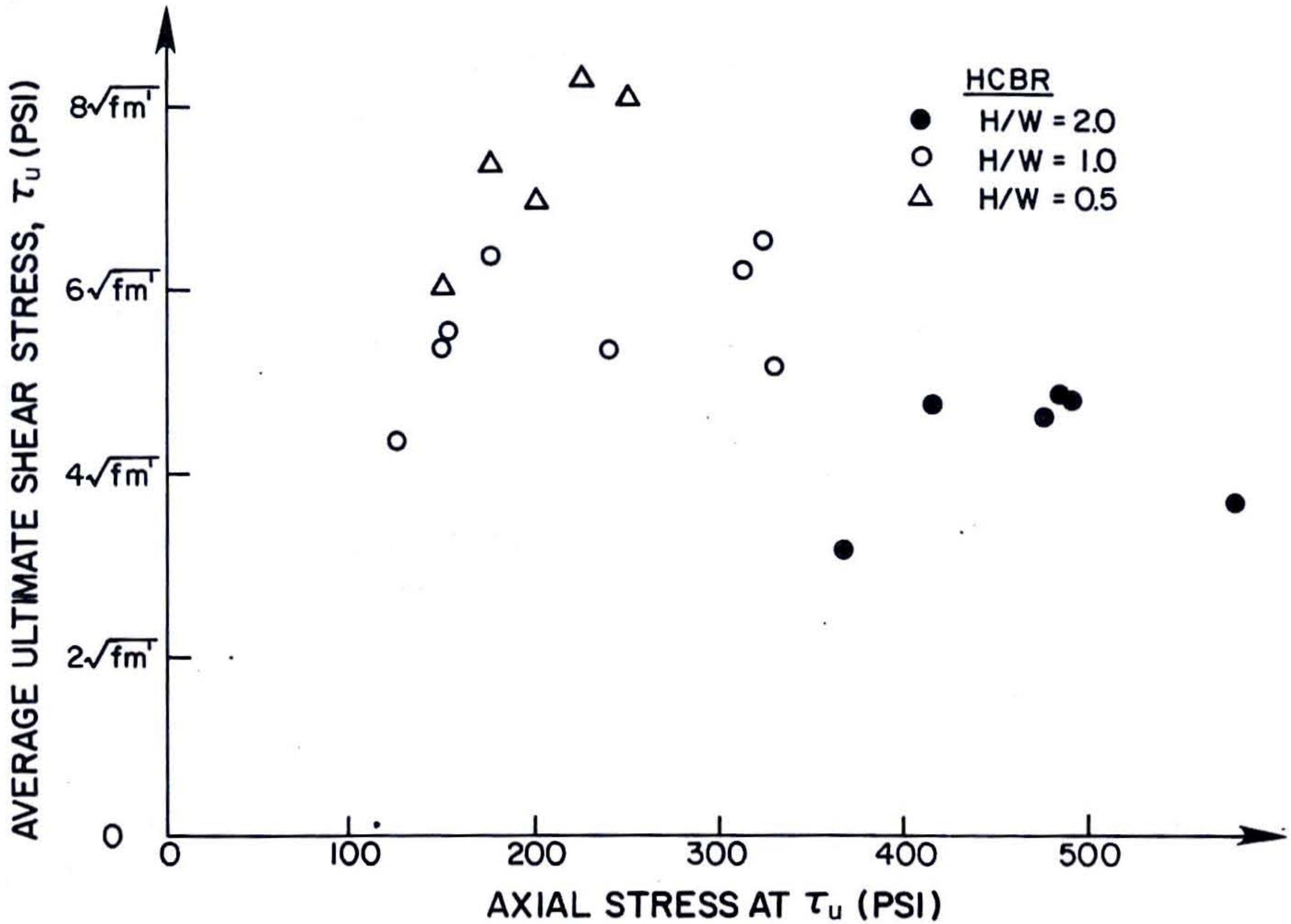


FIG. 2.20b EFFECT OF AXIAL STRESS ON ULTIMATE SHEAR STRESS (HCBR)

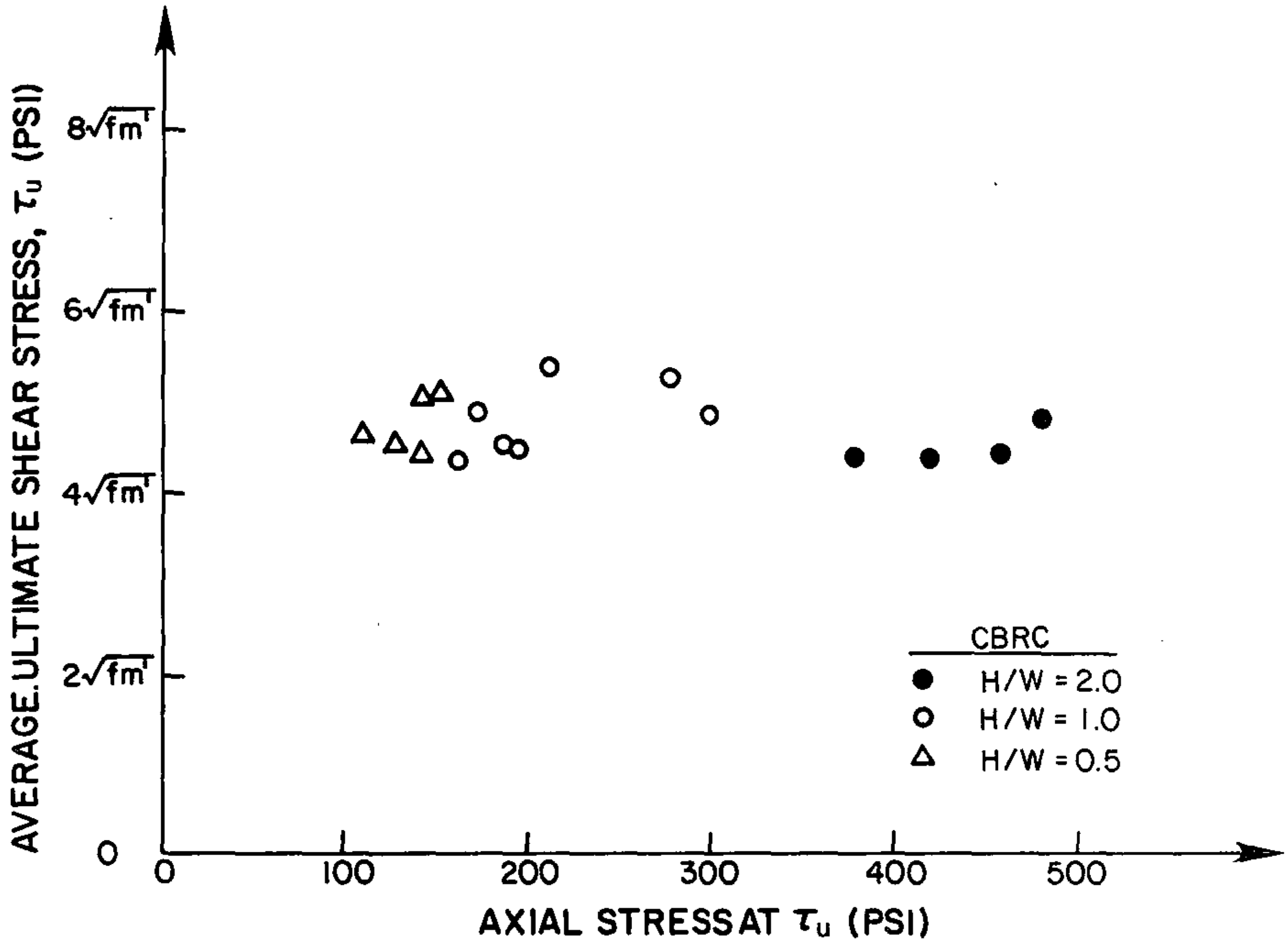
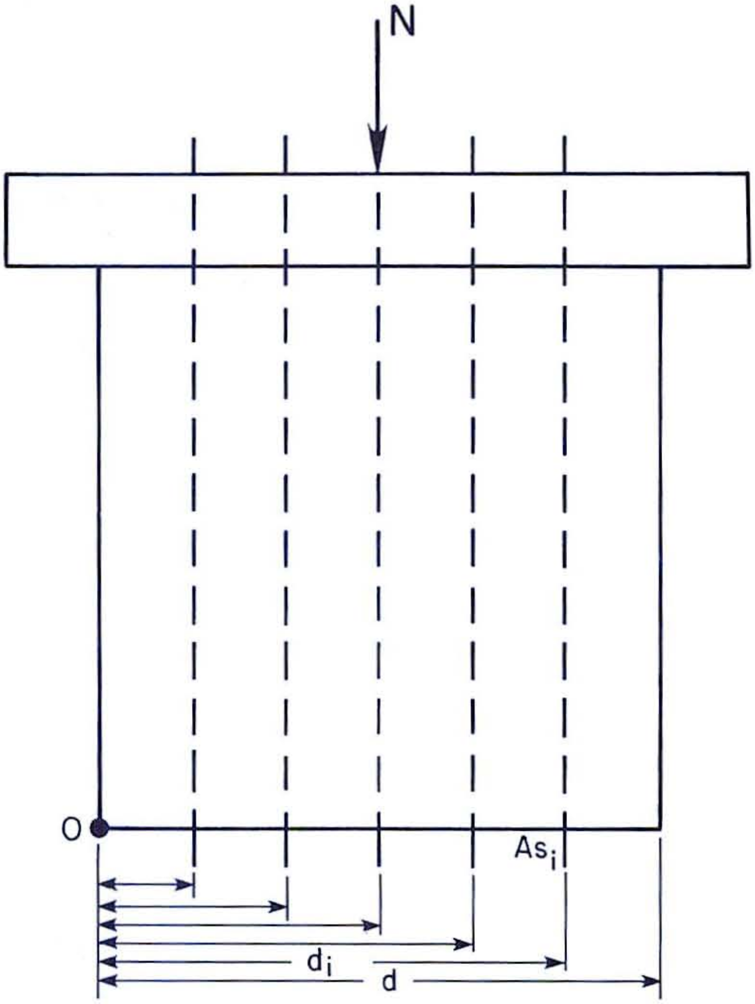


FIG. 2.20c EFFECT OF AXIAL STRESS ON ULTIMATE SHEAR STRESS (CBRC)



$$M_O = \left(\sum A_{s_i} f_y d_i \right) + N \, d/2$$

FIG. 2.21 APPROXIMATE MOMENT CAPACITY OF A SECTION

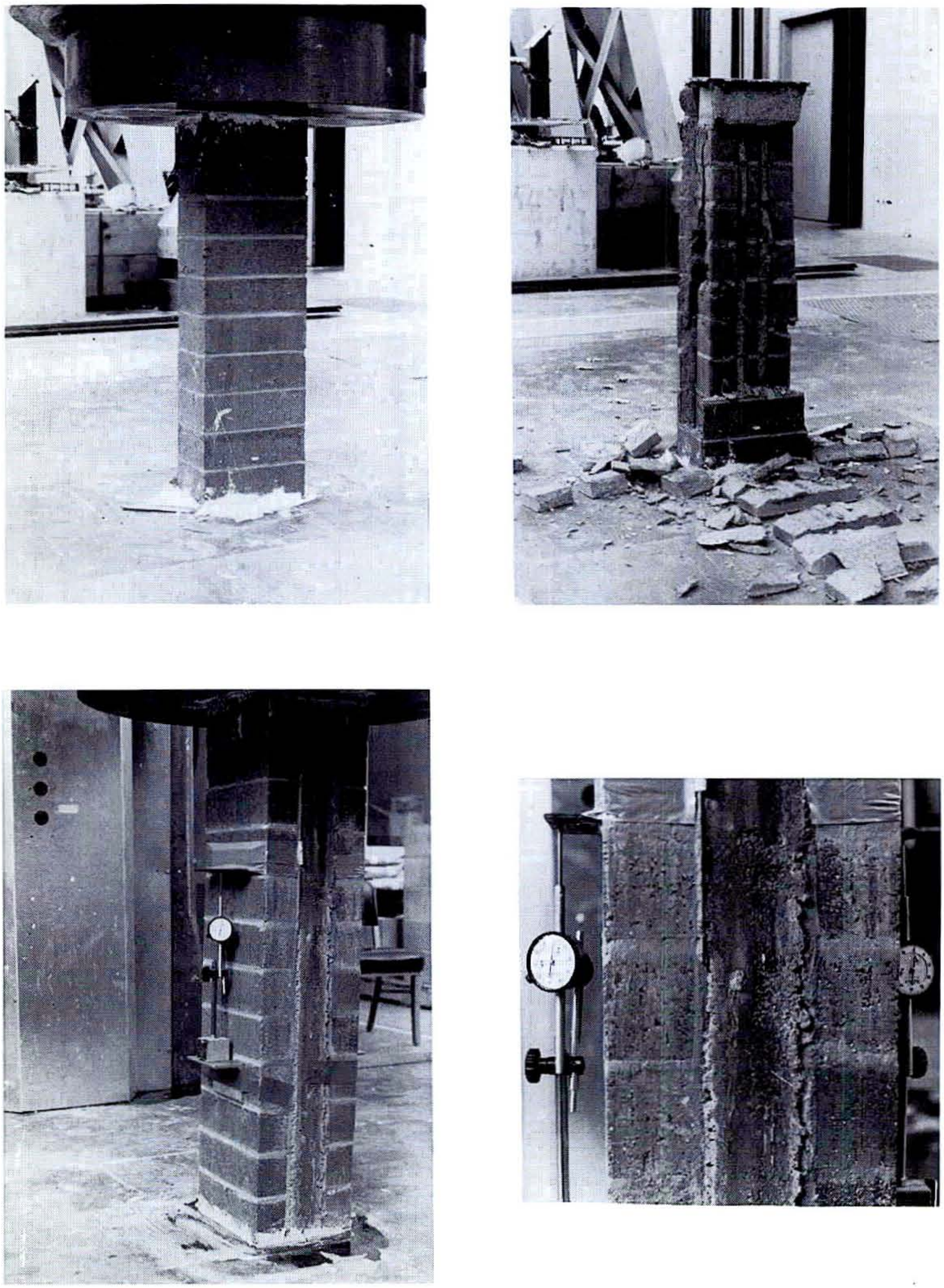


FIG. 2.22 PRISM TEST AND MODULUS OF ELASTICITY MEASUREMENT

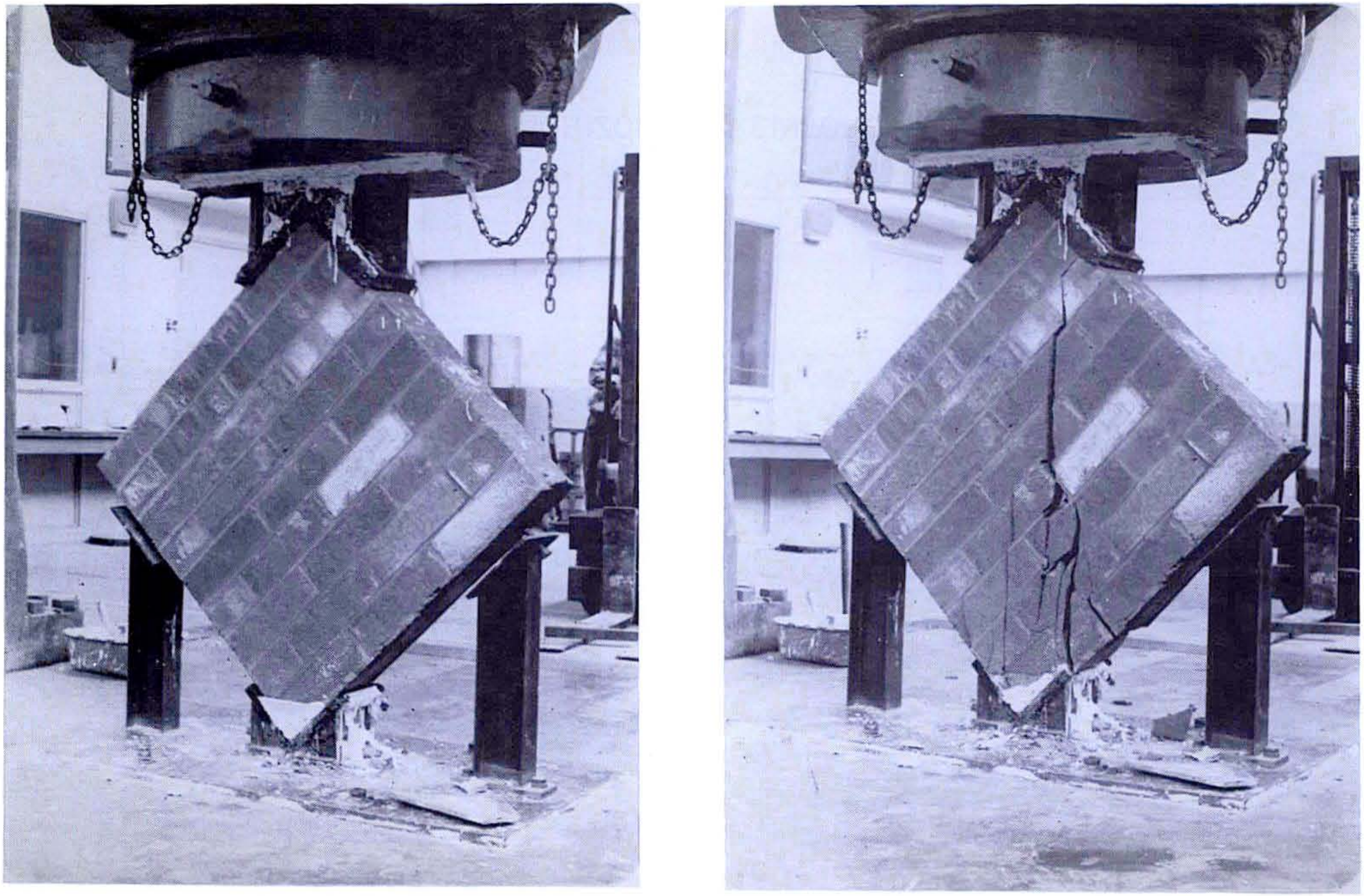


FIG. 2.23 SQUARE PANEL TEST

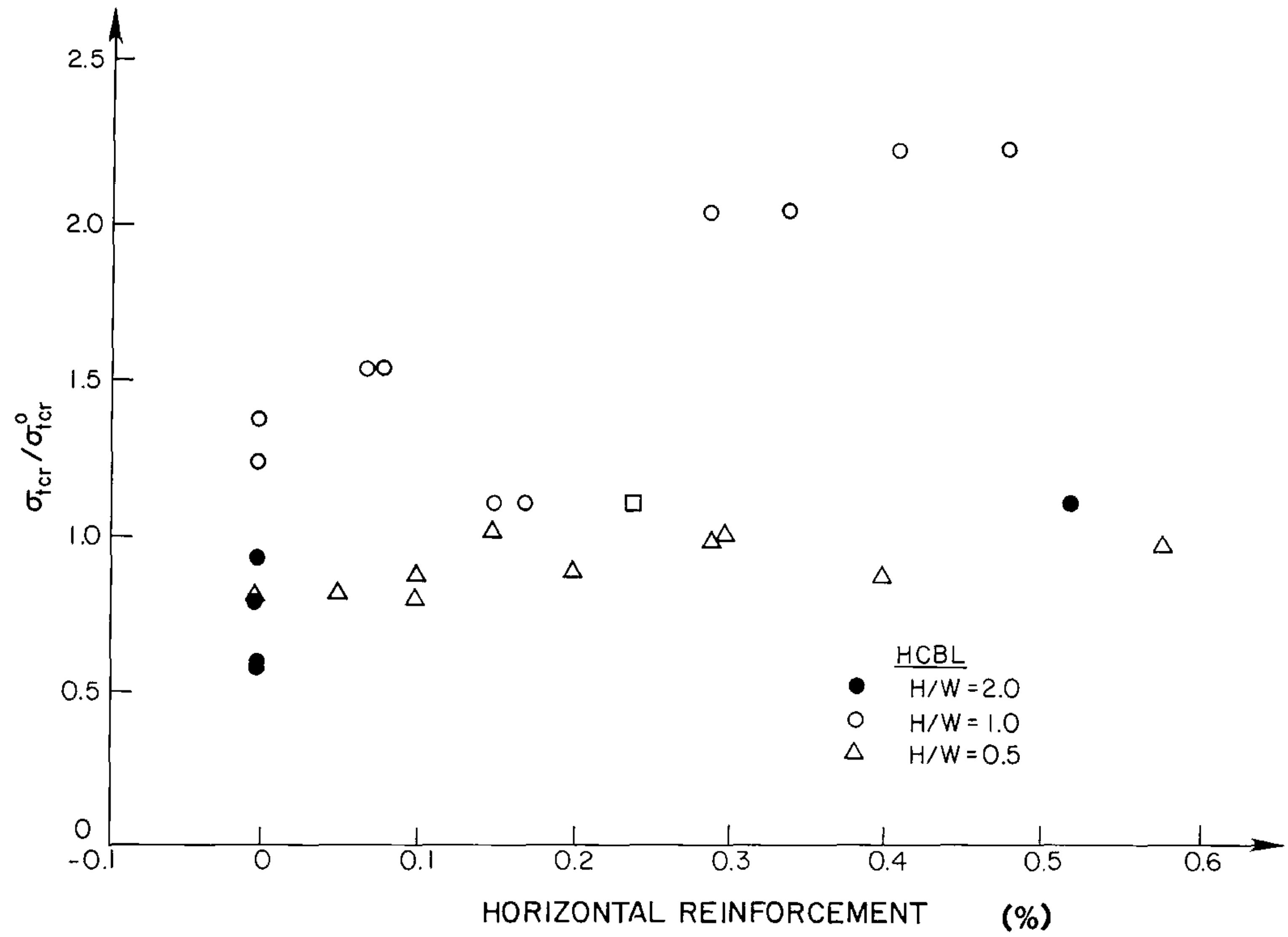


FIG. 2.24a EFFECT OF HORIZONTAL REINFORCEMENT ON THE RATIO OF THE PANEL CRITICAL STRENGTH TO THE PRISM CRITICAL STRENGTH (HCBL)

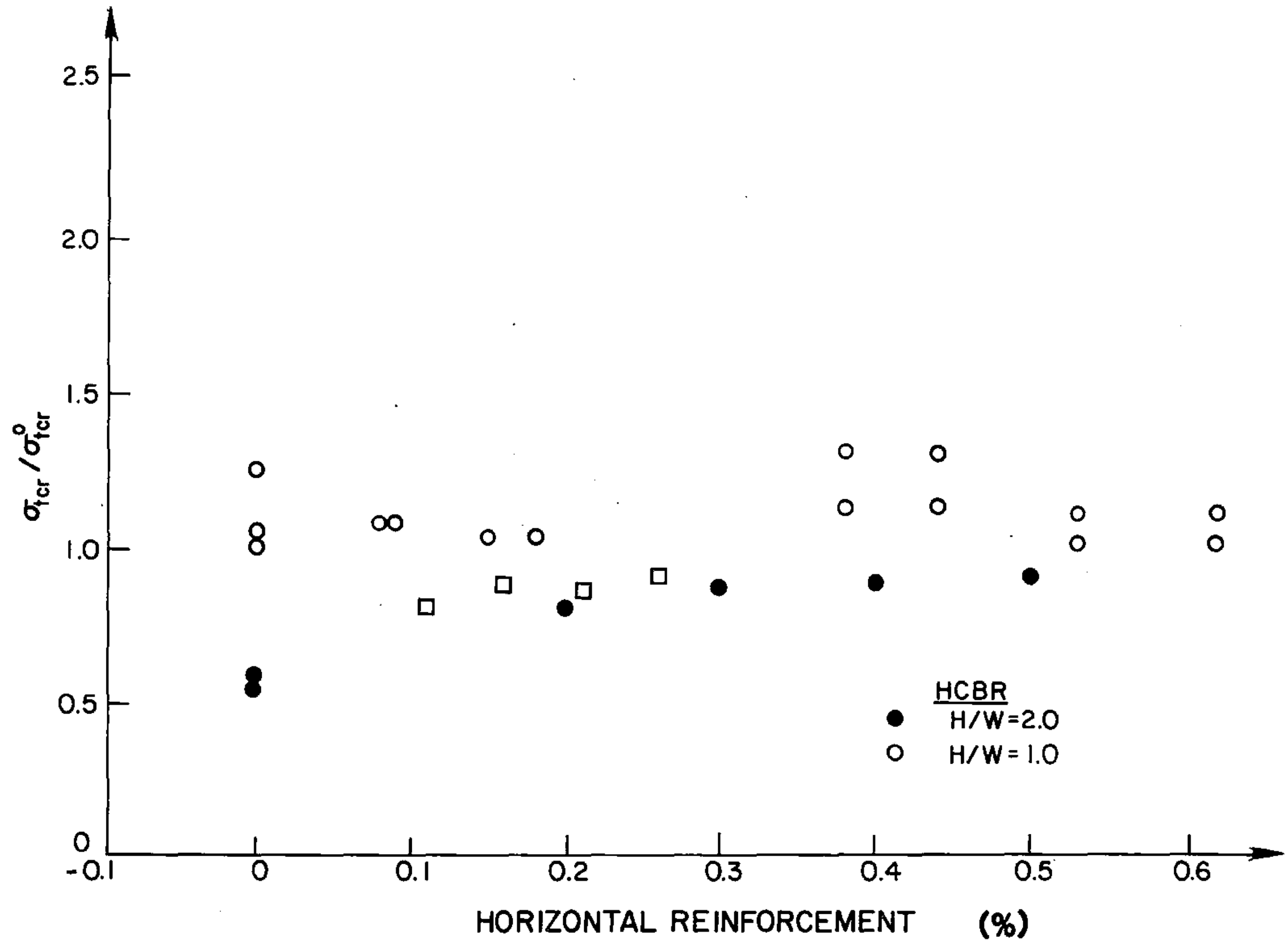


FIG. 2.24b EFFECT OF HORIZONTAL REINFORCEMENT ON THE RATIO OF THE PANEL CRITICAL STRENGTH TO THE PRISM CRITICAL STRENGTH (HCBR)

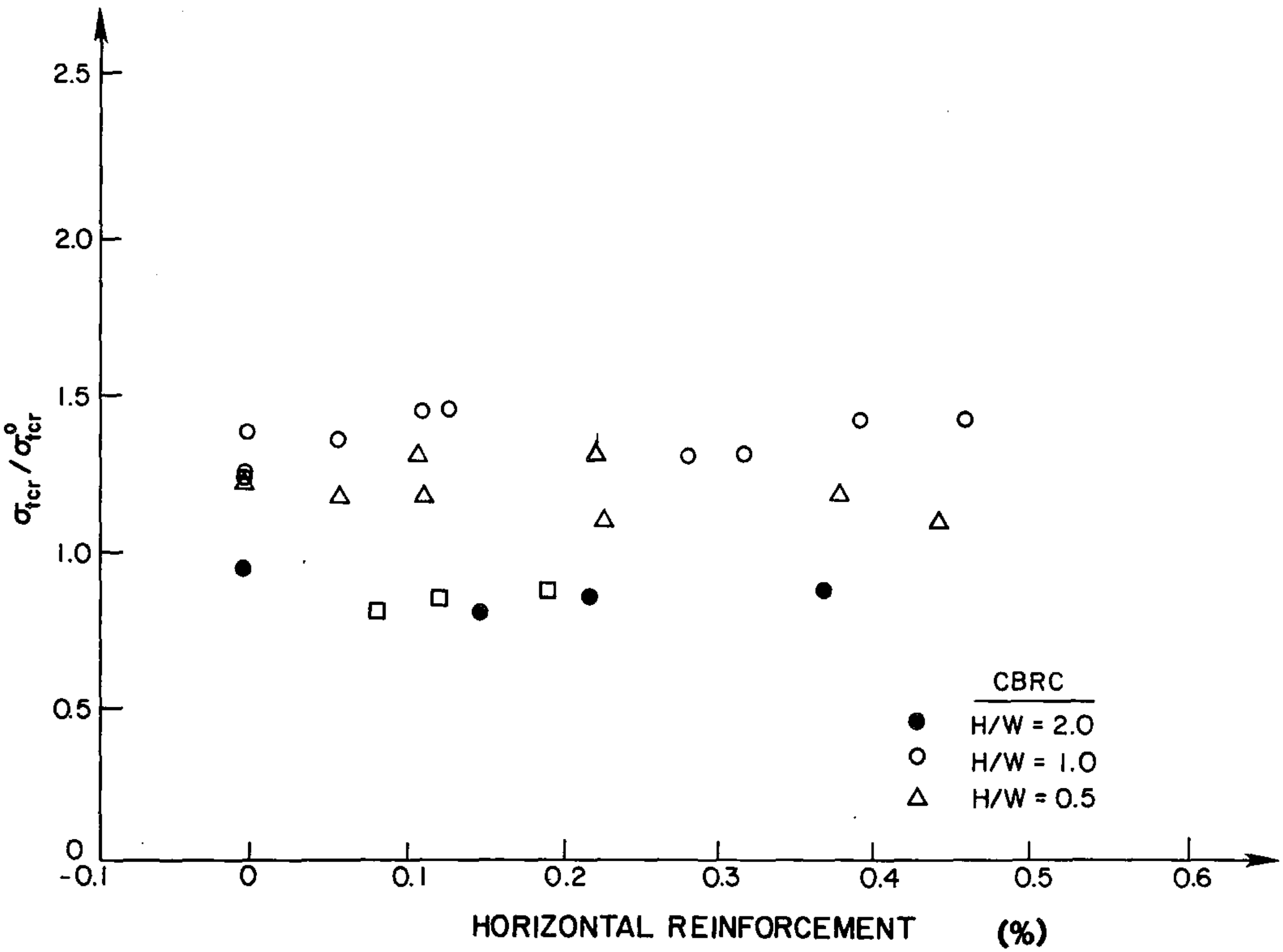


FIG. 2.24c EFFECT OF HORIZONTAL REINFORCEMENT ON THE RATIO OF THE PANEL CRITICAL STRENGTH TO THE PRISM CRITICAL STRENGTH (CBRC)

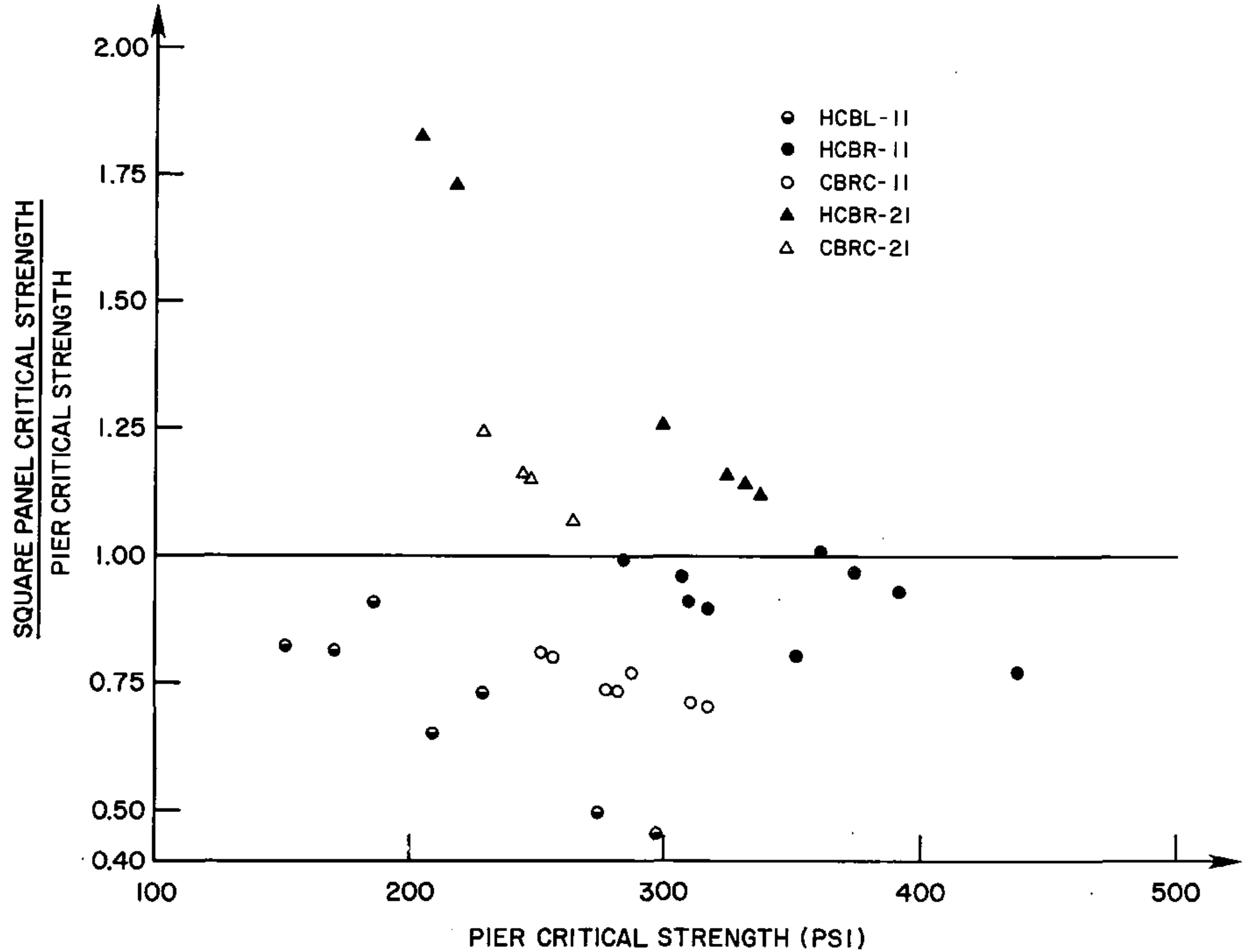


FIG. 2.25 COMPARISON OF PIER CRITICAL STRENGTH AND PANEL CRITICAL STRENGTH

steel is present. The case of light horizontal reinforcement is defined as the ratio of the area of reinforcing steel to the gross vertical area of the pier being less than 0.002, whereas a heavily horizontally reinforced pier has a ratio greater than 0.002. In these two cases jamb steel is assumed to be present.

Some of the recommended values of $\tau_u/\sqrt{f'_m}$ are on the conservative side, but they must account for the effects of some of the variables that have not been included in the test program, such as variations in workmanship, piers with M/Vd ratios greater than 1, and significant variations in f'_m. In addition, the recommended values of 1 and 0 for M/Vd must provide a reasonable estimate when interpolated for an M/Vd ratio of 0.5.

The Newmark-Hall method of reducing the elastic spectra to account for ductility is based on an idealized elasto-plastic force-deflection relationship as discussed in Section 2.5.3. Consequently, the hysteresis envelopes of Figs. 2.6 to 2.17 must be idealized as elasto-plastic curves. To do this, an appropriate ductility factor and corresponding ultimate strength must be evaluated from the test results. For the horizontally reinforced walls a ductility factor of 2 is assumed to represent the test results conservatively. For the piers with light horizontal reinforcement the associated ultimate strength is assessed at 80% of the recommended value given in Table 2.9. For heavily horizontally reinforced walls the ultimate strength associated with a ductility factor of 3 is assessed at 80% of the recommended values given in Table 2.9. For the walls with jamb steel only, no ductile response is assumed and thus 100% of the recommended values of Table 2.9 are used with a ductility factor of 1.

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The ductility factor of 2 for walls with light horizontal reinforcement was assessed from the test results reported here and shown in Figs. 2.6 to 2.17. The limited number of tests since performed with the modified single pier test setup indicates that at high constant compressive loads the post cracking behavior of the piers is more brittle, and therefore less desirable, than that observed in the tests described here. This performance may only be for piers subjected to high compressive loads, but until more test data are available the conservative value of 2 will be used.

The factors for ductility and ultimate strength discussed above are applicable for partially grouted hollow concrete block piers, but they are not applicable for partially grouted hollow clay brick piers. As discussed previously, hollow clay brick piers have little or no ductile capacity and their net ultimate strength is 70 to 100% less than that of the fully grouted piers. Therefore, the analysis presented in the following chapters is applicable to partially grouted hollow clay brick piers if a ductility factor of 1 is used in conjunction with 70% of the recommended net ultimate strengths of fully grouted piers.

TABLE 2.6

PREDICTION OF SHEAR CRACK STRENGTH FOR FULLY GROUTED PIERS, HCBL

Specimen	Vertical Steel Rein-forcement (%)	Horizontal Steel Rein-forcement (%)	Prism Compressive Strength f'_m (psi)	Square Panel Crit. Tens. Str. σ^o_{tcr} (psi)	Pier Shear Crack Strength τ_s (psi)	Pier Axial Stress at Shear Crack σ_c (psi)	Average Ultimate Shear Stress τ_u (psi)	Pier Axial Stress at τ_u (psi)	Pier Crit. Tensile Strength σ_{tcr} (psi)	$\frac{\sigma_{tcr}}{\sigma^o_{tcr}}$	$\frac{\tau_s}{\sqrt{f'_m}}$	$\frac{\tau_u}{\sqrt{f'_m}}$
HCBL-21-1	0.92	--	2432	320	144	- 67	135	- 77	186	0.58	2.92	2.74
-3	0.42	--	2256	337	152	- 68	142	+ 53	264	0.78	3.20	2.99
-5	0.92	--	2592	280	114	+145	106	+119	258	0.92	2.24	2.08
-7	0.92	0.24	2805	326	226	+ 37	212	+ 15	358	1.10	4.27	4.00
-9	0.92	--	2519	244	164	-292	154	-308	140	0.57	3.27	3.06
HCBL-11-1	--	--	1330	124	135	-120	123	-120	151	1.22	3.70	3.37
-3	0.17	--	1833	137	134	- 69	127	- 69	170	1.24	3.13	2.97
-4	0.17	0.07	1833	137	171	-107	165	-107	209	1.53	3.99	3.85
-6	0.17	0.29	1833	135	226	-144	199	-144	275	2.04	5.28	4.65
-7	0.43	--	1905	166	180	- 91	146	- 91	228	1.37	4.12	3.35
-9	0.43	0.15	1905	166	155	-114	146	-114	183	1.10	3.55	3.35
-11	0.43	0.41	1330	133	240	-139	231	-139	297	2.23	6.58	6.33
HCBL-12-1	0.30	--	2988	330	200	- 85	310	-194	260	0.79	3.66	5.67
-2	0.30	0.10	2988	330	206	- 86	330	-200	268	0.81	3.77	6.03
-3	0.30	0.20	2988	330	215	- 83	398	-243	283	0.86	3.93	7.28
-4	0.30	0.30	2988	330	261	-127	344	-212	333	1.01	4.77	6.29
-5	0.30	0.40	2988	330	226	-106	361	-215	290	0.88	4.13	6.60
-6	0.30	0.58	2988	330	244	-102	413	-234	319	0.97	4.46	7.56

NOTE: (1) The prism strength is based on a h/d ratio of 5

TABLE 2.7

PREDICTION OF SHEAR CRACK STRENGTH FOR FULLY GROUTED PIERS, HCBR

Specimen	Vertical Steel Rein-forcement (%)	Horizontal Steel Rein-forcement (%)	Prism Compressive Strength f'_m (psi)	Square Panel Crit. Tens. Str. σ^o_{tcr} (psi)	Pier Shear Crack Strength τ_s (psi)	Pier Axial Stress at Shear Crack σ_c (psi)	Average Ultimate Shear Stress τ_u (psi)	Pier Axial Stress at τ_u (psi)	Pier Crit. Tensile Strength σ_{tcr} (psi)	$\frac{\sigma_{tcr}}{\sigma^o_{tcr}}$	$\frac{\tau_s}{\sqrt{f'_m}}$	$\frac{\tau_u}{\sqrt{f'_m}}$
HCBR-21-1	--	--	4502	375	267	-580	244	580	204	0.54	3.98	3.64
-2	0.51	--	4502	375	238	-368	206	368	218	0.58	3.55	3.07
-4	0.51	0.11	4502	375	308	-415	273	415	299	0.80	4.59	4.67
-6	0.51	0.16	4502	375	343	-492	317	492	325	0.87	5.11	4.72
-8	0.51	0.21	4502	375	346	-485	321	485	331	0.88	5.16	4.78
-9	0.51	0.26	4502	375	348	-476	307	476	336	0.90	5.19	4.58
HCBR-11-1	--	--	2535	282	278	-328	255	328	284	1.01	5.52	5.06
-3	0.18	--	2535	282	279	-148	267	148	352	1.25	5.54	5.30
-4	0.18	0.08	2722	363	353	-323	337	323	391	1.08	6.77	6.46
-6	0.18	0.38	2722	336	346	-175	328	175	438	1.30	6.63	6.29
-7	0.18	0.38	2535	282	280	-241	267	241	317	1.12	5.56	5.30
-8	0.45	--	2866	293	242	-123	227	123	307	1.05	4.52	4.24
-10	0.45	0.15	2722	363	296	-153	287	153	374	1.03	5.67	5.50
-12	0.45	0.53	2535	282	275	-240	266	240	309	1.10	5.46	5.28
-13	0.45	0.53	2722	363	329	-312	320	312	361	0.99	6.31	6.13
HCBR-12-2	0.31	0.15	2838	--	319	-125	318	149	420	--	5.99	5.97
-3	0.31	0.30	2838	--	351	-150	368	198	457	--	6.59	6.91
-4	0.31	0.45	2838	--	356	-143	427	248	467	--	6.68	8.02
-5	0.31	0.60	2838	--	394	-154	389	175	519	--	7.40	7.30
-6	0.31	1.02	2838	--	392	-153	437	223	516	--	7.36	8.20

NOTE: (1) The prism strength is based on a h/d ratio of 5

TABLE 2.8

PREDICTION OF SHEAR CRACK STRENGTH FOR FULLY GROUTED PIERS, CBRC

Specimen	Vertical Steel Rein-forcement (%)	Horizontal Steel Rein-forcement (%)	Prism Compressive Strength f'_m (psi)	Square Panel Crit. Tens. Str. σ^o_{tcr} (psi)	Pier Shear Crack Strength τ_s (psi)	Pier Axial Stress at Shear Crack σ_c (psi)	Average Ultimate Shear Stress τ_u (psi)	Pier Axial Stress at τ_u (psi)	Pier Crit. Tensile Strength σ_{tcr} (psi)	$\frac{\sigma_{tcr}}{\sigma^o_{tcr}}$	$\frac{\tau_s}{\sqrt{f'_m}}$	$\frac{\tau_u}{\sqrt{f'_m}}$
CBRC-21-2	0.38	--	3315	284	295	-477	272	477	264	0.93	5.12	4.72
-3	0.38	0.08	3315	284	264	-458	252	458	228	0.80	4.59	4.38
-4	0.38	0.12	3315	284	267	-417	248	417	244	0.86	4.64	4.31
-5	0.38	0.19	3315	284	262	-377	250	377	247	0.87	4.55	4.34
CBRC-11-1	--	--	2507	205	247	-296	239	296	251	1.22	4.93	4.77
-2	0.13	--	2507	205	244	-193	221	193	282	1.38	4.87	4.41
-3	0.13	0.06	2507	205	239	-186	222	186	276	1.35	4.77	4.43
-4	0.13	0.28	2507	220	268	-276	259	276	287	1.30	5.35	5.17
-5	0.33	--	2507	205	217	-159	213	159	256	1.25	4.33	4.25
-6	0.33	0.11	2507	220	272	-209	267	209	316	1.44	5.43	5.33
-7	0.33	0.39	2507	220	257	-169	241	169	310	1.41	5.13	4.81
CBRC-12-1	0.23	--	2876	269	253	-108	244	108	329	1.22	4.72	4.55
-2	0.23	0.11	2876	269	250	-127	239	127	316	1.17	4.66	4.46
-3	0.23	0.22	2876	269	275	-138	267	150	349	1.30	5.13	4.98
-5	0.23	0.44	2876	269	231	-116	232	141	293	1.09	4.31	4.33
-6	0.23	0.75	2876	269	240	- 94	266	142	316	1.17	4.48	4.96

NOTE: (1) The prism strength is based on a h/d ratio of 5

TABLE 2.9

RATIOS OF TEST AND RECOMMENDED ULTIMATE SHEAR STRESS TO SQUARE ROOT OF PRISM COMPRESSIVE STRENGTH

			Range of Ratio of Average Ultimate Shear Stress τ_u to $\sqrt{f'_m}$ From Test Results			Recommended Ratio of the Ultimate Shear Stress to $\sqrt{f'_m}$ for Each Material		
							Reinforced	
Material	M/Vd Ratio	Height to Width Ratio	Jamb Steel Only	Light Horizontal Reinforcement (<0.002)(1)	Heavy Horizontal Reinforcement (>0.002)(1)	Partially Rein-forced	Masonry Takes the Shear	Reinforcement Takes the Shear
Hollow Concrete Block	1.0	2:1	2.08 - 3.06	--	4.00	1.5	2.0	3.0
	0.5	1:1	2.97 - 3.37	3.35 - 3.85	4.65 - 6.33	3.0	3.5	4.5
	0.25	1:2	5.67	6.03 - 7.28	6.29 - 7.56	-	-	-
	0	-	--	--	--	4.5	5.0	6.0
Hollow Concrete Brick	1.0	2:1	3.07	4.07 - 4.72	4.58 - 4.78	3.0	4.0	4.5
	0.5	1:1	4.24 - 5.30	5.50 - 6.46	5.28 - 6.29	4.0	5.0	5.5
	0.25	1:2	--	5.97	6.91 - 8.20	-	-	-
	0	-	--	--	--	5.0	6.0	6.5
Grouted Core Clay Brick	1.0	2:1	4.72	4.31 - 4.38		3.5	4.0	4.5
	0.5	1:1	4.25 - 4.41	4.43 - 5.33	4.81 - 5.17	-	-	-
	0.25	1:2	4.55	4.46	4.33 - 4.98	-	-	-
	0	-	-	--	--	3.5	4.0	4.5

NOTE: (1) This ratio is the area of horizontal steel to the gross vertical area of the pier.

TABLE 2.10

ULTIMATE STRENGTH REDUCTION FACTORS

	DUCTILITY FACTOR μ_{max}	STRENGTH REDUCTION FACTOR S_{μ}
JAMB STEEL ONLY	1	1.0
LIGHT HORIZONTAL REINFORCEMENT (< 0.002)	2	0.8
HEAVY HORIZONTAL REINFORCEMENT (> 0.002)	3	0.8
FLEXURAL FAILURE[(1)]	4	0.8

NOTE: (1) This assumed strength reduction factor is valid for this study only, since its value is a function of the dimensions, amount of reinforcement etc.

3. COMPARISON AND EVALUATION OF U.S. SEISMIC DESIGN PROVISIONS

3.1 Introduction

The seismic design loads and stresses for the two sets of design provisions have been summarized in Chapter 2; a "realistic" earthquake load was defined and recommended ultimate strengths presented. In Chapter 3 these values are compared and evaluated. First, in Section 3.2 a comparison is made between the design base shear forces for the two sets of code provisions; then these are compared with the base shear forces resulting from a "realistic" earthquake load. Section 3.3 provides a tabular comparison of the allowable shear stresses recommended by the codes with those recommended from experimental results, and Section 3.4 combines the results of Sections 3.2 and 3.3 to provide a comparison of the minimum required shear areas for seismic loads for the two sets of provisions. In Section 3.5 the Over-Design Ratios are determined for the two sets of provisions using the base shear of a building, and these then provide the basis for the evaluation of the design provisions presented in Section 3.6.

3.2 Comparison of Loads

3.2.1 ATC-3-06 Tentative Provisions

Equations 2.3 and 2.4 are the base shear equations used in ATC-3-06. For the purpose of this study we assume the soil properties to be unknown; it follows that S is 1.2.

Then, using

$$A_a = A_v = 0.4 \text{ for the zone of highest seismicity}$$

and

$$R = \begin{cases} 3.5 \text{ for reinforced masonry} \\ 1.25 \text{ for partially reinforced and unreinforced masonry,} \end{cases}$$

from Eq. 2.4 of Section 2.3.1 the seismic design coefficient is

$$C_s = 0.1646\ T^{-2/3} \leq 0.2857 \text{ for reinforced masonry,} \tag{3.1a}$$

$$C_s = 0.4608\ T^{-2/3} \leq 0.8000 \text{ for partially reinforced and unreinforced masonry.} \tag{3.1b}$$

Equation 3.1 is plotted in Fig. 3.1.

3.2.2 1979 Uniform Building Code

Equation 2.7 is the base shear equation used in the 1979 UBC. Thus, if $Z = 1.0$, $I = 1.0$, $K = 1.33$, $S = 1.5$ and $CS \nmid 0.14$, from Eq. 2.7b of Section 2.3.2 the seismic design coefficient is

$$C_s' = 0.1330\ T^{-1/2} \leq 0.1862 \tag{3.2}$$

for all masonry buildings. Equation 3.2 is plotted in Figure 3.1.

3.2.3 "Realistic" Earthquake

Equation 2.23, together with Eqs. 2.20, 2.16 and 2.24, gives the base shear for the "realistic" earthquake. For $A_v = 0.4$, from Eqs 2.20 and 2.16 of Section 2.5.1

$$C_\mu^{eq} = \frac{0.5854}{\mu T} \leq \frac{1}{\sqrt{2\mu-1}}. \tag{3.3}$$

Eq. 3.3 is plotted in Fig. 3.2 for different ductilities, μ.

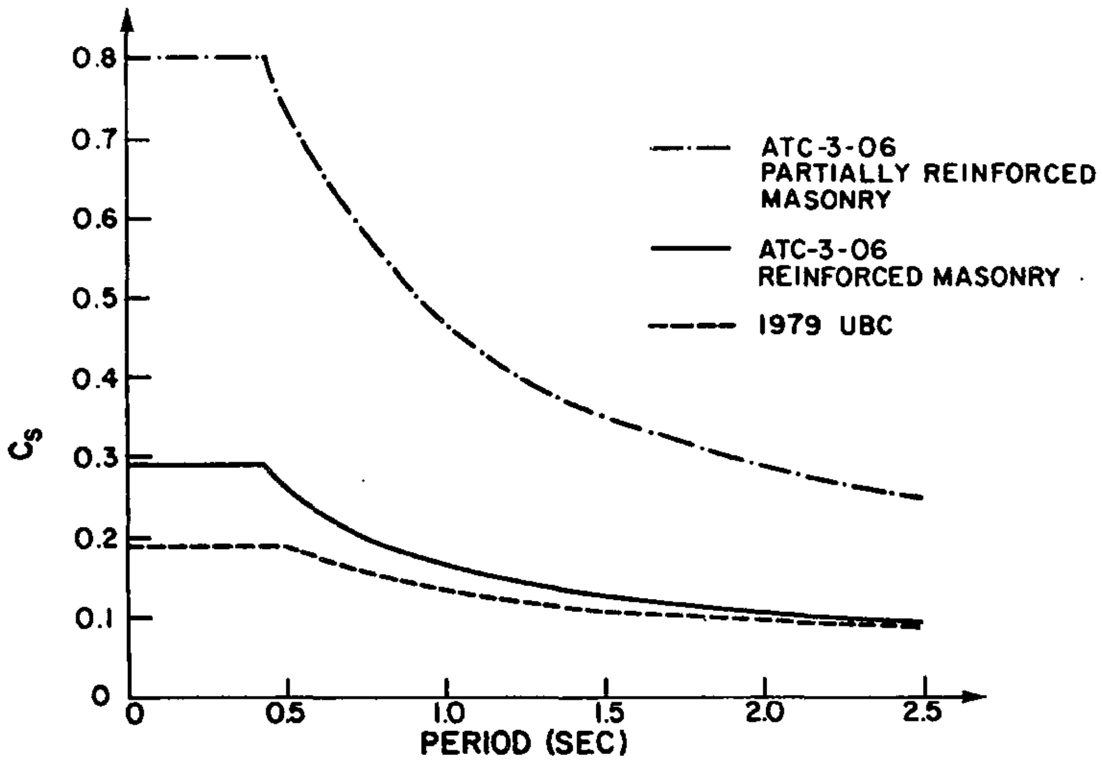

FIG. 3.1 EFFECTIVE BASE SHEAR COEFFICIENT FROM THE DESIGN PROVISIONS

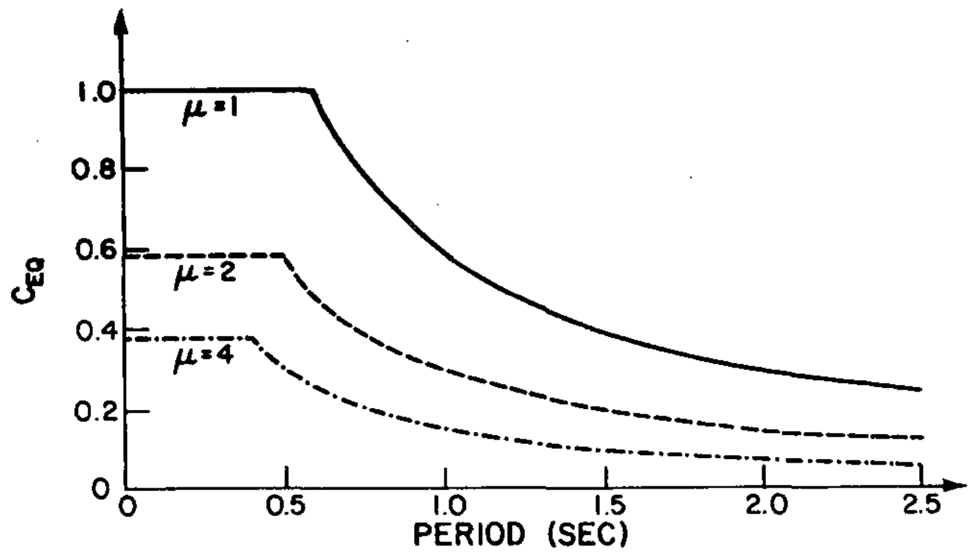

FIG. 3.2 ACCELERATION RESPONSE SPECTRA FOR MAP AREA 7 OF ATC-3-06
FOR DIFFERENT DUCTILITY FACTORS

3.2.4 Design Provisions vs. "Realistic" Earthquake Load

A direct comparison between the design provisions and the realistic earthquake loads can now be made. Ignoring the modal-participation factor, α, for the time being, we can evaluate the ratio

$$\frac{L_c}{L_{eq}} = \frac{C_s}{C_\mu^{eq}} \tag{3.4}$$

using Eqs. 3.1 and 3.3.

It follows that, for the ATC-3-06 Tentative Provisions, for reinforced masonry

$$\frac{C_s}{C_\mu^{eq}} = \frac{\begin{Bmatrix} 0.2857 & ; & T \leq 0.4373 \text{ sec.} \\ \dfrac{0.1646}{T^{2/3}} & ; & T > 0.4373 \text{ sec.} \end{Bmatrix}}{\begin{Bmatrix} \dfrac{1}{\sqrt{2\mu-1}} & ; & T \leq \dfrac{0.5854\sqrt{2\mu-1}}{\mu} \\ \dfrac{0.5854}{\mu T} & ; & T > \dfrac{0.5854\sqrt{2\mu-1}}{\mu} \end{Bmatrix}} \; ; \tag{3.5a}$$

and for partially reinforced and unreinforced masonry

$$\frac{C_s}{C_\mu^{eq}} = \frac{\begin{Bmatrix} 0.8000 & ; & T \leq 0.4373 \text{ sec.} \\ \dfrac{0.4608}{T^{2/3}} & ; & T > 0.4373 \text{ sec.} \end{Bmatrix}}{\begin{Bmatrix} \dfrac{1}{\sqrt{2\mu-1}} & ; & T \leq \dfrac{0.5854\sqrt{2\mu-1}}{\mu} \\ \dfrac{0.5854}{\mu T} & ; & T > \dfrac{0.5854\sqrt{2\mu-1}}{\mu} \end{Bmatrix}} \tag{3.5b}$$

Similarly, from Eqs. 3.2 and 3.3, for the 1979 UBC for all masonry buildings

$$\frac{C_s'}{C_\mu^{eq}} = \frac{\begin{Bmatrix} 0.1862 & ; & T \le 0.5102 \text{ sec.} \\ \dfrac{0.1330}{\sqrt{T}} & ; & T > 0.5102 \text{ sec.} \end{Bmatrix}}{\begin{Bmatrix} \dfrac{1}{\sqrt{2\mu-1}} & ; & T < \dfrac{0.5854\ \sqrt{2\mu-1}}{\mu} \\ \dfrac{0.5854}{T} & ; & T > \dfrac{0.5854\ \sqrt{2\mu-1}}{\mu} \end{Bmatrix}} \tag{3.6}$$

Equations 3.5a and 3.6 are plotted in Fig. 3.3 to represent the reinforced case for both design provisions, and Eqs. 3.5b and 3.6 are plotted together in Fig. 3.4 to represent the unreinforced and partially reinforced cases for both design provisions.

3.3 Comparison of Stresses

The allowable stresses, as defined by the two design provisions, are given in Tables 2.1 and 2.2 for ATC-3-06 and 1979 UBC, respectively. In most cases the allowable stresses are a function of f_m', but with an upper limit.

Estimates of the ultimate shear strengths of different materials derived from the Berkeley test results on single piers are given in Table 2.9. For the purpose of this study these ultimate strengths should be modified by the strength reduction factors dependent on the assumed ductility and amount of reinforcement, as given in Table 2.10. These strengths are directly proportional to $\sqrt{f_m'}$ with no upper limit.

The values from the Tables 2.1, 2.2 and 2.9 are used to evaluate the ratio R_{eq}/R_c of Eq. 2.2 as follows:

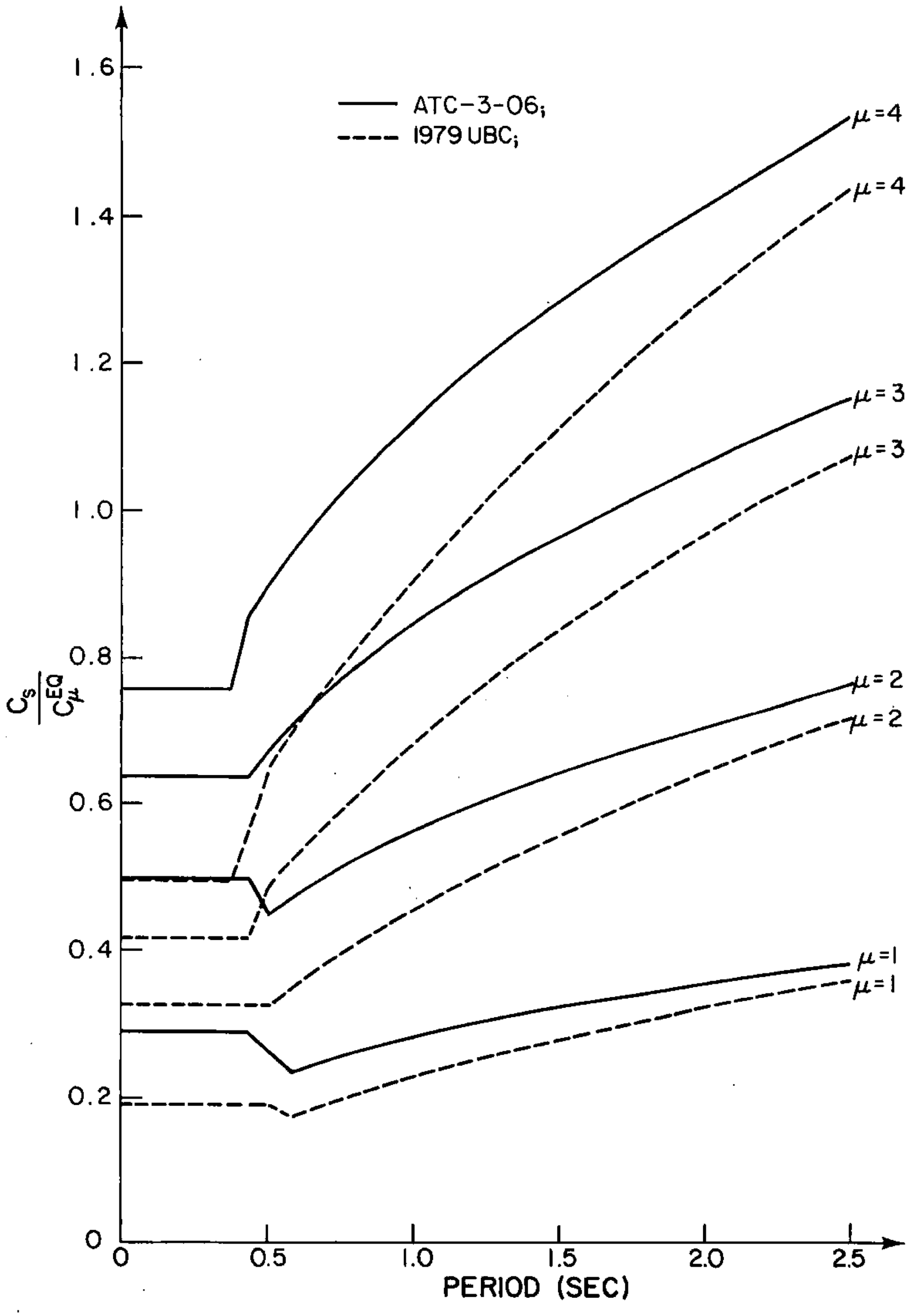

FIG. 3.3 RATIO OF DESIGN PROVISIONS LOAD TO SPECTRAL LOAD FOR DIFFERENT DUCTILITY FACTORS - REINFORCED MASONRY

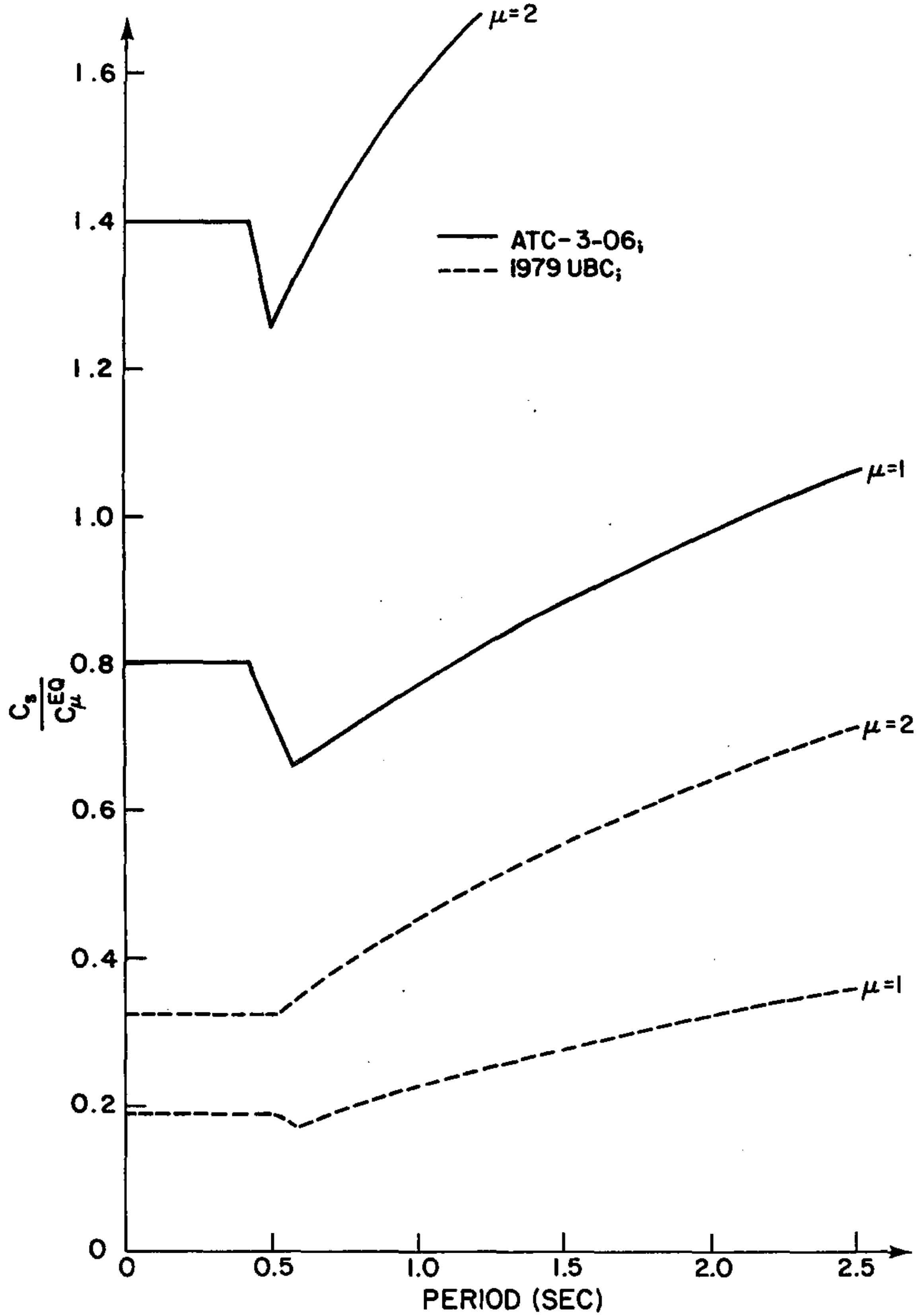

FIG. 3.4 RATIO OF DESIGN PROVISIONS LOAD TO SPECTRAL LOAD FOR DIFFERENT DUCTILITY FACTORS - PARTIALLY REINFORCED MASONRY

TABLE 3.1

COMPARISON OF ULTIMATE STRENGTH AND EFFECTIVE CODE ALLOWABLE SHEAR STRESSES: R_{eq}/R_c

Material	$\frac{M}{Vd}$ Ratio	ATC 3-06 AND 1979 UBC		ATC 3-06		1979 UBC	
				REINFORCED		REINFORCED	
		Partially Reinforced		Masonry Takes the Shear	Reinforcement Takes the Shear	Masonry Takes the Shear	Reinforcement Takes the Shear
HCBL	≥ 1	Hollow Unit	$0.125\sqrt{f'_m}$	$0.050\sqrt{f'_m}$ ≥ 2.222	$0.027\sqrt{f'_m}$ ≥ 1.333	$0.066\sqrt{f'_m}$ ≥ 2.500	$0.045\sqrt{f'_m}$ ≥ 2.256
	0	Hollow Unit	$0.375\sqrt{f'_m}$	$0.100\sqrt{f'_m}$ ≥ 2.500	$0.033\sqrt{f'_m}$ ≥ 2.000	$0.113\sqrt{f'_m}$ ≥ 2.809	$0.056\sqrt{f'_m}$ ≥ 3.371
HCBR	≥ 1	Hollow Unit	$0.250\sqrt{f'_m}$	$0.100\sqrt{f'_m}$ ≥ 4.444	$0.040\sqrt{f'_m}$ ≥ 2.000	$0.133\sqrt{f'_m}$ ≥ 5.000	$0.068\sqrt{f'_m}$ ≥ 3.384
	0	Hollow Unit	$0.417\sqrt{f'_m}$	$0.120\sqrt{f'_m}$ ≥ 3.000	$0.036\sqrt{f'_m}$ ≥ 2.167	$0.135\sqrt{f'_m}$ ≥ 3.371	$0.061\sqrt{f'_m}$ ≥ 3.652
CBRC	≥ 1	Grouted	$0.140\sqrt{f'_m}$	$0.100\sqrt{f'_m}$ ≥ 4.444	$0.040\sqrt{f'_m}$ ≥ 2.000	$0.133\sqrt{f'_m}$ ≥ 5.000	$0.068\sqrt{f'_m}$ ≥ 3.384
	0	Grouted	$0.140\sqrt{f'_m}$	$0.080\sqrt{f'_m}$ ≥ 2.000	$0.025\sqrt{f'_m}$ ≥ 1.500	$0.090\sqrt{f'_m}$ ≥ 2.247	$0.042\sqrt{f'_m}$ ≥ 2.528

For M/Vd Between 0 and 1 Interpolate by Straight Lines

i. For partially reinforced masonry the recommended values of Table 2.9 are used. These are divided by the unreinforced allowable stresses of Table 2.1 and 2.2. This is valid since the design provisions explicitly state that in general "partially reinforced masonry shall be designed as unreinforced masonry." (ATC-3-06: 12A, 3.7; 1979 UBC: 2419.(a)).

ii. For the case where masonry takes all the shear, we use corresponding values from Tables 2.1 and 2.2 for R_c, and the recommended ultimate values from Table 2.9 for R_{eq}.

iii. For the case where the reinforcement is assumed to take all the shear, the corresponding values from Tables 2.1 and 2.2 are used for R_c and for R_{eq} the recommended ultimate values from Table 2.9.

The results are presented in Table 3.1.

3.4 Comparison of Minimum Required Seismic Shear Areas

A comparison of the ATC-3-06 Tentative Provisions and the 1979 UBC will be made by comparing the area of shear wall that each of the design provisions requires, for both reinforced and partially reinforced masonry in the zone of highest seismicity. The required area is defined in Eq. 2.1 as

$$A_{required} = \frac{\text{Load}}{\text{Resistance}} \cdot$$

See Section 3.2 for a definition of the stresses used for reinforced and partially reinforced masonry.

3.4.1 ATC-3-06 Tentative Provisions

Using Eq. 2.1 with Eqs. 2.3 and 3.1 and Table 2.1, the minimum required area is

$$A^{min}_{required} = \frac{C_s W}{\text{Max. values from Table 2.1}} \tag{3.7}$$

where for reinforced masonry

$$C_s = 0.1646\ T^{-2/3} \leq 0.2757,$$

and for partially reinforced and unreinforced masonry

$$C_s = 0.4608\ T^{-2/3} \leq 0.8000.$$

Equation 3.7 is plotted in Figs. 3.5 (reinforcement takes all the shear) and 3.6 (masonry takes all the shear) for reinforced masonry and in Fig. 3.7 for partially reinforced masonry, where $A^{min}_{required}$ is given in terms of the weight of the building and is the minimum area required by ATC-3-06.

3.4.2 1979 Uniform Building Code

Using Eq. 2.1 with Eqs. 2.7a and 3.2 and Table 2.2,the minimum required area is

$$A^{min}_{required} = \frac{C'_s W}{\text{Max. values from Table 2.2}} \tag{3.8}$$

where for all masonry

$$C'_s = 0.1330\ T^{-1/2} \leq 0.1862 .$$

Equation 3.8 is plotted in Figs. 3.5 (reinforcement takes all the shear) and 3.6 (masonry takes all the shear) for reinforced masonry and in Fig. 3.7 for partially reinforced and unreinforced masonry in terms of the weight of the building.

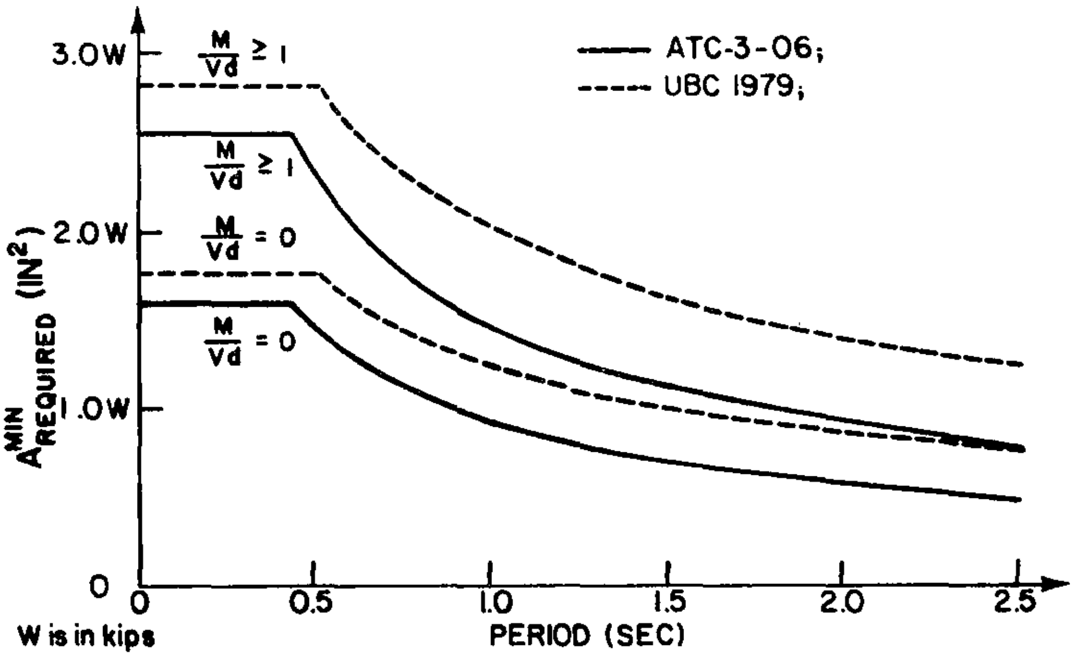

FIG. 3.5 MINIMUM REQUIRED AREA FOR REINFORCED MASONRY SHEAR WALLS IN THE ZONE OF HIGHEST SEISMICITY FOR DIFFERENT M/Vd RATIOS - REINFORCEMENT TAKES THE SHEAR

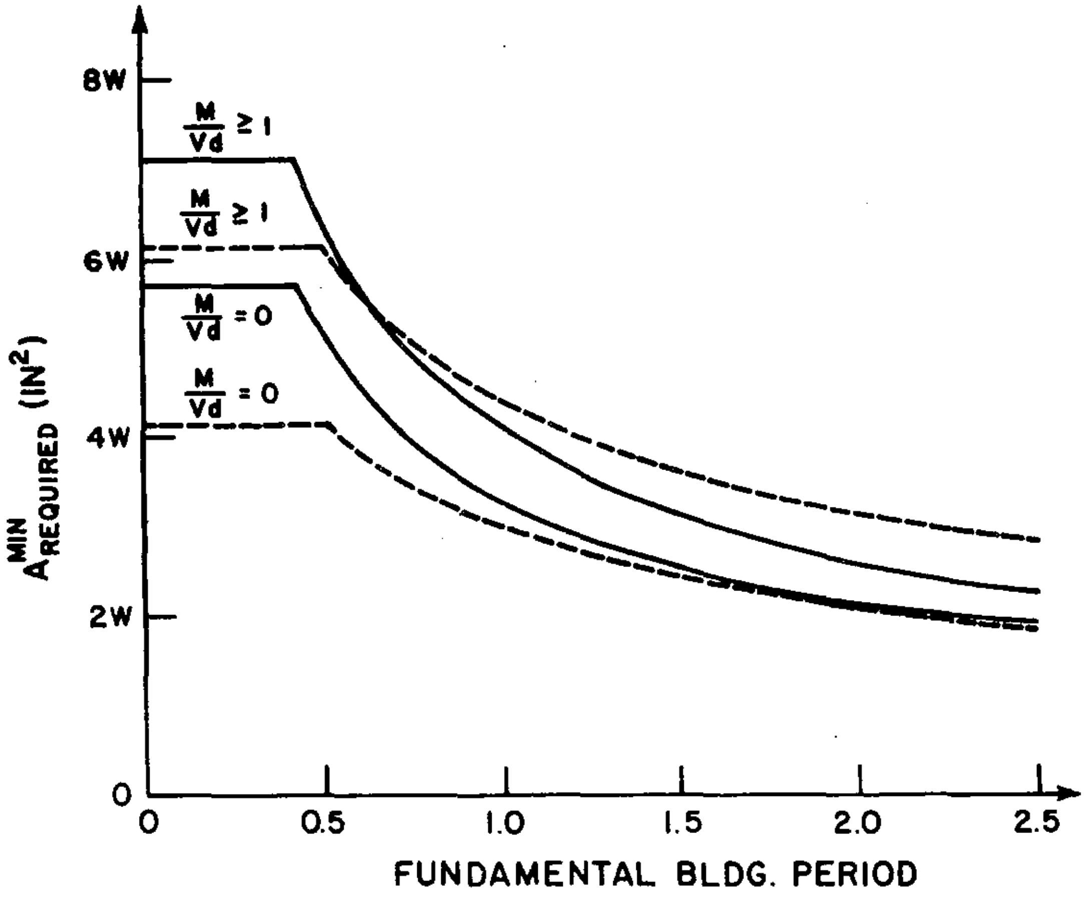

FIG. 3.6 MINIMUM REQUIRED AREA FOR REINFORCED MASONRY SHEAR WALLS IN THE ZONE OF HIGHEST SEISMICITY FOR DIFFERENT M/Vd RATIOS - MASONRY TAKES THE SHEAR

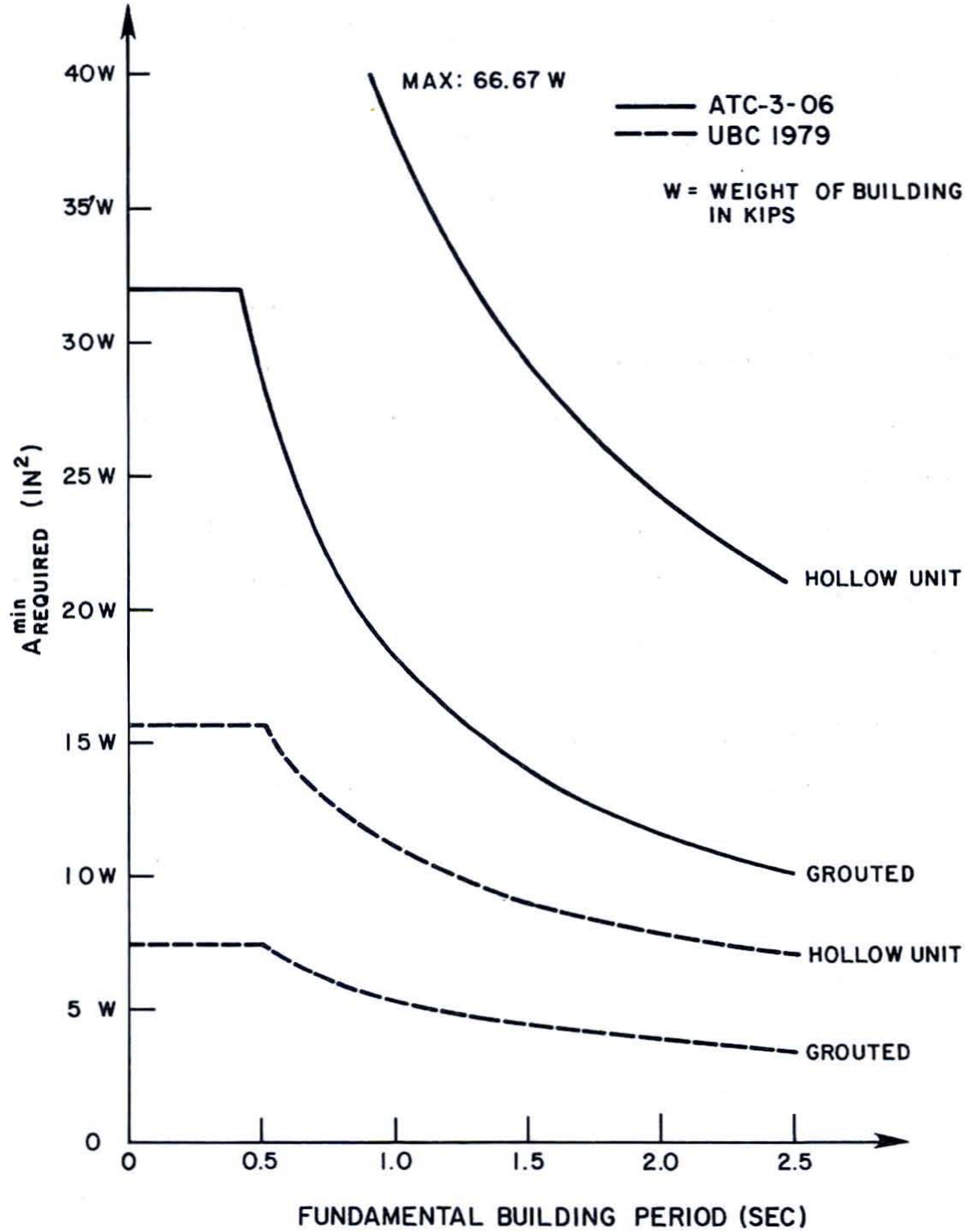

FIG. 3.7 MINIMUM REQUIRED AREA FOR PARTIALLY REINFORCED AND UNREINFORCED MASONRY SHEAR WALLS IN THE ZONE OF HIGHEST SEISMICITY

3.4.3 Discussion

It is clear, from Figs. 3.5, 3.6 and 3.7, that the 1979 UBC is more conservative than ATC-3-06 for the case of reinforced masonry when reinforcement takes all the shear (Fig. 3.5); whereas ATC-3-06 is more conservative for stiff or low period buildings for the case of reinforced masonry when the masonry is assumed to take all the shear (Fig. 3.6).

For partially reinforced masonry (Fig. 3.7), which uses the shear values for unreinforced masonry, ATC-3-06 requires approximately 3 to 4 times more shear wall area than the 1979 UBC for the two cases considered. This is primarily due to the change in R-factor from 3.5 to 1.25 in the ATC-3-06 Tentative Provisions for partially reinforced masonry. For the 1979 UBC there is no change in the design force level for partially reinforced masonry.

3.5 Over-Design Ratio for the Design Provisions

The Over-Design-Ratio, as defined in Eq. 2.2, is

$$ODR = \frac{L_c}{L_{eq}} \frac{R_{eq}}{R_c} \qquad (3.9)$$

The first factor

$$\frac{L_c}{L_{eq}} = \frac{C_s W}{C_\mu^{eq} \alpha W} = \frac{C_s}{\alpha C_\mu^{eq}} . \qquad (3.10)$$

This ratio is plotted for $\alpha = 1.00$ and the zone of highest seismicity of the two design provisions in Fig. 3.3 for reinforced masonry and Fig. 3.4 for partially reinforced masonry. The second factor, R_{eq}/R_c, is the resistance ratio which is given in Table 3.1 as a function of

f'_m and M/Vd. Then, if S_u is a strength reduction factor used when inelastic deformation is assumed the ODR for the zone of highest seismicity can be written as

$$ODR = S_\alpha S_\mu \frac{C_s}{C_\mu^{eq}} \frac{R_{eq}}{R_c} \tag{3.11}$$

where

$S_\alpha = \frac{1}{\alpha}$; α = modal-participation factor (see Appendix A and Eq. 2.24)

S_μ is a strength reduction factor given in Table 2.10

$\frac{C_s}{C_\mu^{eq}}$ is obtained from Fig. 3.3 or Fig. 3.4

$\frac{R_{eq}}{R_c}$ is given in Table 3.1.

The ODR for each set of design provisions for the zone of highest seismicity is plotted in Figs. 3.8 through 3.13; for the cases when reinforcement takes all the shear and masonry takes all the shear, and for three different types of fully grouted construction - hollow concrete block, hollow clay brick and grouted core clay brick.

In Tables 3.2 and 3.3 the ODR values corresponding to the zero period are listed for various ductilities, materials and the three cases, partially reinforced, reinforced where the masonry takes the shear and reinforced where the reinforcement takes the shear.

3.5.1 Generalization of the Over-Design Ratio

It is possible to express the ODR in such a way that it is valid for any seismic zone. To do this the following points must be considered.

i. Equations 2.4 and 2.20 are functions of A_v (or A_a) which depends on the seismic zone of a building location.

ii. Equation 2.10 is a function of Z which depends on the seismic zone of a building location.

iii. In all the above equations the zone of highest seismicity is assumed to apply. Hence $A_a = A_v = 0.4$; $Z = 1.00$.

Accordingly, we introduce a scaling factor, S^c_{eq}, for the seismic zone and the final equation for the ODR becomes

$$ODR = S^c_{eq} \, S_\mu \, S_\alpha \, \frac{C_s}{C^{eq}_\mu} \, \frac{R_{eq}}{R_c} \tag{3.12}$$

where

$$S^c_{eq} = \begin{cases} 1.00 \text{ for ATC-3-06} \\ \dfrac{Z}{2.5\,A_a} \text{ for UBC 1979.} \end{cases}$$

S_μ = a strength reduction factor listed in Table 2.10.

$S_\alpha = \frac{1}{\alpha}$ (see Appendix A and Eq. 2.24).

$\frac{C_s}{C^{eq}_\mu}$ is obtained from Fig. 3.3 (Eq. 3.5a or Eq. 3.6) for reinforced masonry, or Fig. 3.4 (Eq. 3.5b or Eq. 3.6) partially reinforced masonry.

$\frac{R_{eq}}{R_c}$ is given in Table 3.1.

It is apparent that the plots in Figs. 3.8 through 3.13 are only affected by the factor S^c_{eq} when different seismic zones are considered.

3.6 Discussion of the Over-Design Ratio

It is clear from Tables 3.2 and 3.3 and Figs. 3.8 through 3.13 that there are significant variations in the ODR for the various material types, M/Vd ratios and amount of reinforcement. In Figs. 3.8 through 3.13, it is apparent that the ODR increases as the period increases. This is a reflection of the conservatism that is included in the design spectra of the provisions because of a number of reasons associated with the structural behavior of longer period buildings. The ATC-3-06 Commentary states the reasons as follows:

1. The fundamental period of a building increases with number of stories. Hence, the longer the T, the larger the likely number of stories and therefore the number of degrees of freedom; hence, the more likely that high ductility requirements can be concentrated in a few stories of the building, at least for some earthquakes.

2. The number of potential modes of failure increases, generally with T. If design spectra were proportional to response spectra for single-degree-of-freedom systems, the probability of failure would increase with T.

3. Instability of a building is more of a problem with increasing T.

The conservatism included in the design spectra at longer periods does not have any significant impact for load-bearing masonry shear wall buildings, because most masonry shear wall buildings will have a period of one second or less (see Table 4.2). Thus the following discussion is based on the ODR ratios for zero period given in Tables 3.2 and 3.3.

The allowable shear stresses for reinforced masonry in both sets of provisions do not provide any differentiation for different types of materials of construction, whereas for unreinforced masonry they do.

From Table 2.9, however, it is clear that there are significant differences in the ultimate shear stresses for the different types of materials. Thus, consideration should be given to this differentiation in the design provisions. The following discussion shows that adjusting the effective allowable shear stresses equally for all materials yields a conservative ODR, and that the amount of conservatism will be very significant for some materials.

3.6.1 Partially Reinforced Masonry

For the ATC-3-06 Tentative Provisions, the ODR for partially reinforced masonry varies from 3.5 to 10.4 for the recommended ductility factor of 1. This results primarily from the use for partially reinforced masonry of the allowable stresses and R-factor of 1.25 specified for unreinforced masonry. An increase in the effective allowable shear stresses or an increase in the R-factor above 1.25 for this type of construction would lower the ODR to a value closer to 1 and thus result in less conservatism.

For the 1979 UBC, the ODR for partially reinforced masonry varies from 0.81 to 2.42 for the recommended ductility factor of 1. This variation above and below 1 results from the use of an allowable shear stress for masonry which is independent of the M/Vd ratio. If this provision is not changed than the effective allowable shear stresses must be decreased so that the ODR is equal to or greater than 1. To achieve this for hollow concrete block a decrease of approximately 25% in the effective allowable shear stresses is required, and this would then result in a conservative ODR for an M/Vd ratio equal to 0. On the other hand, for hollow clay brick the effective allowable shear stress could be increased by 50 to 60% and the ODR would still be greater than 1.

Whereas for the grouted core clay brick the effective allowable shear stress must be decreased only by 12 to 15% for the ODR to be equal to or greater than 1.

3.6.2 Reinforced Masonry - Masonry Takes The Shear

For the ATC-3-06 Tentative Provisions and an M/Vd ratio equal to 0, the ODR varies from 0.79 to 1.19 (Table 3.2) for the three material types and the recommended ductility factor of 2. Currently there is no differentiation in the allowable stresses for different materials and therefore the effective allowable shear stress for an M/Vd ratio of 0 should be decreased by 25% if the ODR is to be approximately equal to 1 for all material types. The effect of this change would result in an ODR of 1.50 for hollow clay brick construction and 1.25 for hollow concrete block construction.

For M/Vd $\geq$ 1, the ODR varies from 0.88 to 1.76 (Table 3.2) for the three material types. Decreasing the effective allowable shear stress by 15% for this M/Vd ratio would result in an ODR of approximately 1 for hollow concrete block and 2 for clay brick construction.

For the 1979 UBC and an M/Vd ratio equal to 0, the ODR varies from 0.58 to 0.87 (Table 3.3) for the three material types and the recommended ductility factor of 2. This is clearly non-conservative and the effective allowable shear stress should be decreased by 70% for the ODR to be approximately equal to or greater than 1 for all material types.

For M/Vd $\geq$ 1 the ODR varies from 0.65 to 1.29 (Table 3.3). As for the ATC-3-06 Tentative Provisions, if the effective allowable shear stress is decreased so that the ODR is approximately equal to 1 for hollow concrete block the provision will result in a conservative ODR for clay brick, since there is no differentiation in allowable stresses for different materials. The decrease in effective allowable shear stress should be of the order of 50% for the ODR to be approximately equal to or greater than 1.

3.6.3 Reinforced Masonry - Reinforcement Takes the Shear

For the ATC-3-06 provisions and an M/Vd ratio equal to 0, the ODR varies from 0.77 to 1.11 (Table 3.2) for the three material types and the recommended ductility factor of 3. A decrease in the effective allowable shear stress of 30% for this M/Vd ratio would ensure that the ODR was approximately equal to or greater than 1 for all material types. However, the ODR for hollow clay brick would be equal to 1.44 and thus be conservative.

For M/Vd $\geq$ 1 the ODR varies from 0.68 to 1.02. A decrease in the effective allowable shear stress of 50% would make the ODR approximately equal to or greater than 1 for all material types.

For the 1979 UBC provisions and M/Vd equal to 0 the ODR varies from 0.84 to 1.22 (Table 3.3) for the three material types and the recommended ductility factor of 3. A decrease of 20% in the effective allowable shear stress would ensure that the ODR was approximately equal to or greater than 1 for all material types.

For M/Vd $\geq$ 1 the ODR varies from 0.75 to 1.13. In this case a decrease of 33-1/3% in the effective allowable shear stress would ensure that the ODR was approximately equal to or greater than 1 for all material types.

TABLE 3.2

OVER-DESIGN RATIO FOR ZERO PERIOD - ATC-3-06 - ALL SEISMIC ZONES

ODR FOR ZERO PERIOD ATC-3-06 - All Seismic Zones		$\frac{M}{Vd} = 0$			$\frac{M}{Vd} \leq 1$		
MATERIAL TYPE	TYPE OF REINFORCEMENT	DUCTILITY			DUCTILITY		
		$\mu = 1$	$\mu = 2$	$\mu = 3$	$\mu = 1$	$\mu = 2$	$\mu = 3$
HCBL	Partially Reinforced - Hollow Unit $f'_m = 1200$ psi	10.39	--	--	3.46	--	--
	Reinforced - Masonry Takes the Shear	0.71	0.99	--	0.63	0.88	--
	Reinforced - Reinforcement Takes the Shear	0.57	0.89	1.02	0.38	0.59	0.68
HCBR	Partially Reinforced - Hollow Unit $f'_m = 1200$ psi	11.56	--	--	6.93	--	--
	Reinforced - Masonry Takes the Shear	0.86	1.19	--	1.27	1.76	--
	Reinforced - Reinforcement Takes the Shear	0.62	0.96	1.11	0.57	0.89	1.02
CBRC	Partially Reinforced - Grouted $f'_m = 1200$ psi	3.88	--	--	3.88	--	--
	Reinforced - Masonry Takes the Shear	0.57	0.79	--	1.27	1.76	--
	Reinforced - Reinforcement Takes the Shear	0.43	0.67	0.77	0.57	0.89	1.02

TABLE [illegible]

OVER-DESIGN RATIO FOR ZERO PERIOD - 1979 UBC - ZONE OF HIGHEST SEISMICITY

ODR FOR ZERO PERIOD 1979 UBC - Highest Seismic Zone		$\frac{M}{Vd} = 0$			$\frac{M}{Vd} \leq 1$		
MATERIAL TYPE	TYPE OF REINFORCEMENT	DUCTILITY			DUCTILITY		
		$\mu = 1$	$\mu = 2$	$\mu = 3$	$\mu = 1$	$\mu = 2$	$\mu = 3$
HCBL	Partially Reinforced - Hollow Unit f'_m = 1200 psi	2.42	--	--	0.81	--	--
	Reinforced - Masonry Takes the Shear	0.52	0.72	--	0.47	0.65	--
	Reinforced - Reinforcement Takes the Shear	0.63	0.98	1.12	0.42	0.65	0.75
HCBR	Partially Reinforced - Hollow Unit f'_m = 1200 psi	2.69	--	--	1.61	--	--
	Reinforced - Masonry Takes the Shear	0.63	0.87	--	0.93	1.29	--
	Reinforced - Reinforcement Takes the Shear	0.68	1.06	1.22	0.63	0.98	1.13
CBRC	Partially Reinforced - Grouted f'_m = 1200 psi	0.90	--	--	0.90	--	--
	Reinforced - Masonry Takes the Shear	0.42	0.58	--	0.93	1.29	--
	Reinforced - Reinforcement Takes the Shear	0.47	0.73	0.84	0.63	0.98	1.13

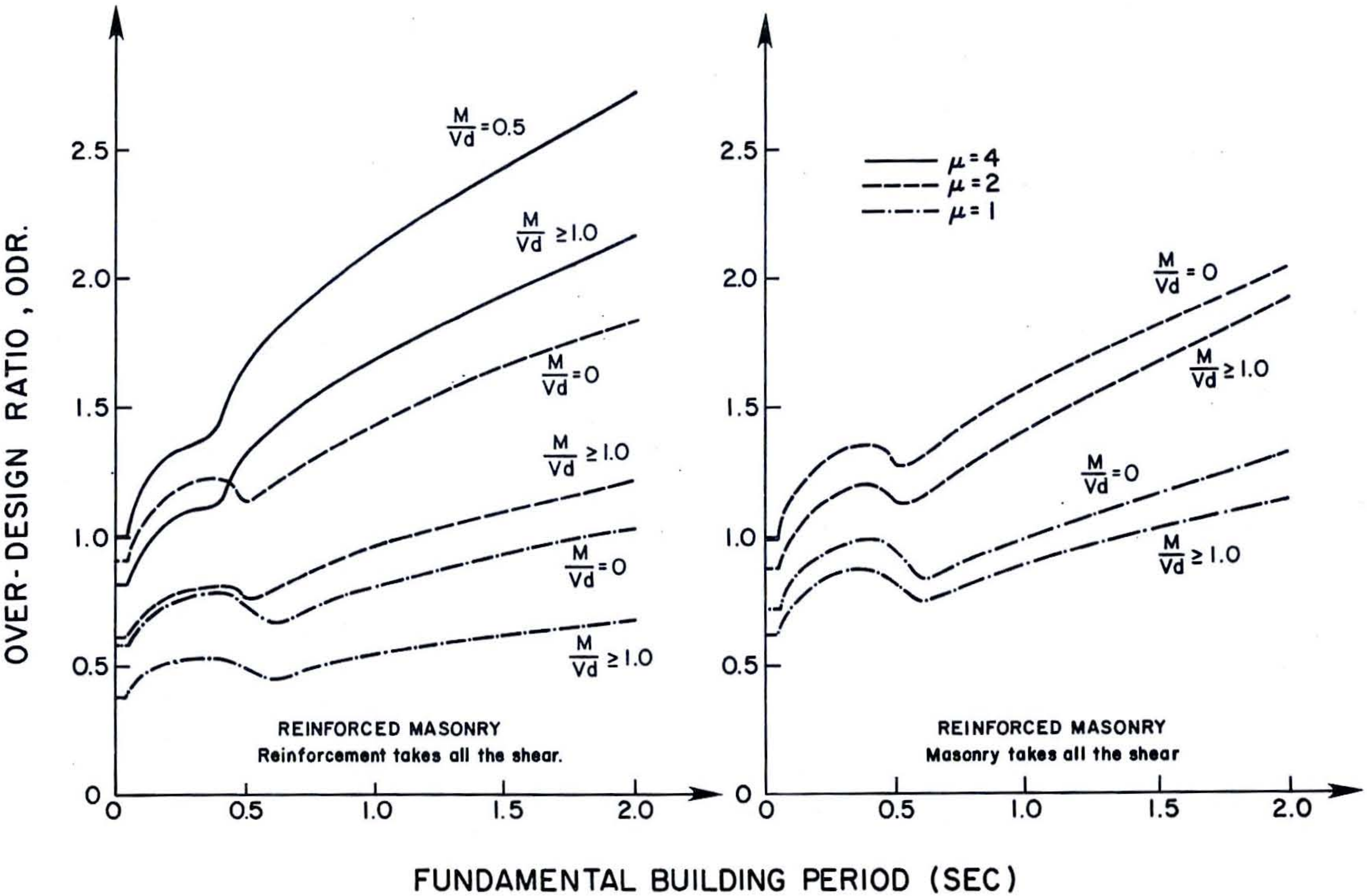

FIG. 3.8 THE OVER-DESIGN RATIO FOR ATC-3-06, HCBL (MINIMUM VALUES FROM TABLE 3.1)

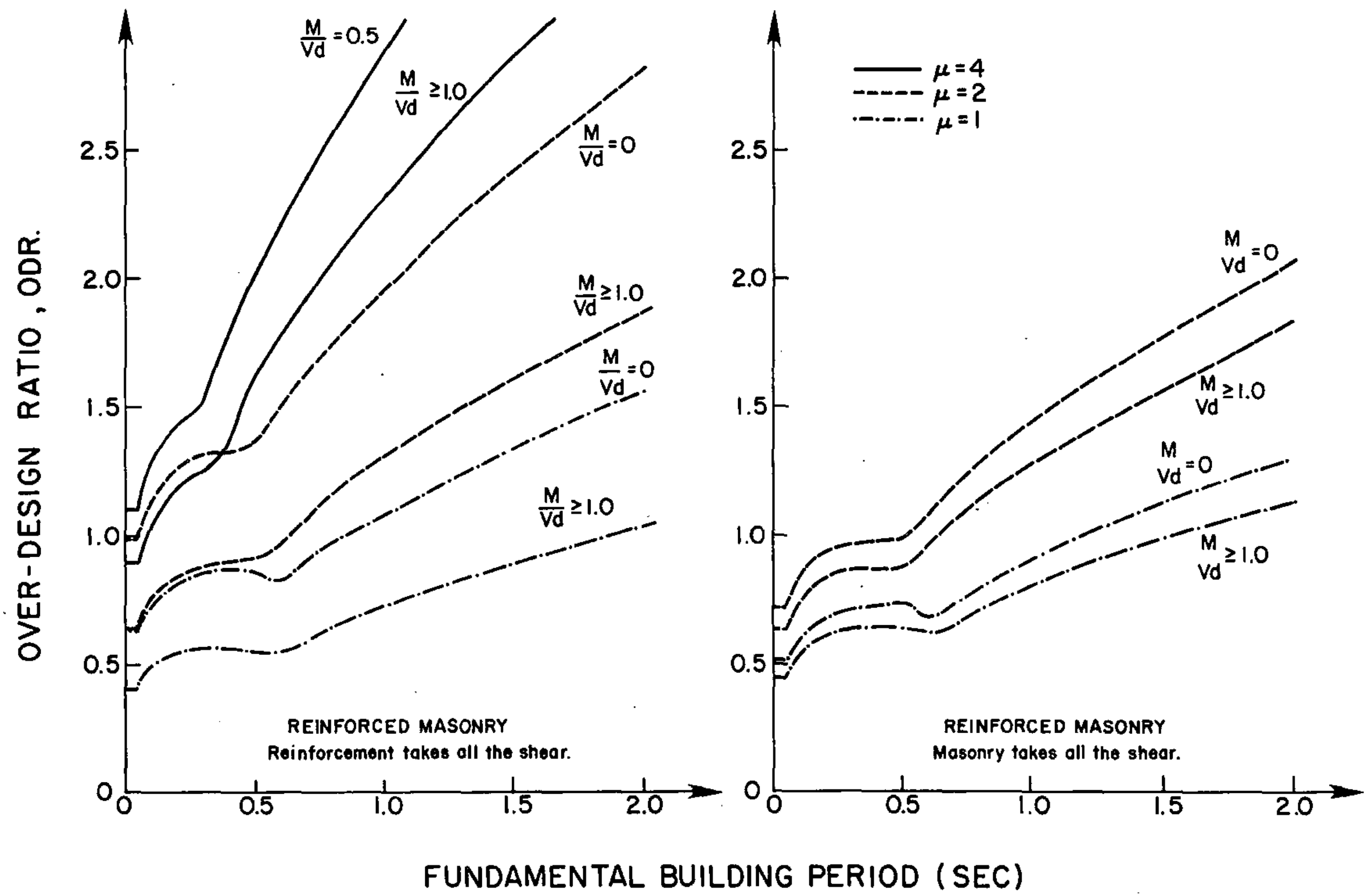

FIG. 3.9 THE OVER-DESIGN RATIO FOR 1979 UBC, HCBL (MINIMUM VALUES FROM TABLE 3.1)

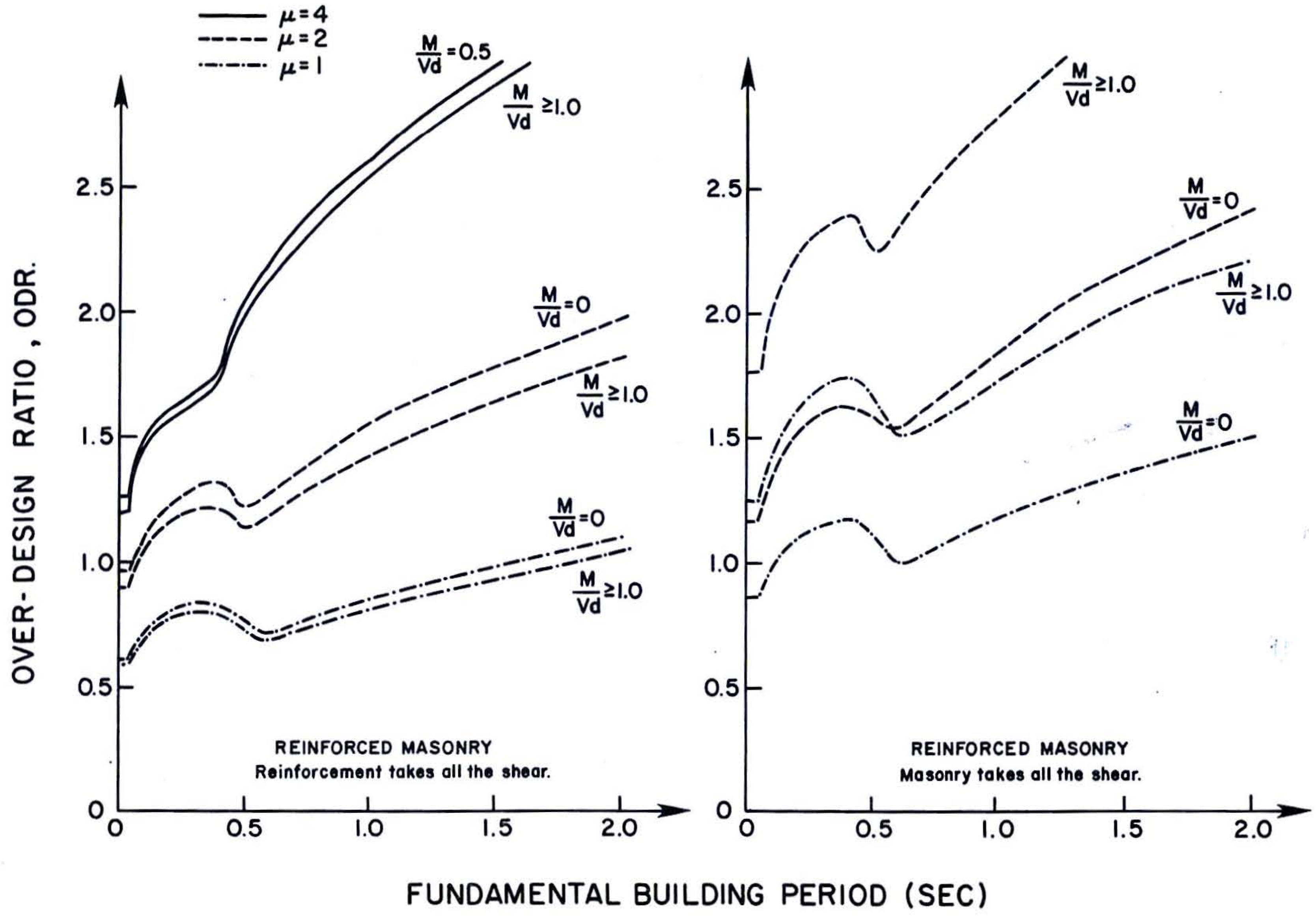

FIG. 3.10 THE OVER-DESIGN RATIO FOR ATC-3-06, HCBR (MINIMUM VALUES FROM TABLE 3.1)

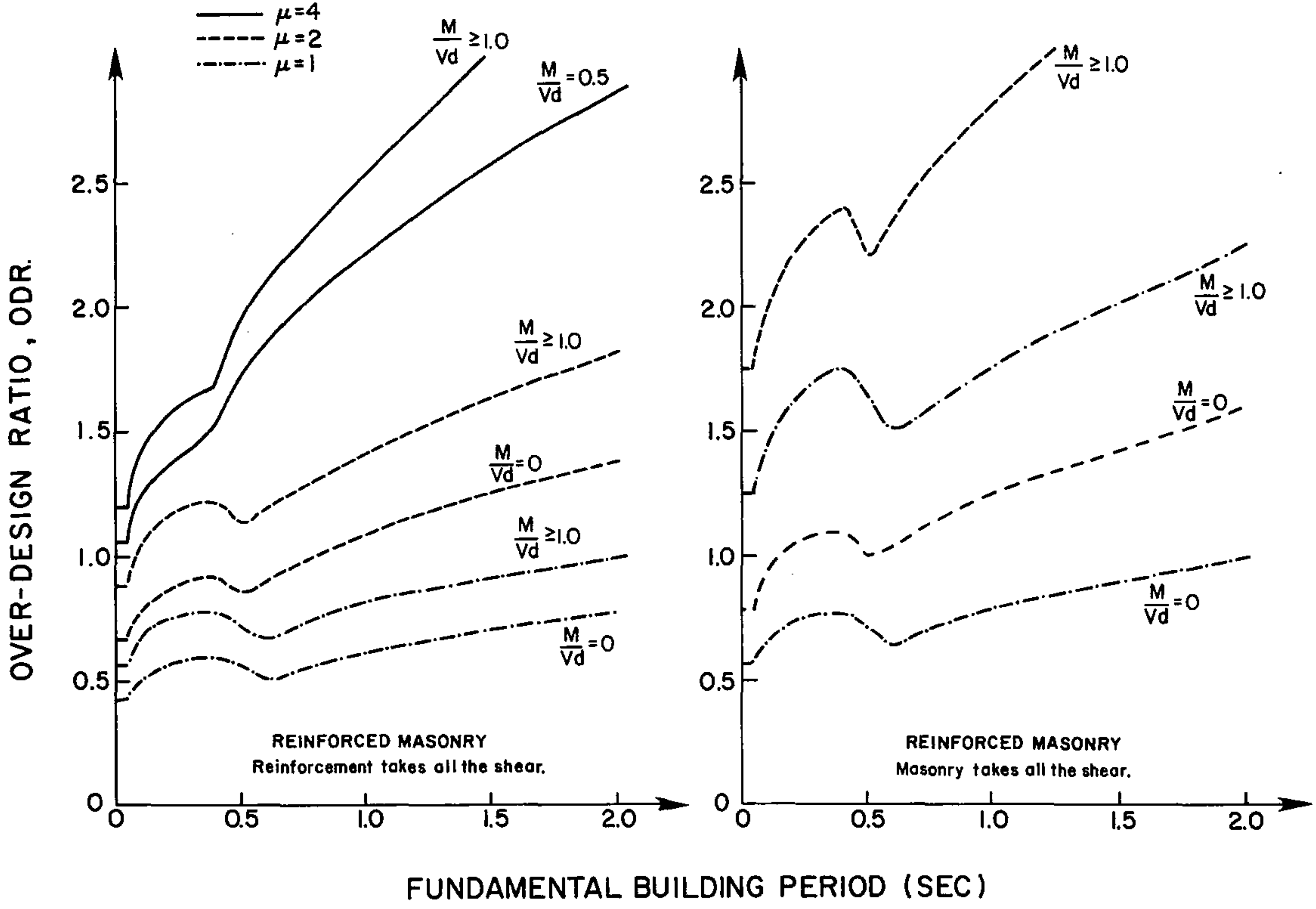

FIG. 3.11 THE OVER-DESIGN RATIO FOR 1979 UBC, HCBR (MINIMUM VALUES FROM TABLE 3.1)

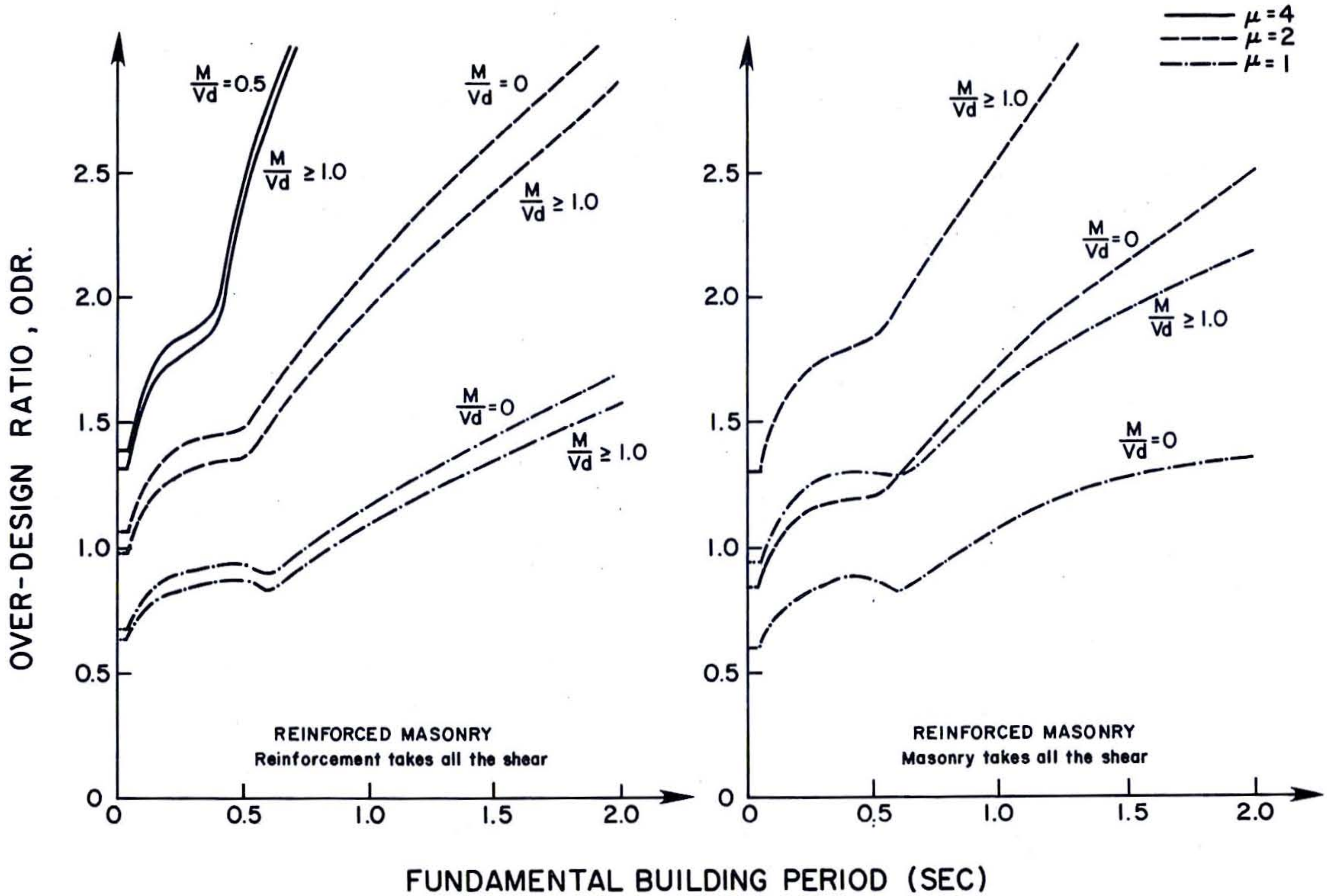

FIG. 3.12 THE OVER-DESIGN RATIO FOR ATC-3-06, CBRC (MINIMUM VALUES FROM TABLE 3.1)

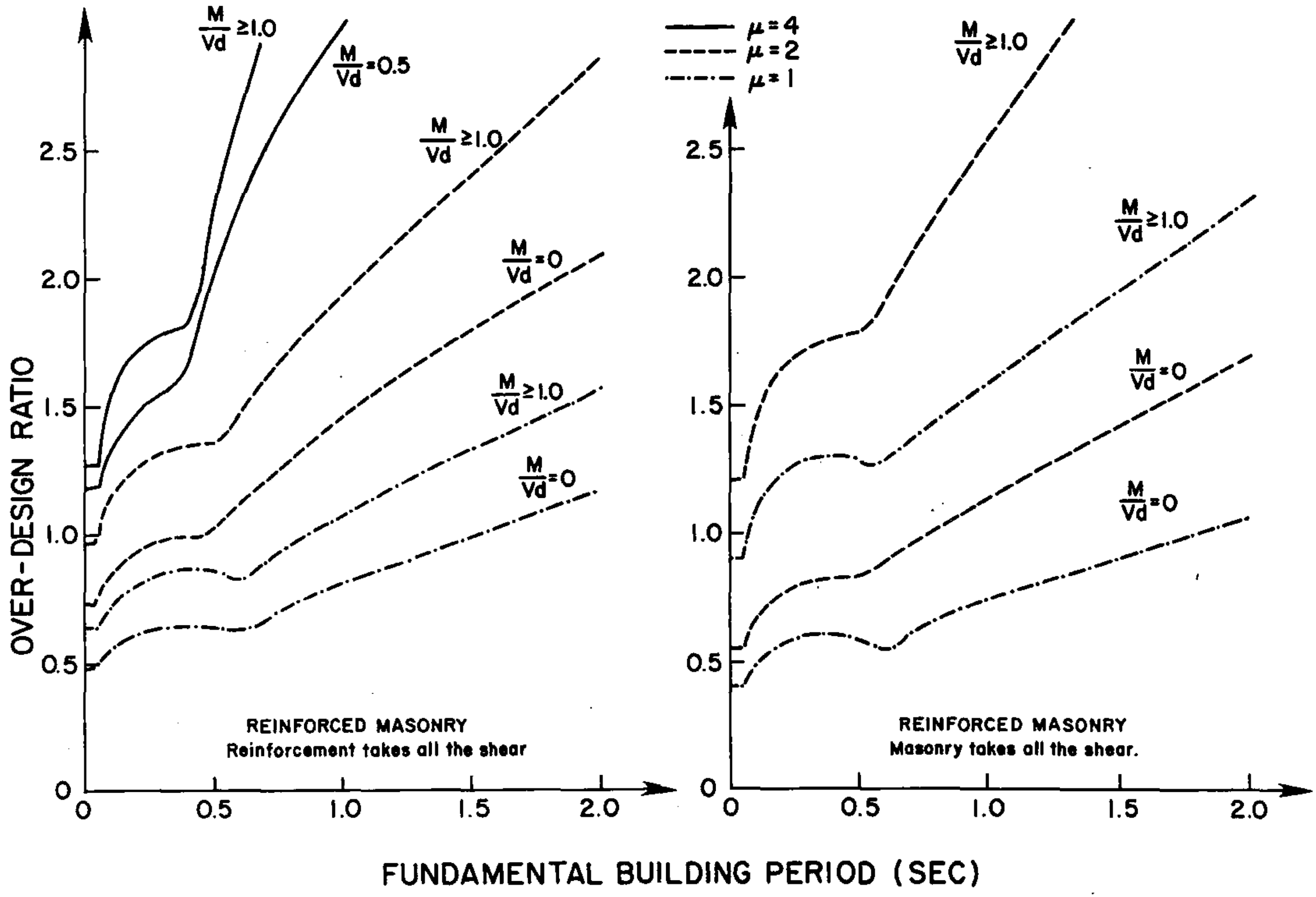

FIG. 3.13 THE OVER-DESIGN RATIO FOR 1979 UBC, CBRC (MINIMUM VALUES FROM TABLE 3.1)

4. EVALUATION OF THE OVER-DESIGN RATIO FOR 3, 9 and 17-STORY BUILDINGS

4.1 Introduction

Chapter 3 presented the Over-Design Ratios for the 1979 UBC and for the ATC-3-06 provisions using the design base shear forces to evaluate the two sets of provisions. In this chapter the ODRs are calculated for three buildings with identical floor plans and varying heights; namely, 3, 9 and 17 stories. The structural details of the three buildings and their computed dynamic characteristics are given in Section 4.2. Section 4.3 presents the story shears and overturning moments of the three buildings when subjected to loads for each set of provisions and to the "realistic" earthquake load. Section 4.4 contains the results of the calculations of the ODRs and, finally, a discussion of the results is presented in Section 4.5.

4.2 Characteristics and Properties of the Buildings

4.2.1 Plan and Elevation of the Buildings

The general floor plan which is shown in Fig. 4.1, is the same for all buildings. The overall plan dimensions are 74 ft by 132 ft. The assumed wall thicknesses are given in Table 4.1. Typical elevation

TABLE 4.1
WALL THICKNESS

Building Type	Thickness		
	9 in.	11 in.	13 in.
3 Story	Fl. 1 - Fl. 3	--	--
9 Story	Fl. 5 - Fl. 9	Fl. 1 - Fl. 4	--
17 Story	Fl. 13 - Fl. 17	Fl. 7 - Fl. 12	Fl. 1 - Fl. 6
All Floor Heights are 9 ft 4 in. = 112 in.			

and section views are shown in Figs. 4.2 through 4.5 for the 9-story building. The shear wall arrangement is symmetric in both directions with walls varying in width from 10 ft to 31 ft.

4.2.2 Structural Modeling

The program ETABS, which was used to compute the dynamic characteristics of the buildings as well as the building responses to the various loads, is a three-dimensional dynamic and static analysis program for buildings written by Wilson, Hollings and Dovey [14].

In modeling the buildings for ETABS, the following simplifications were made:

1. Narrow shear walls (< 20 ft wide) are modeled by the "equivalent frame" or "deep column analogy" concept [7] which is described as follows:

   i. The center lines of the wall sections (except corner walls) and of all connecting beams form the equivalent frame.

   ii. The cross-sectional properties of the column sections in the equivalent frame are identical to those of the corresponding wall section in the real building.

   iii. The central portions of all model beams have the same cross-sectional properties as the connecting beams of the actual structure. The fictitious portion of the beams contained within the shear walls are modeled as a "rigid" link as shown in Fig. 4.6. To account for the beam-column joint flexibility, the "rigid" link is taken as five sixths of the real length.

2. Stiffness and rigidity of all members are based on uncracked sections.

3. The wider shear walls are represented by shear panel-elements connected to the columns of the equivalent frame or to dummy columns in accordance with the wall position. The shear panel-elements have both shear and flexural stiffness as described in reference 14.

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

5. Foundation supports are assumed to be rigid (fixed).

The model resulting from these simplifications was used for the computer analysis. It is shown in Figs. 4.7 through 4.11 for the 9-story building.

4.2.3 Dynamic Characteristics of the Buildings

The periods of the various modes used in the analysis of each of the three buildings are presented in Table 4.2. It is interesting to compare the code-calculated period and that computed by the dynamic analysis. For the 3-story building, the code-calculated values in both directions are greater than those computed from the dynamic analysis. This also occurs for the 9-story building in the Y-direction (short direction) of the building. In the X-direction (long direction) the code and dynamic analysis values are in reasonable agreement. For the 17-story building the code-calculated period is less than that computed from the dynamic analysis in the X-direction, and in the Y-direction this is reversed.

The number of modes used for each direction different for each building. For the 3-story building, all three modes for each direction were used in the spectral analysis. For the 9-story building four modes were used for each direction and for the 17-story building six modes were used. This was considered sufficient as the SRSS modal combination method is not significantly affected by the higher modes.

Because the buildings are symmetric both in mass and in geometry, and because, for simplicity, no accidental eccentricity is considered (as is required by the codes) all the mode shapes are uncoupled.

4.3 Detailed Results

The results presented in this section are for both design provisions and for the "realistic" earthquake assuming ductility values of 1, 2 and 4. Plots of the story shear forces, panel shear forces and overturning moments (OTM) for the design provisions only will be presented for each story level.

4.3.1 The Design Provisions

The story and panel shears and the overturning moments for the three buildings are determined for the zone of highest seismicity of both design provisions. For other zones the values can be obtained by scaling the results by the appropriate factor.

The design story shear forces and the OTM of the three buildings are calculated using the computed first mode periods and the weights of the buildings given in Table 4.2 with Eqs. 2.3, 3.1 and 2.6 for ATC-3-06 and with Eqs. 2.7, 3.2 and 2.11 for the 1979 UBC. The results are plotted in Figs. 4.12, 4.14 and 4.16 for the 3, 9 and 17-story buildings, respectively. The panel forces, calculated using the story shears in ETABS [14], are plotted in Figs. 4.13, 4.15 and 4.17 for the 3, 9 and 17-story buildings, respectively.

4.3.2 The "Realistic" Earthquake

Using the spectra of Fig. 3.2 for ductilities of 1, 2 and 4, and the zone of highest seismicity, a spectral analysis was performed on the buildings using ETABS [14]. The SRSS modal combination approach was used to obtain the story shear forces and the panel forces. The plots of these are presented in Figs. 4.18 and 4.19 for the 3-story building, Figs. 4.20 and 4.21 for the 9-story building, and in Figs. 4.22 and 4.23 for the 17-story building. It is evident that the forces resulting from the realistic earthquake are much higher than those predicted by the design provisions.

4.4 Over-Design Ratios for the Three Buildings

The ODRs for the three buildings are presented on a story-by-story basis in conjunction with a comparison of the code loads to the "realistic" earthquake loads in Figs. 4.24 through 4.28.

In Figs. 4.24 and 4.25 the ratios of the code loads and "realistic" earthquake loads are compared for ductility ratios of 1, 2 and 4 for the ATC-3-06 and 1979 UBC provisions, respectively. Also included in these plots are the ODRs for the cases in which masonry takes the shear and in which reinforcement takes the shear. The values of R_{eq}/R_c used to calculate the ODR are for hollow concrete block using a strength reduction factor of 0.8 and M/Vd = 0.

For the three buildings the ratio of code to "realistic" load is close to or greater than 1 when a ductility factor of 4 is used with the "realistic" earthquake load. When a ductility factor of 2 is used, the ratio is between 0.50 and 0.65 and decreases below 0.50 when a ductility factor of 1 is used. The values of the ODRs are close to those presented in Tables 3.2 and 3.3 for the 3 and 9-story buildings.

For the 17-story building the ODR is greater than 1 for a ductility ratio of 2. This reflects the conservatism in the code loads at longer periods exhibited in Figs. 3.8 through 3.13.

Figures 4.26, 4.27 and 4.28 present similar plots to those given in Figs. 4.24 and 4.25, except they are based on the shear forces in the specific walls W1, W2 and W3, respectively, and the allowable stress corresponding to the M/Vd ratios for a particular wall is used. Specific values of these plots of specific walls are tabulated in Tables 4.3 and 4.4 for allowable shear stresses corresponding to hollow concrete block ultimate strengths using $f'_m = 3{,}000$ psi.

The main conclusion from this analysis on a building by building and wall by wall basis is that the results are similar to those presented in Chapter 3 which were based on the base shear coefficient.

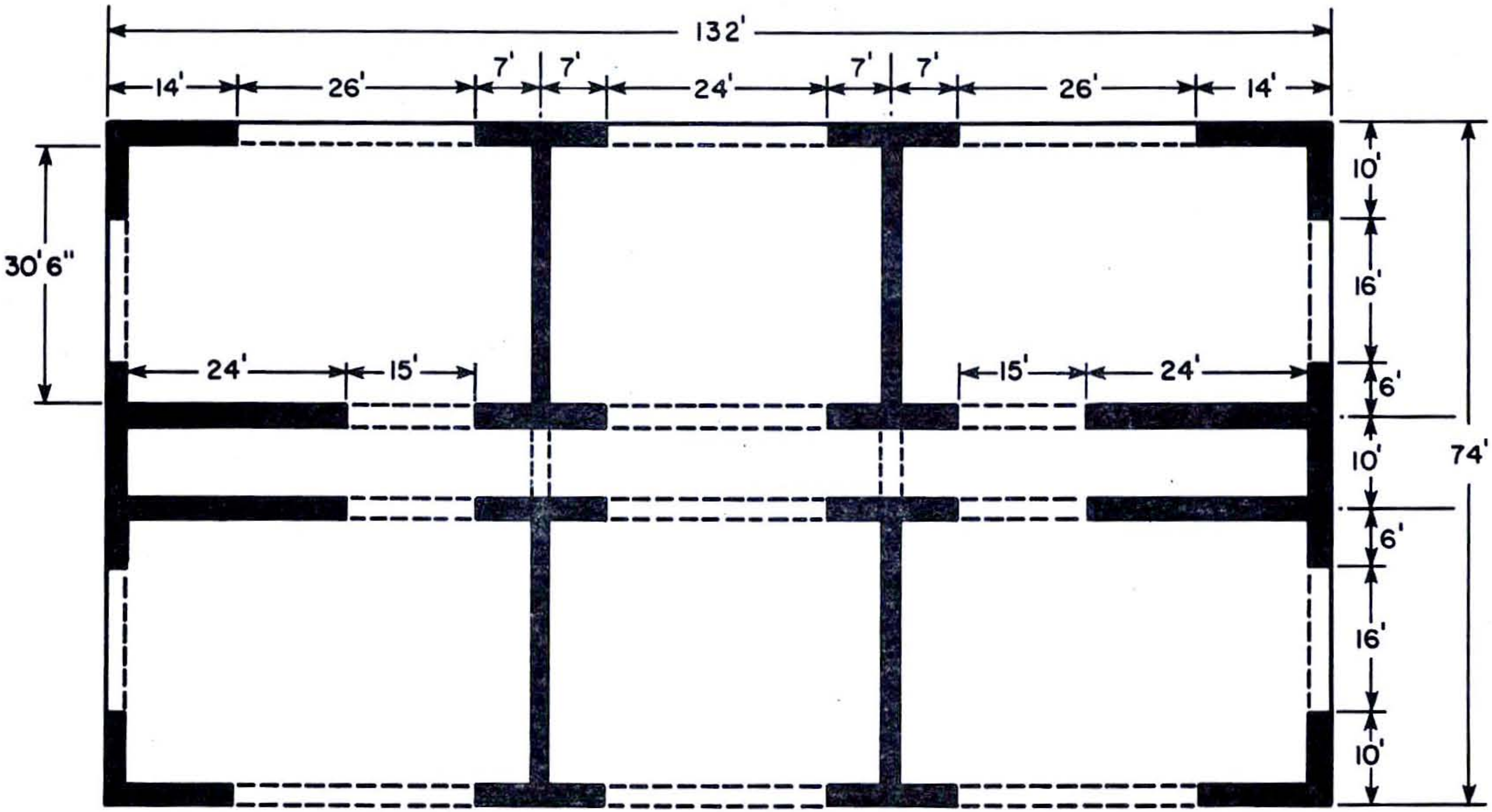

FIG. 4.1 FLOOR PLAN OF THE BUILDINGS

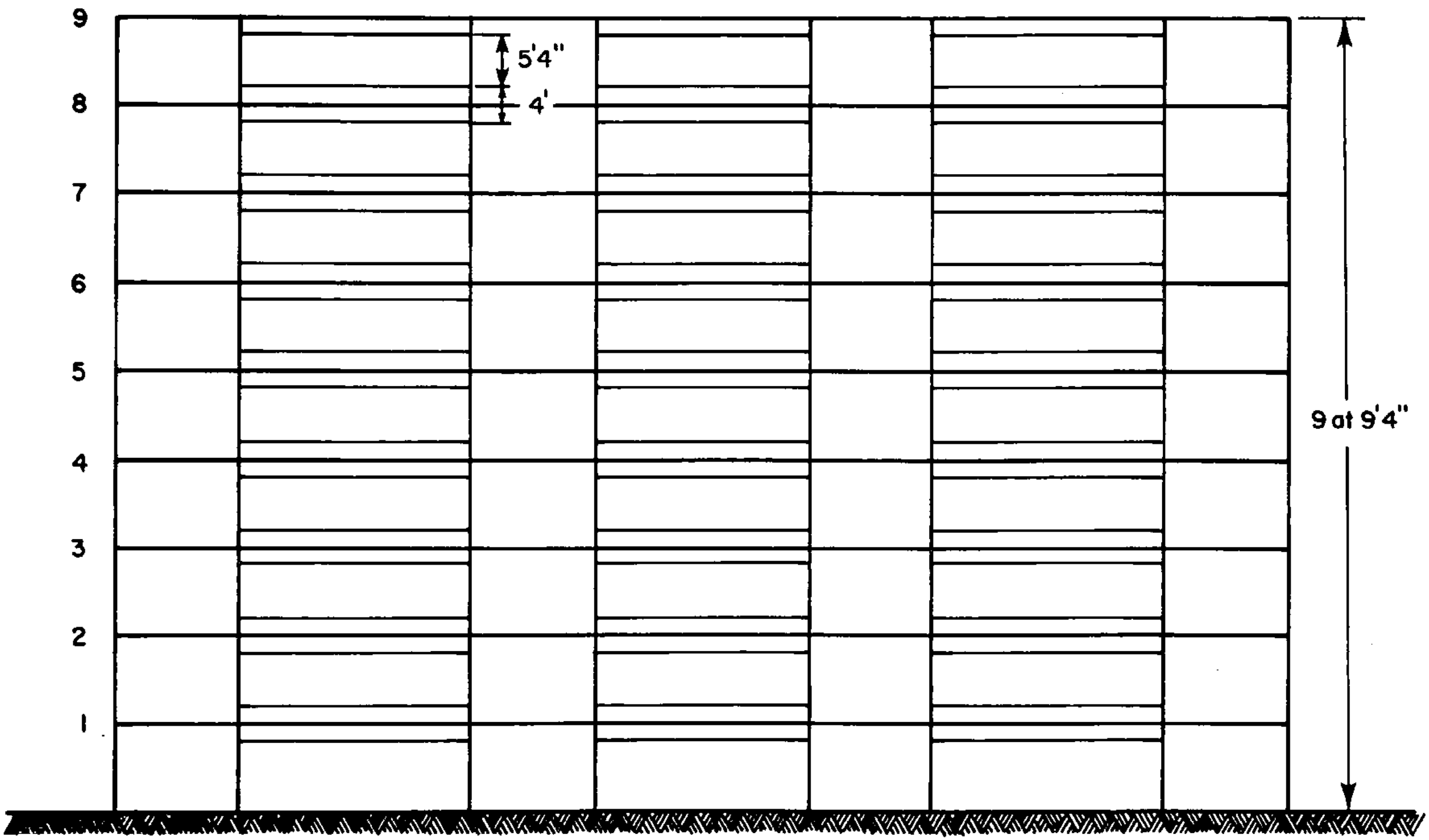

FIG. 4.2 EXTERIOR FRAME (LONGITUDINAL DIRECTION)

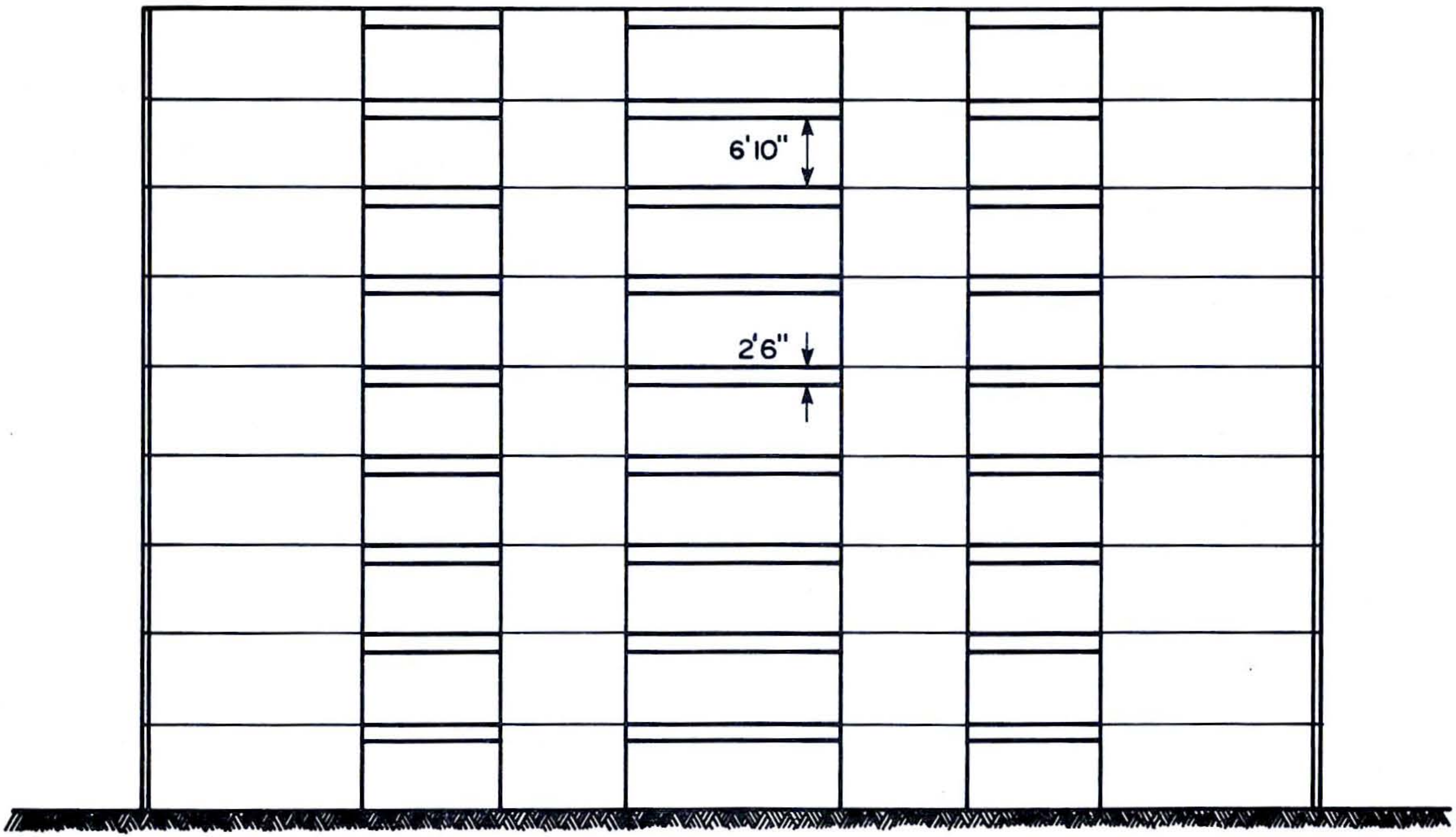

FIG. 4.3 INTERIOR FRAME (LONGITUDINAL DIRECTION)

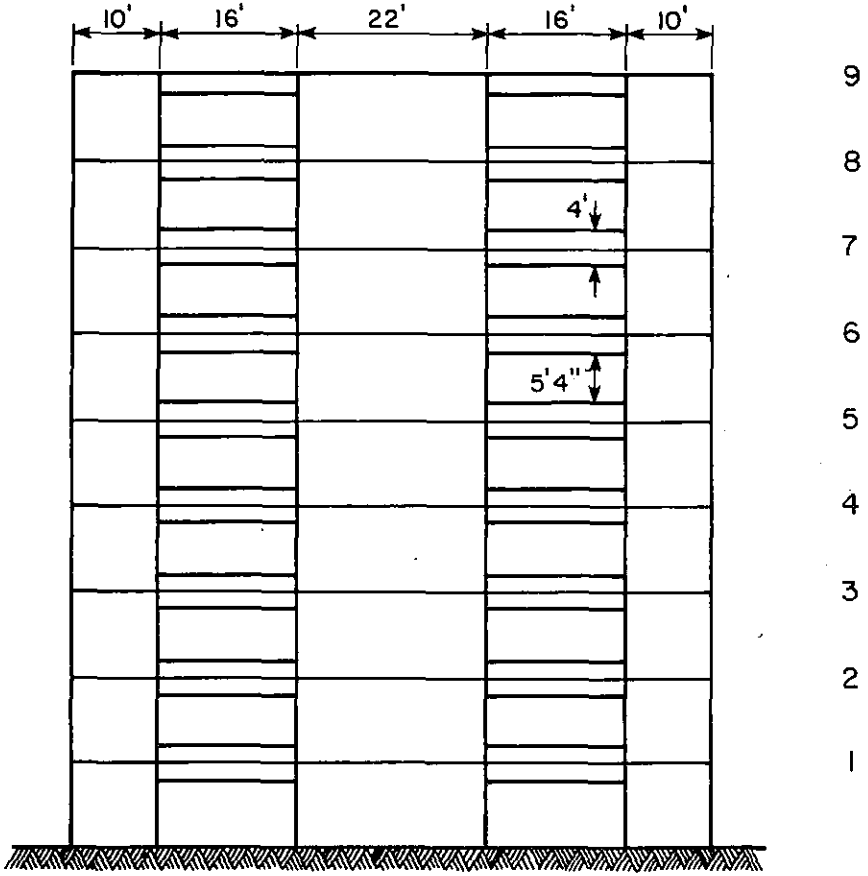

FIG. 4.4 EXTERIOR FRAME (TRANSVERSE DIRECTION)

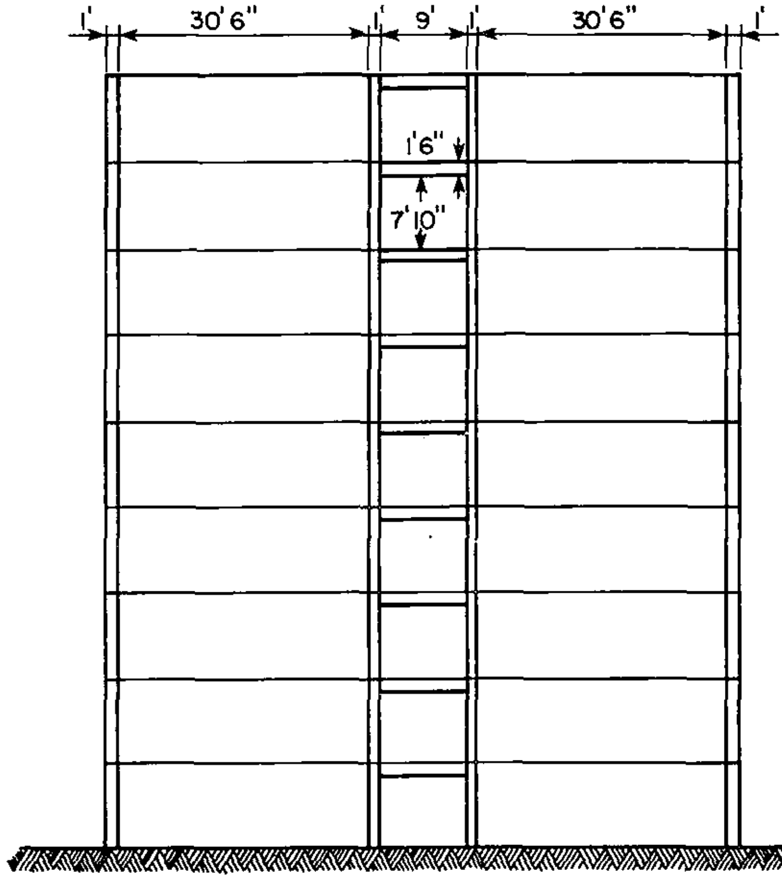

FIG. 4.5 INTERIOR FRAME (TRANSVERSE DIRECTION)

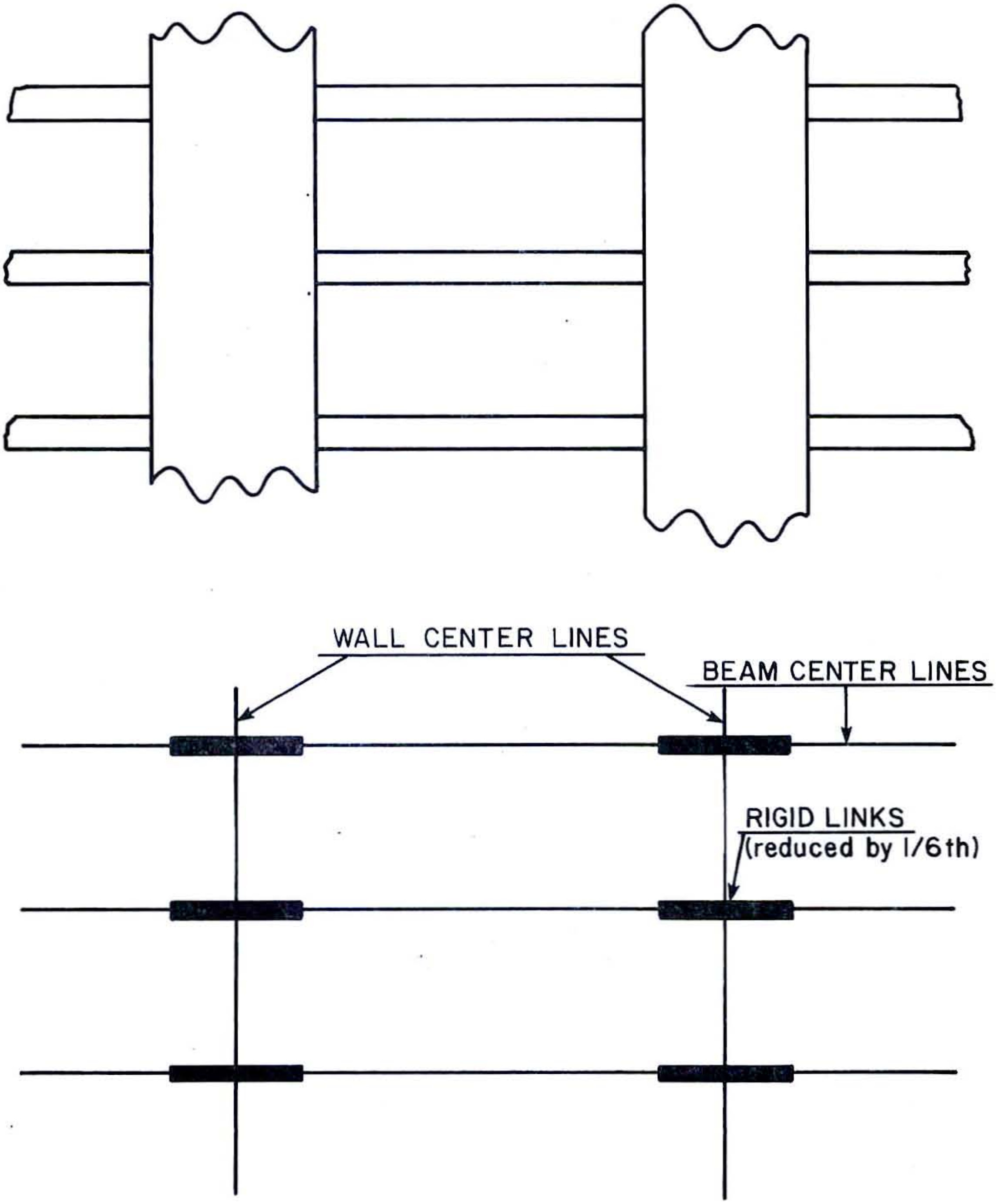

FIG. 4.6 RIGID BEAM LINK MODEL

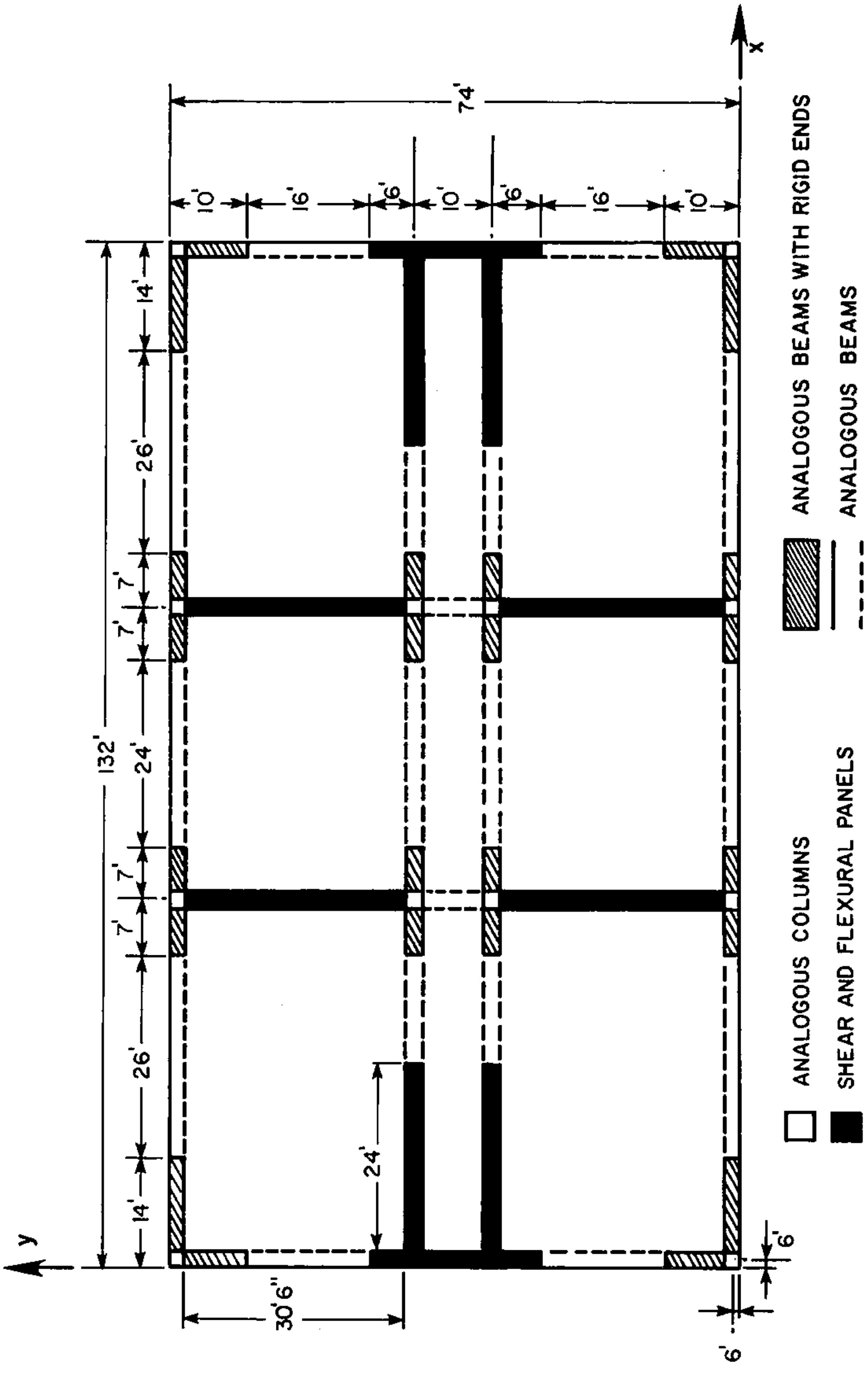

FIG. 4.7 MODEL - PLAN VIEW

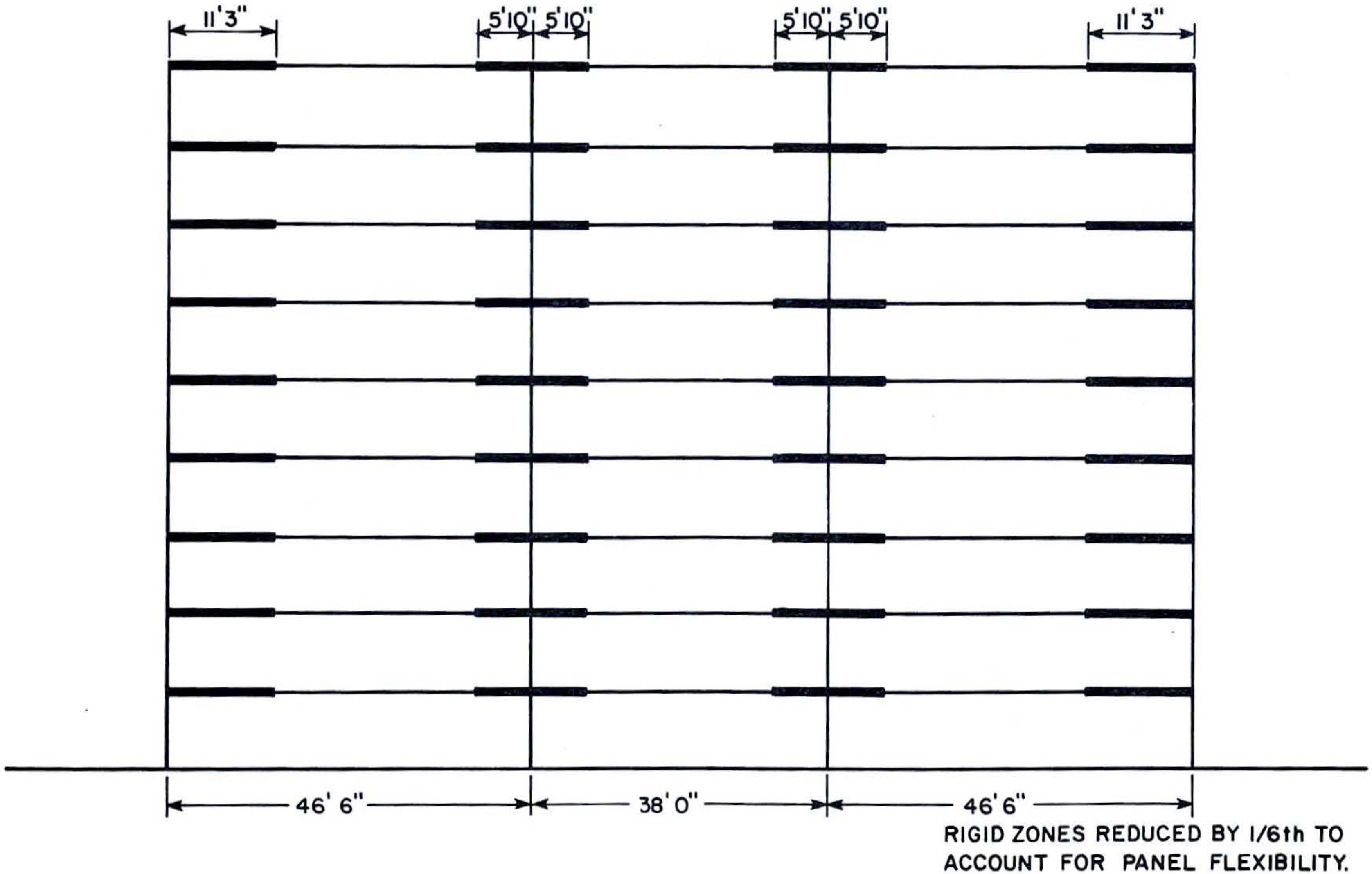

FIG. 4.8 EXTERIOR MODEL FRAME (LONGITUDINAL DIRECTION)

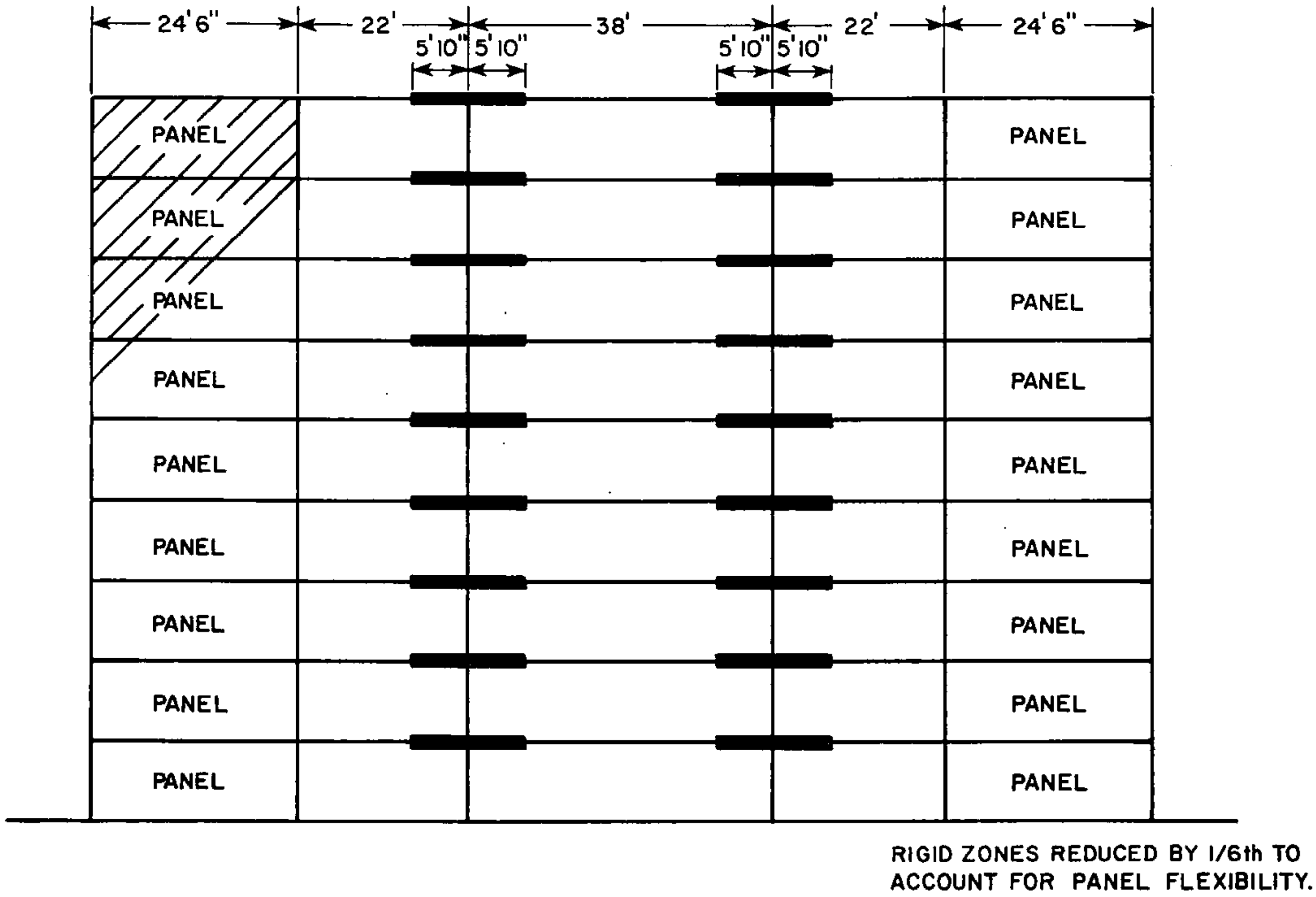

FIG. 4.9 INTERIOR MODEL FRAME (LONGITUDINAL DIRECTION)

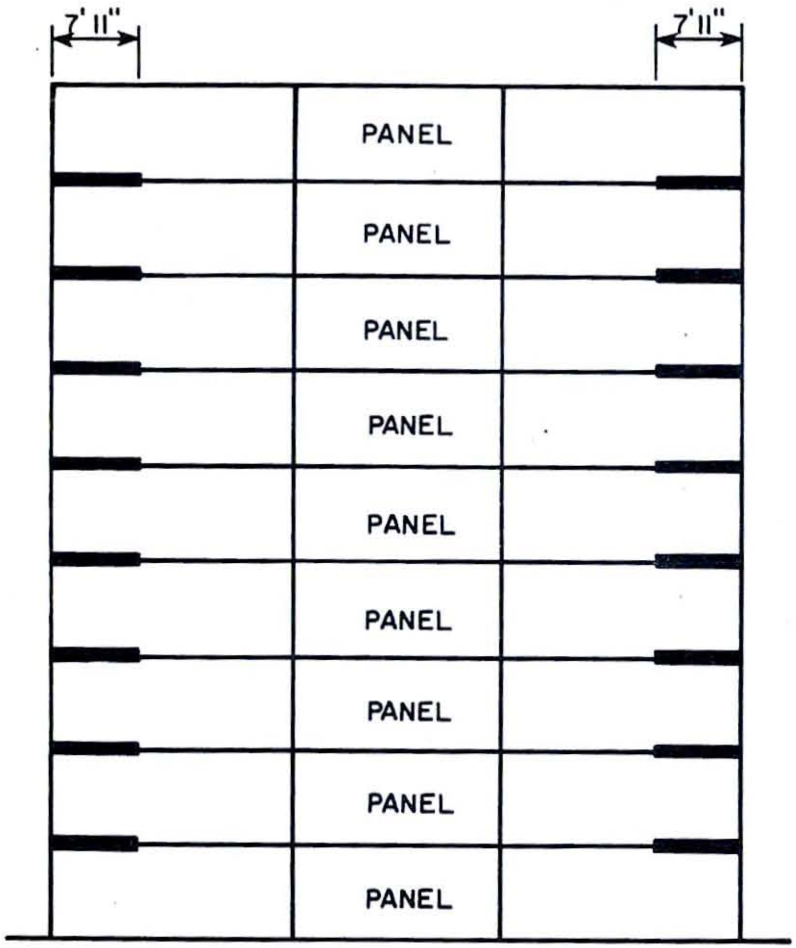

FIG. 4.10 EXTERIOR MODEL FRAME (TRANSVERSE DIRECTION)

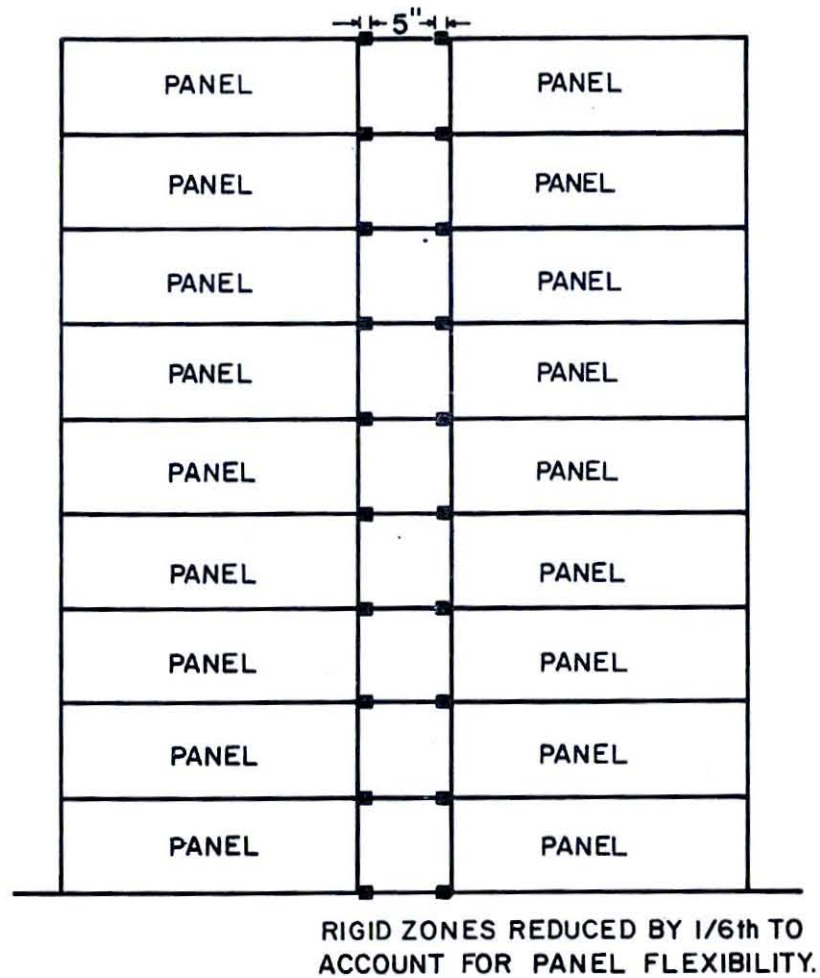

FIG. 4.11 INTERIOR MODEL FRAME (TRANSVERSE DIRECTION)

TABLE 4.2

BUILDING PERIODS OF VIBRATION

Building	Direction \ Period (sec)	From $\frac{0.05H}{\sqrt{D}}$	From ETABS 1st	2nd	3rd	4th	5th	6th	Weight of Building (kips)
3-Story	X - X	0.122	0.098	0.030	0.019	--	--	--	4300
	Y - Y	0.163	0.087	0.030	0.020	--	--	--	
9-Story	X - X	0.366	0.409	0.109	0.053	0.034	--	--	13900
	Y - Y	0.488	0.315	0.093	0.049	0.034	--	--	
17-Story	X - X	0.691	0.903	0.256	0.122	0.074	0.052	0.040	27600
	Y - Y	0.922	0.732	0.201	0.100	0.065	0.048	0.039	

X - X = Longitudinal

Y - Y = Transverse

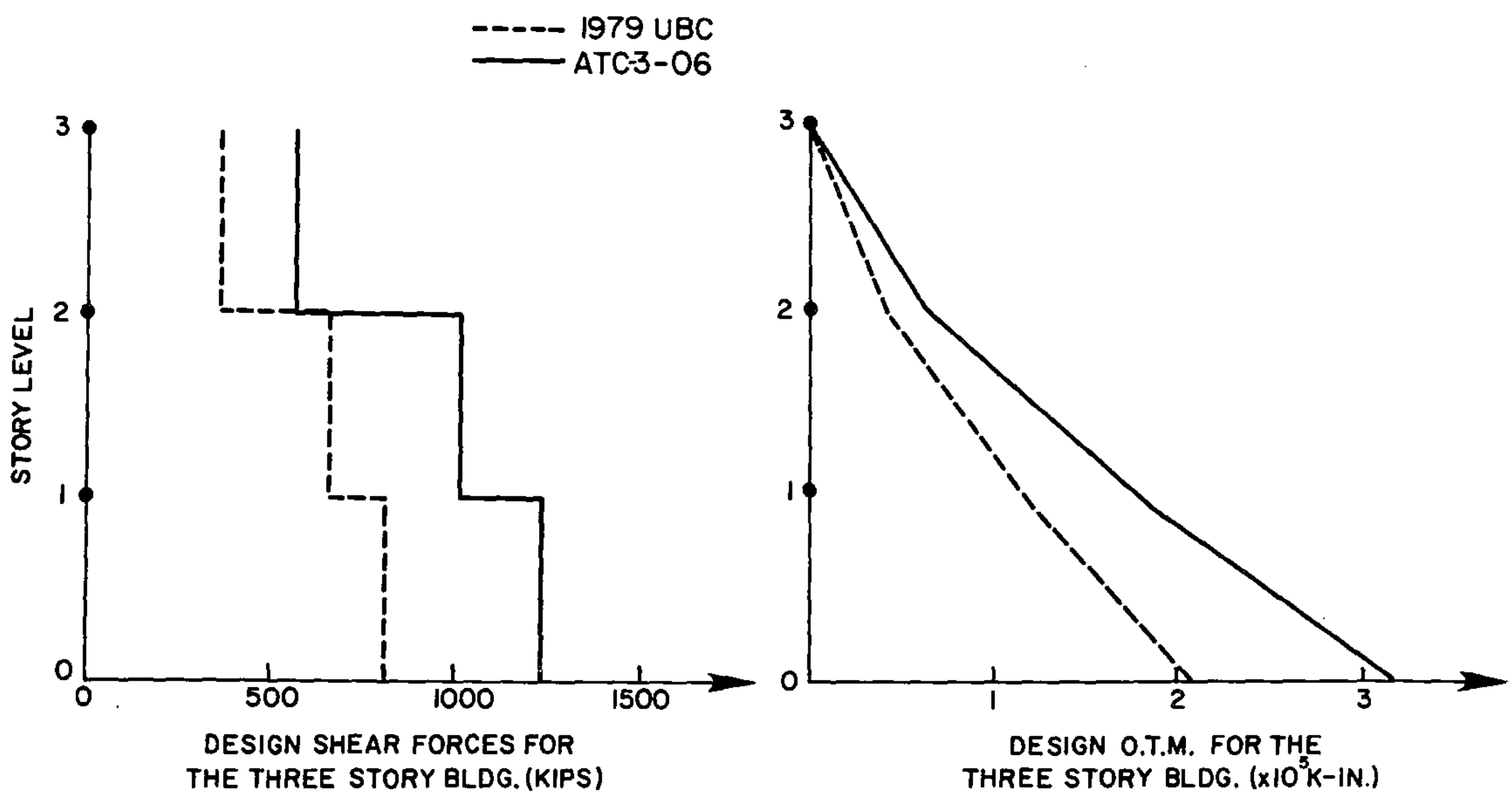

FIG. 4.12 CODE DESIGN SHEAR FORCES AND OVERTURNING MOMENTS FOR THE 3-STORY BUILDING - BOTH DIRECTIONS

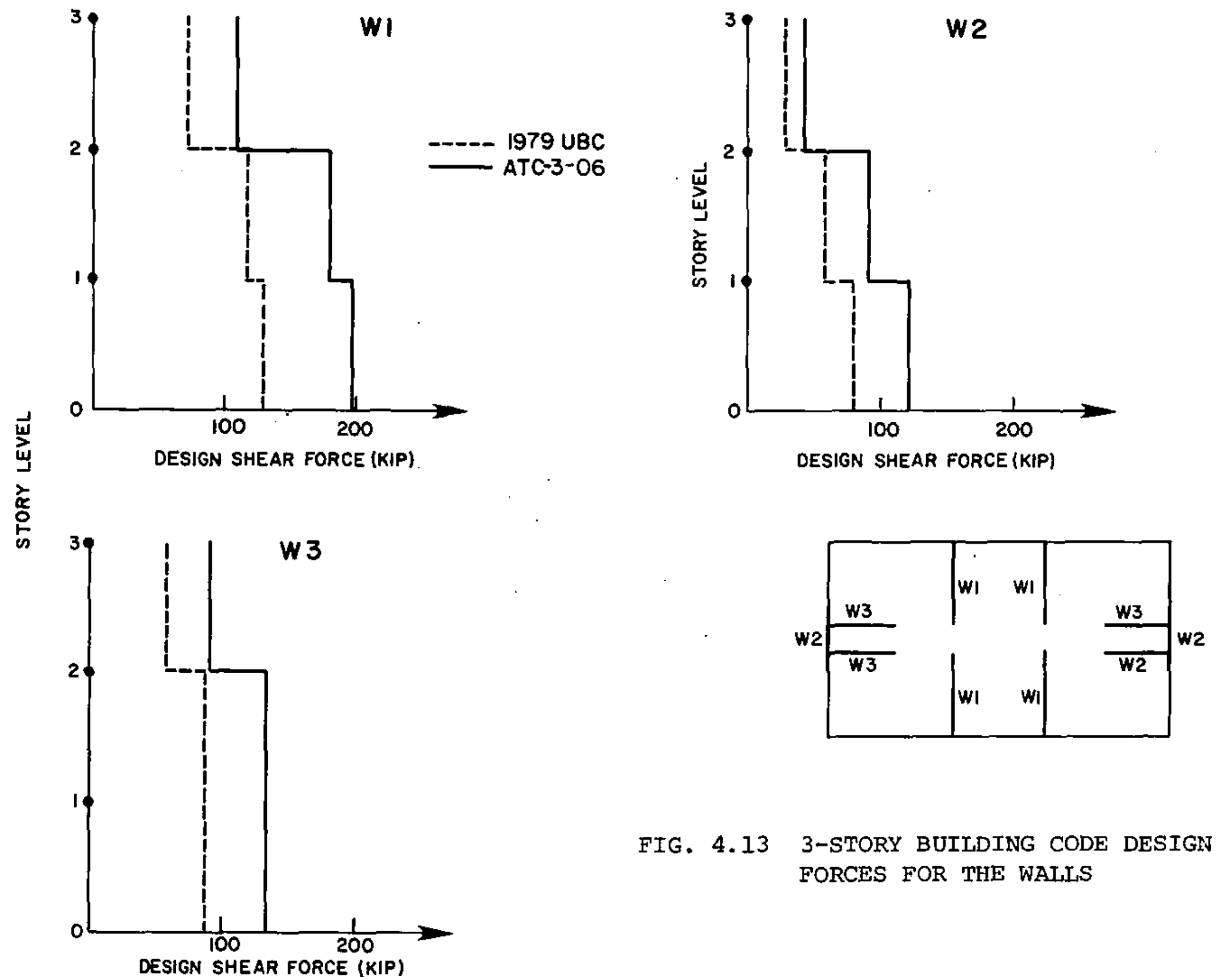

FIG. 4.13 3-STORY BUILDING CODE DESIGN SHEAR FORCES FOR THE WALLS

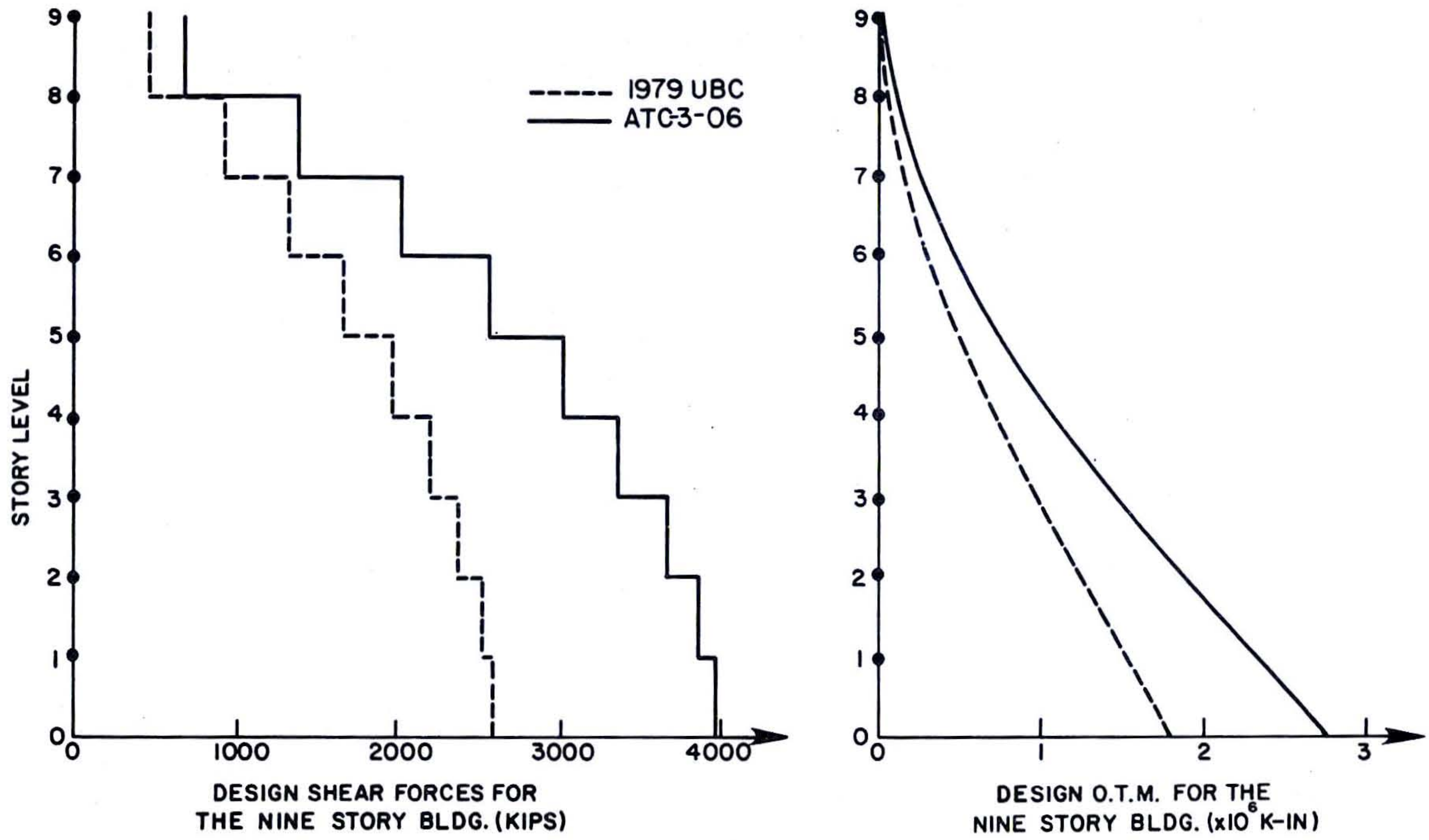

FIG. 4.14 CODE DESIGN SHEAR FORCES AND OVERTURNING MOMENTS FOR THE 9-STORY BUILDING - BOTH DIRECTIONS

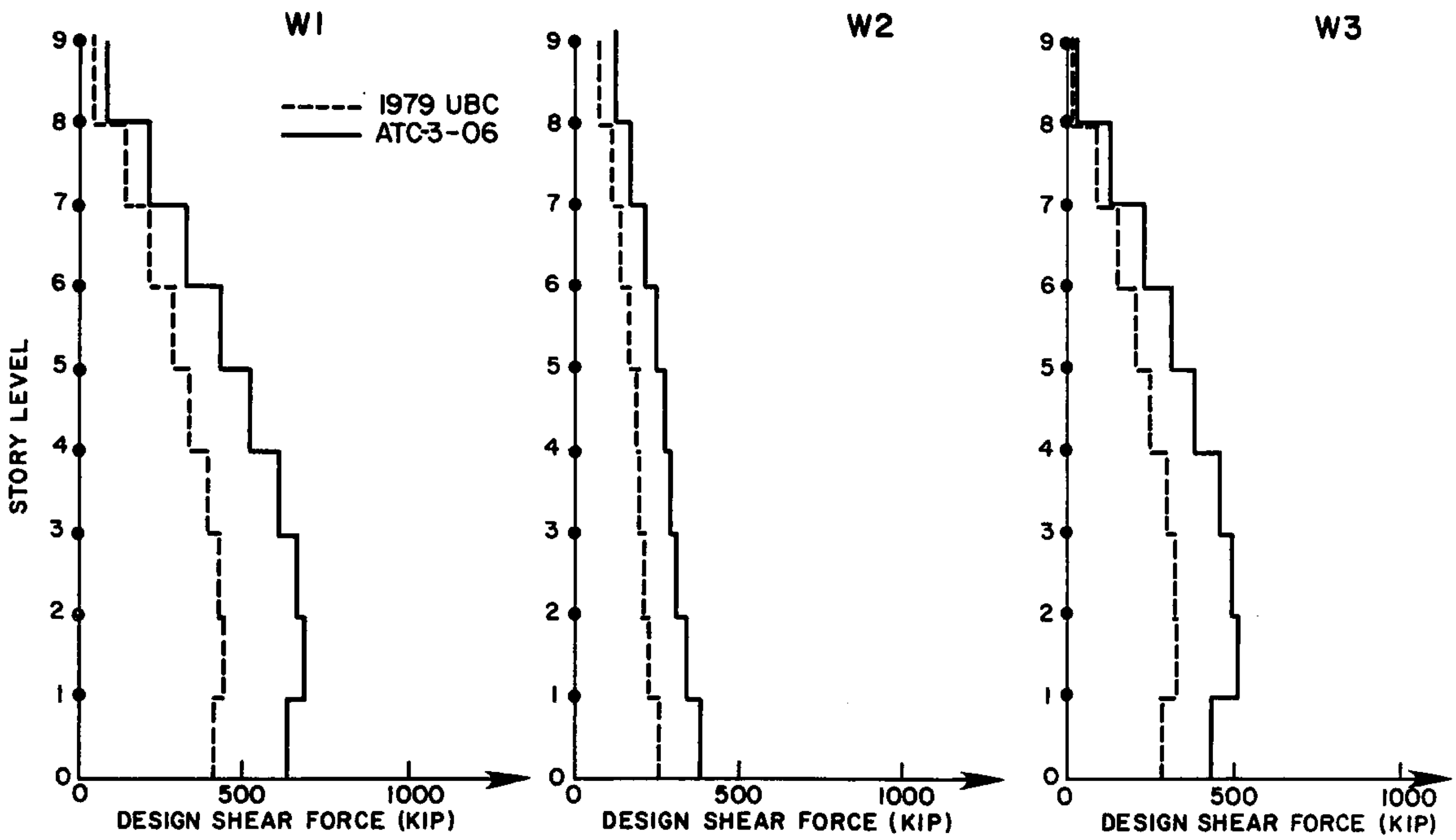

FIG. 4.15 9-STORY BUILDING CODE DESIGN SHEAR FORCES FOR THE WALLS

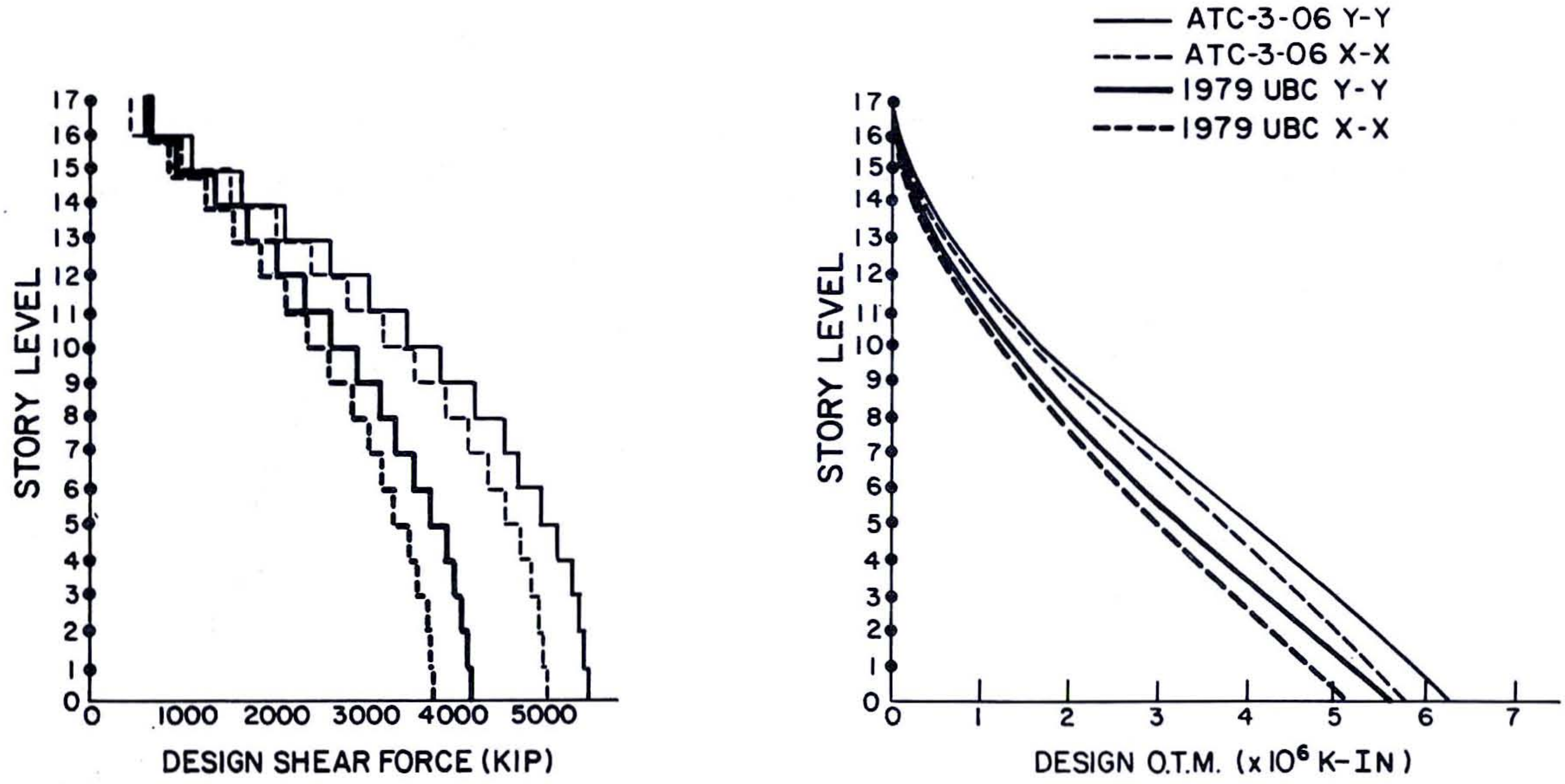

FIG. 4.16 CODE DESIGN SHEAR FORCES AND OVERTURNING MOMENTS FOR THE 17-STORY BUILDING - BOTH DIRECTIONS

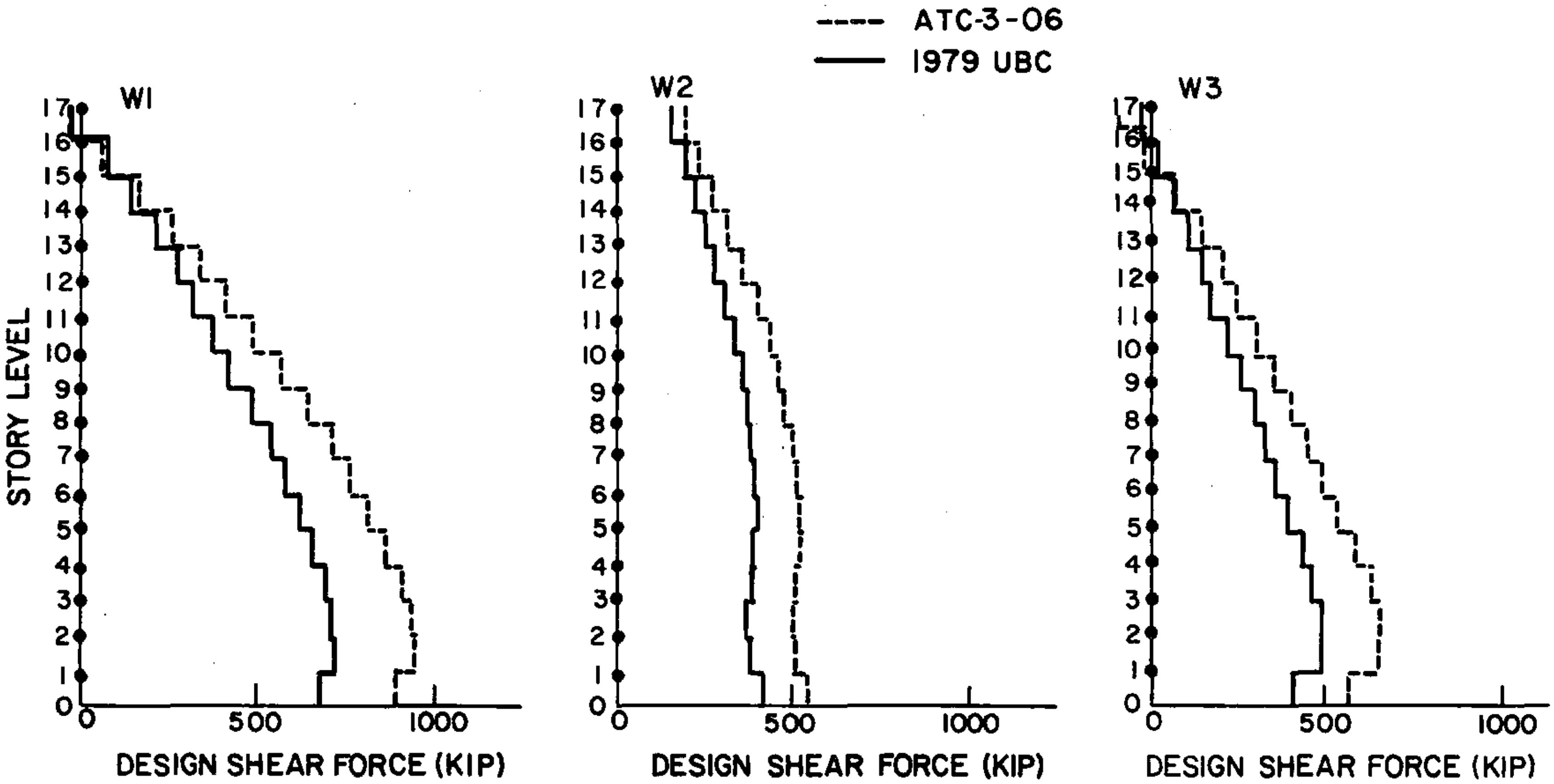

FIG. 4.17 17-STORY BUILDING CODE DESIGN SHEAR FORCES FOR THE WALLS

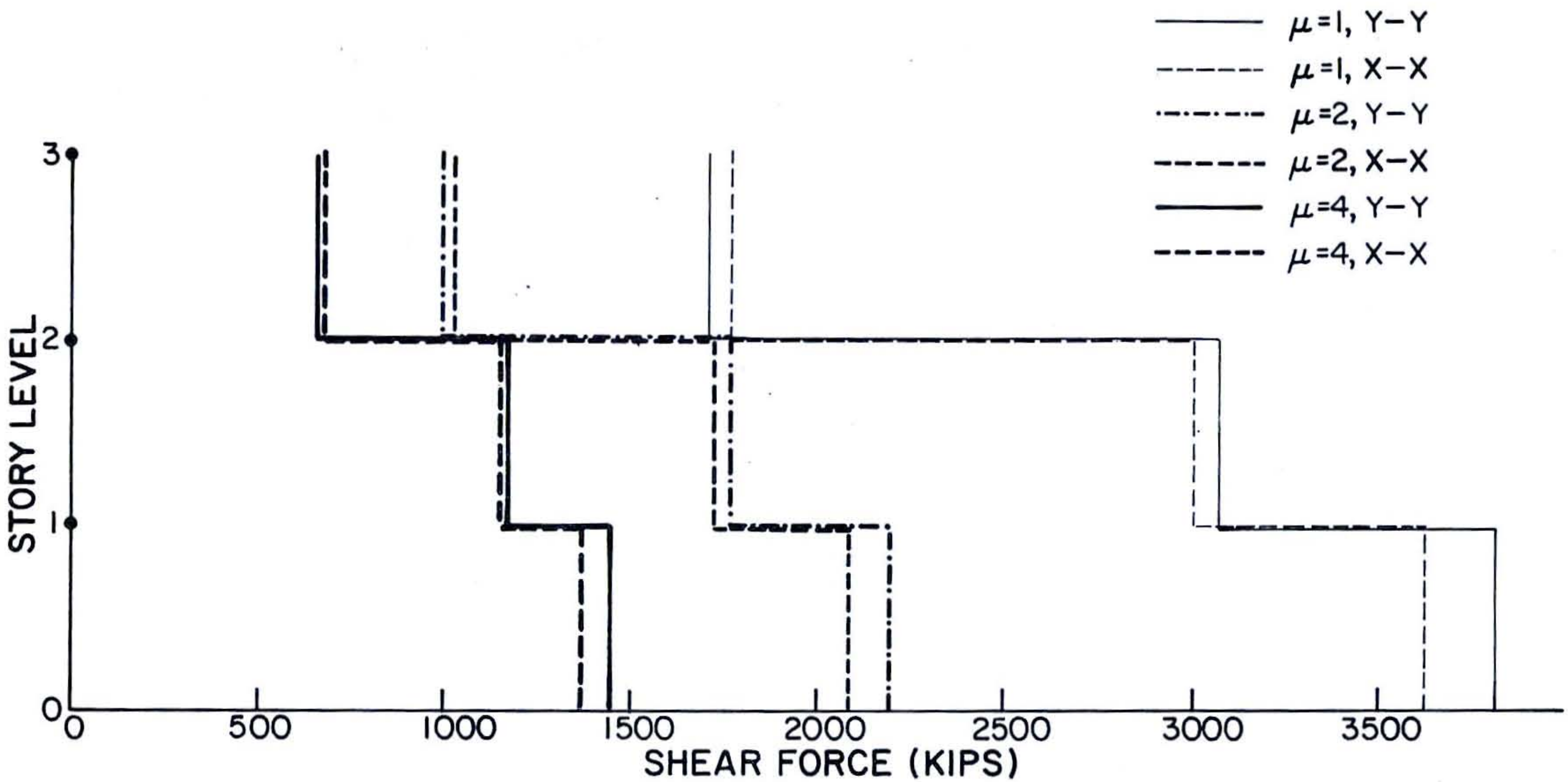

FIG. 4.18 3-STORY BUILDING - STORY SHEARS RESULTING FROM THE RESPONSE SPECTRUM ANALYSIS

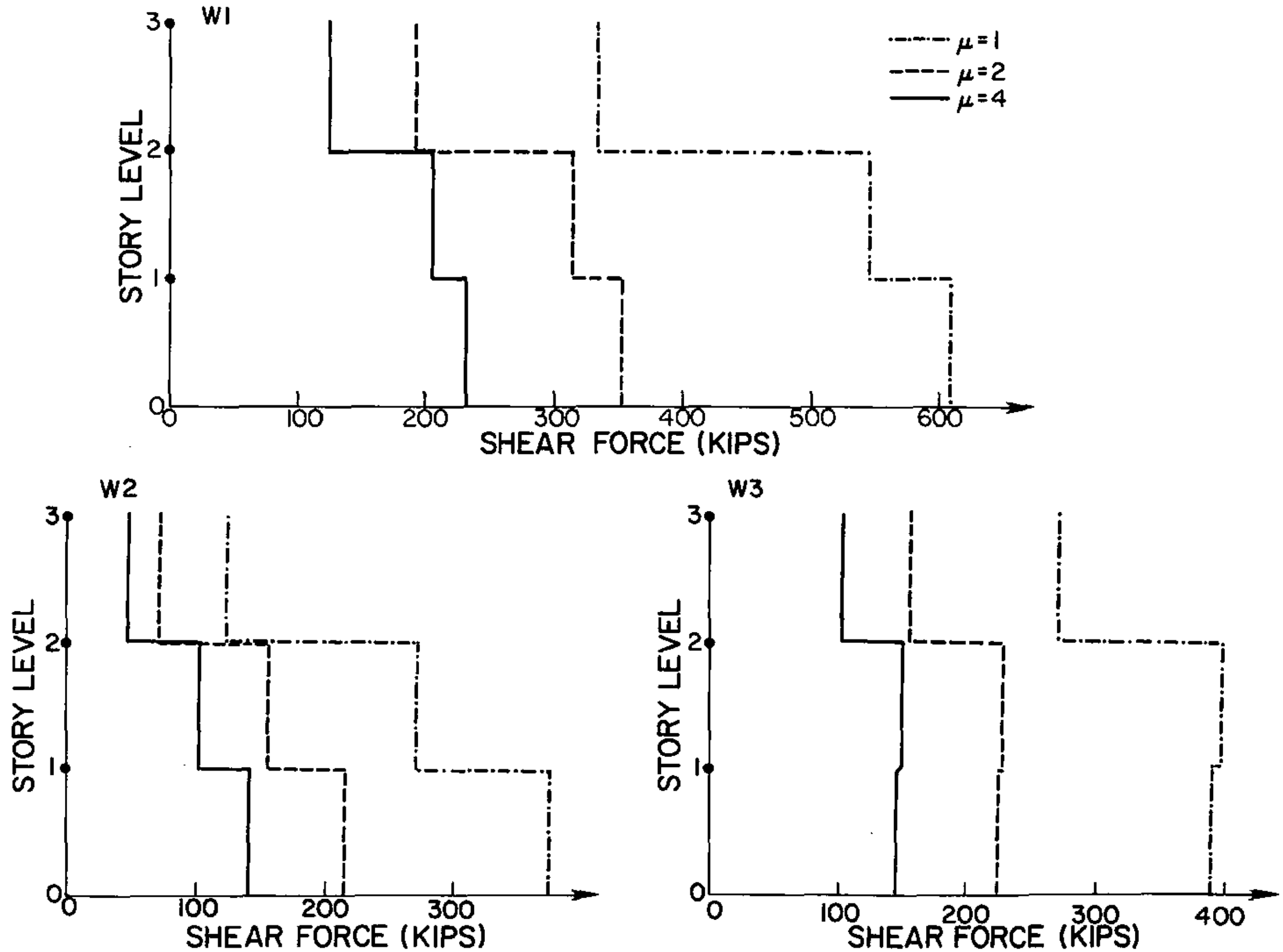

FIG. 4.19 3-STORY BUILDING - SPECTRAL ANALYSIS SHEAR FORCES FOR THE WALLS

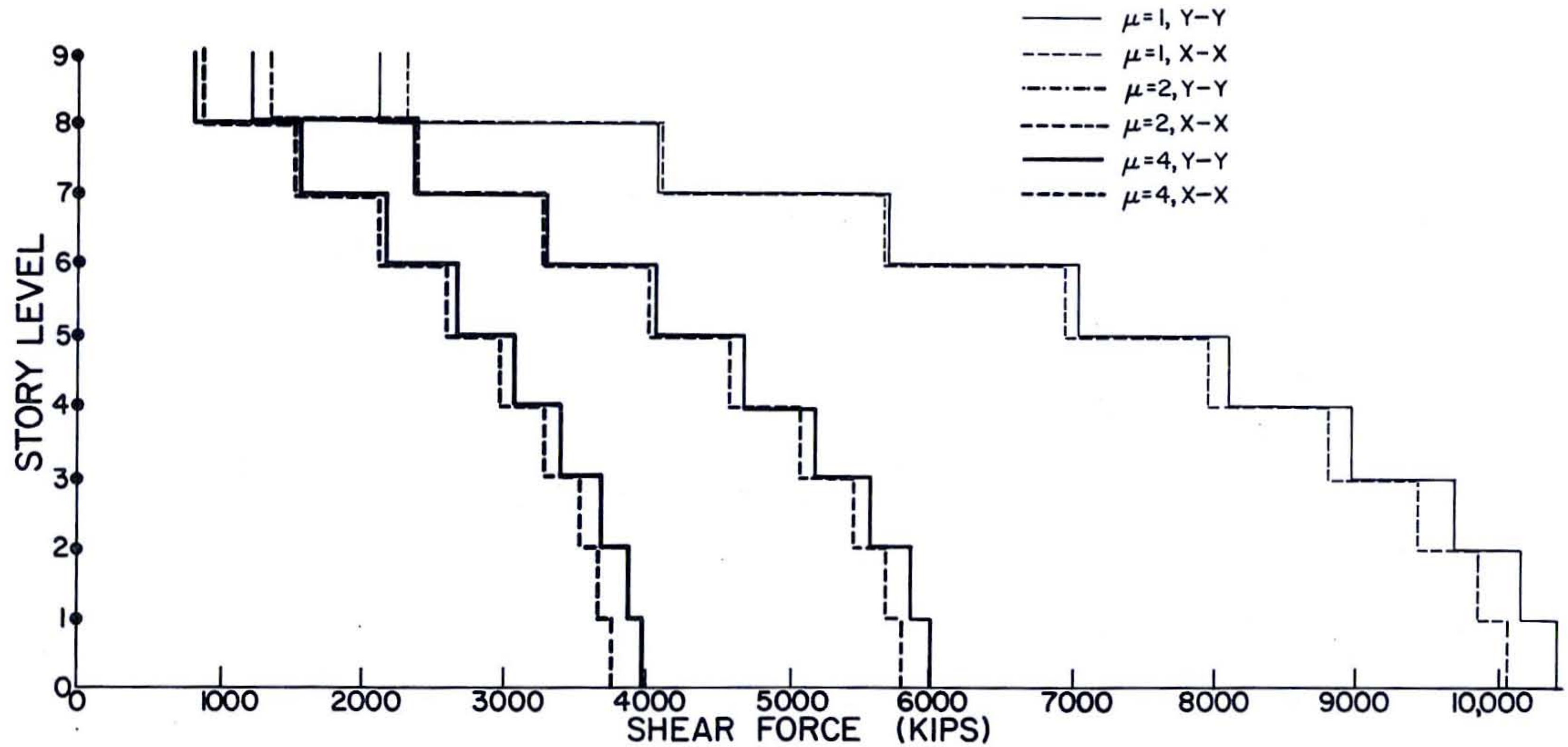

FIG. 4.20 9-STORY BUILDING - STORY SHEARS RESULTING FROM RESPONSE SPECTRUM ANALYSIS

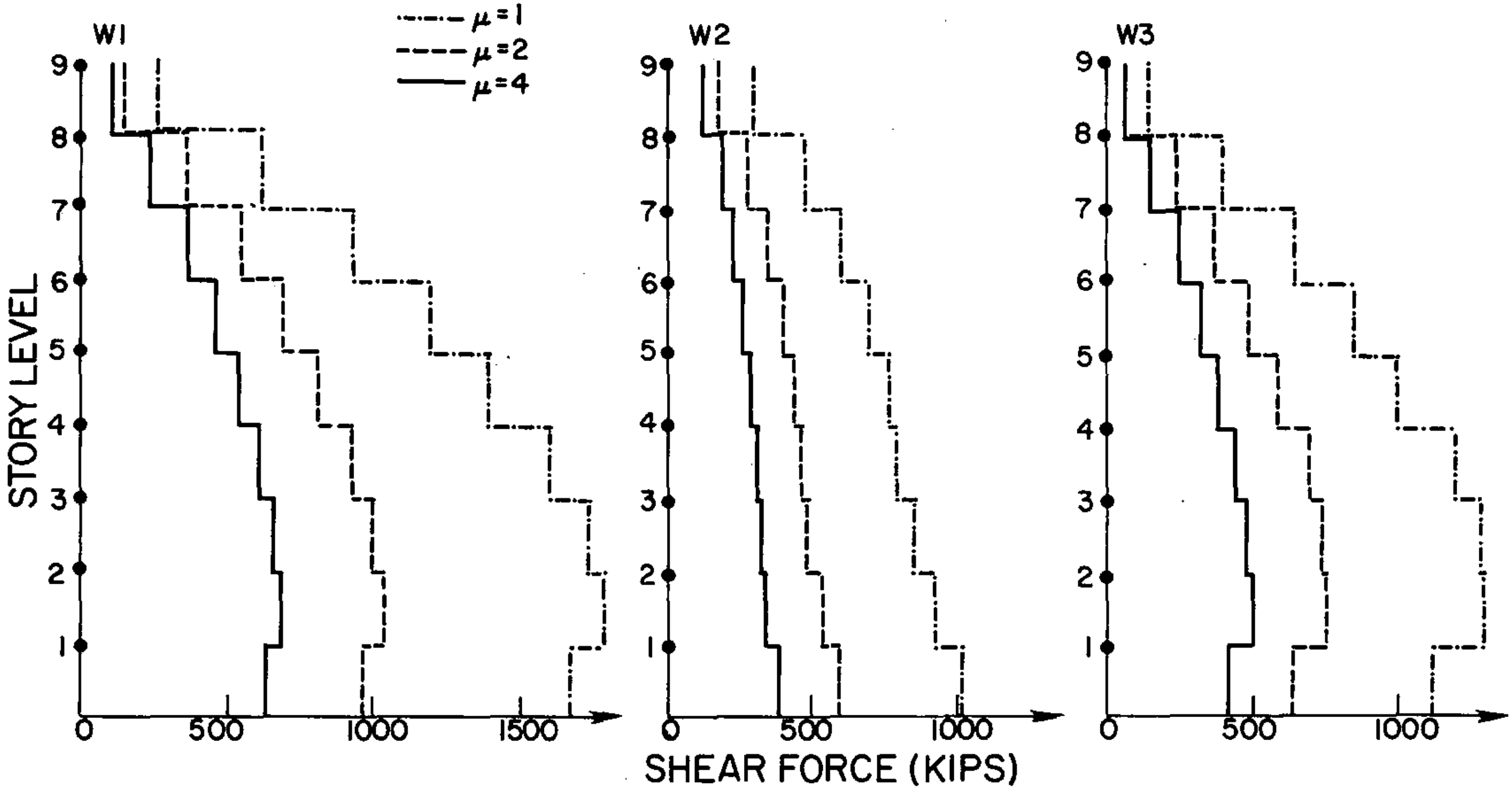

FIG. 4.21 9-STORY BUILDING - SPECTRAL ANALYSIS SHEAR FORCES FOR THE WALLS

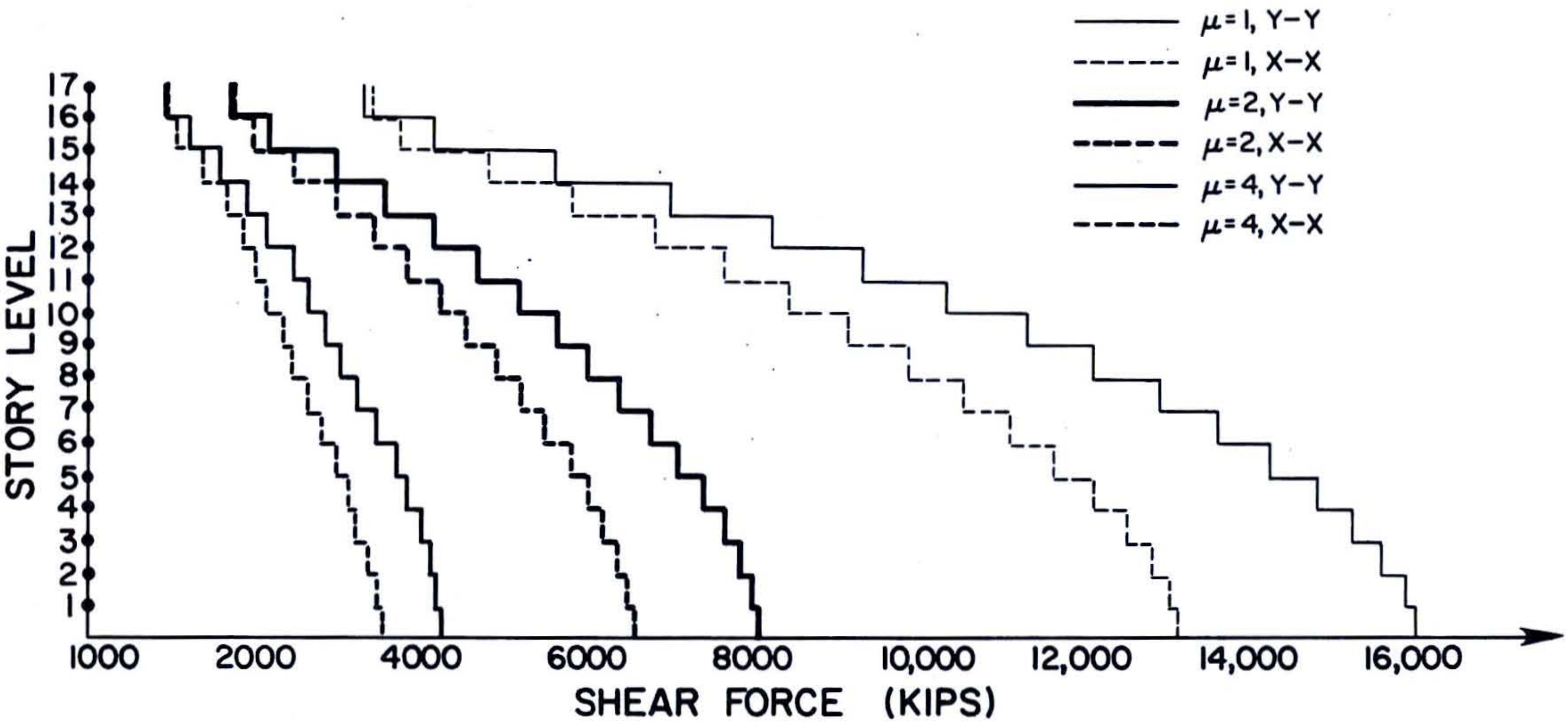

FIG. 4.22 17-STORY BUILDING - STORY SHEARS RESULTING FROM THE RESPONSE SPECTRUM ANALYSIS

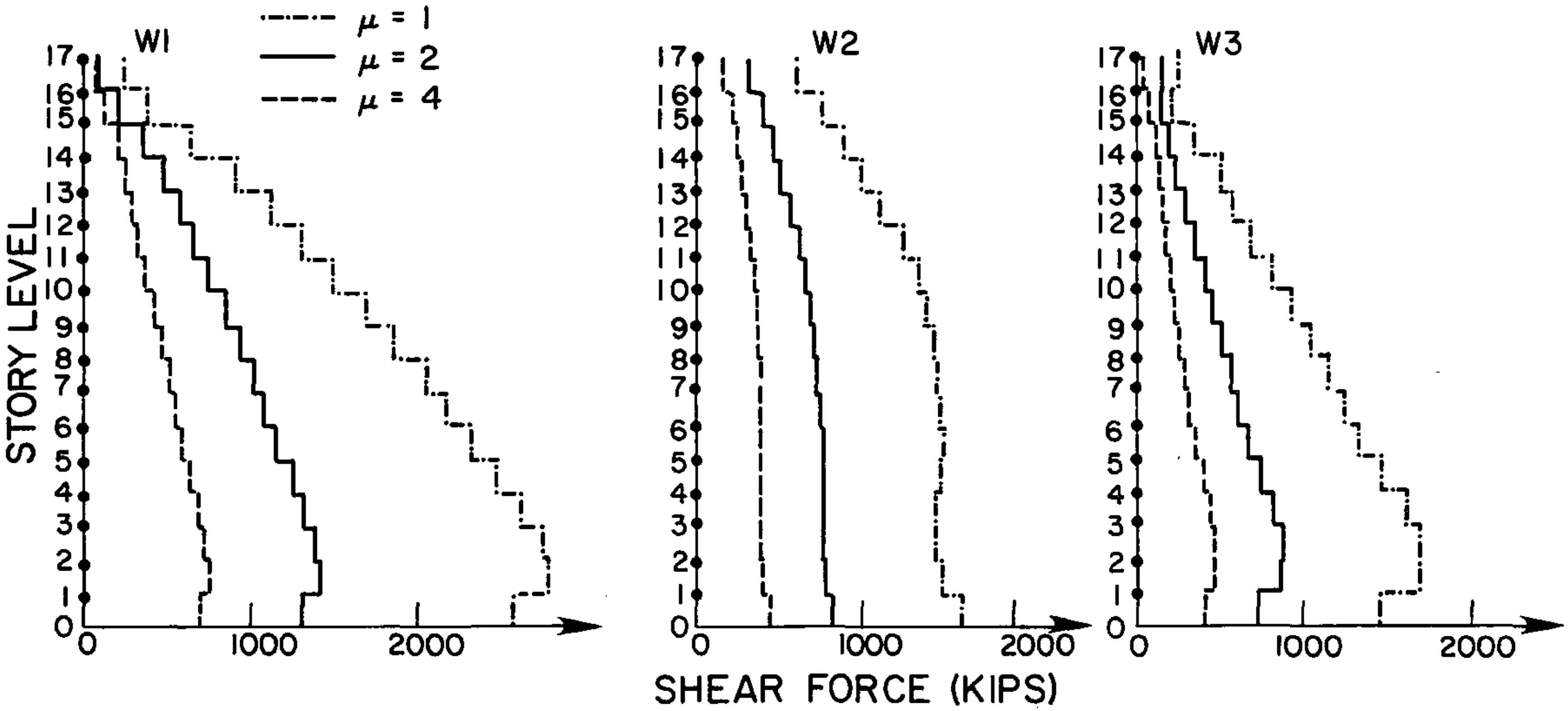

FIG. 4.23 17-STORY BUILDING - SPECTRAL ANALYSIS SHEAR FORCES FOR THE WALLS

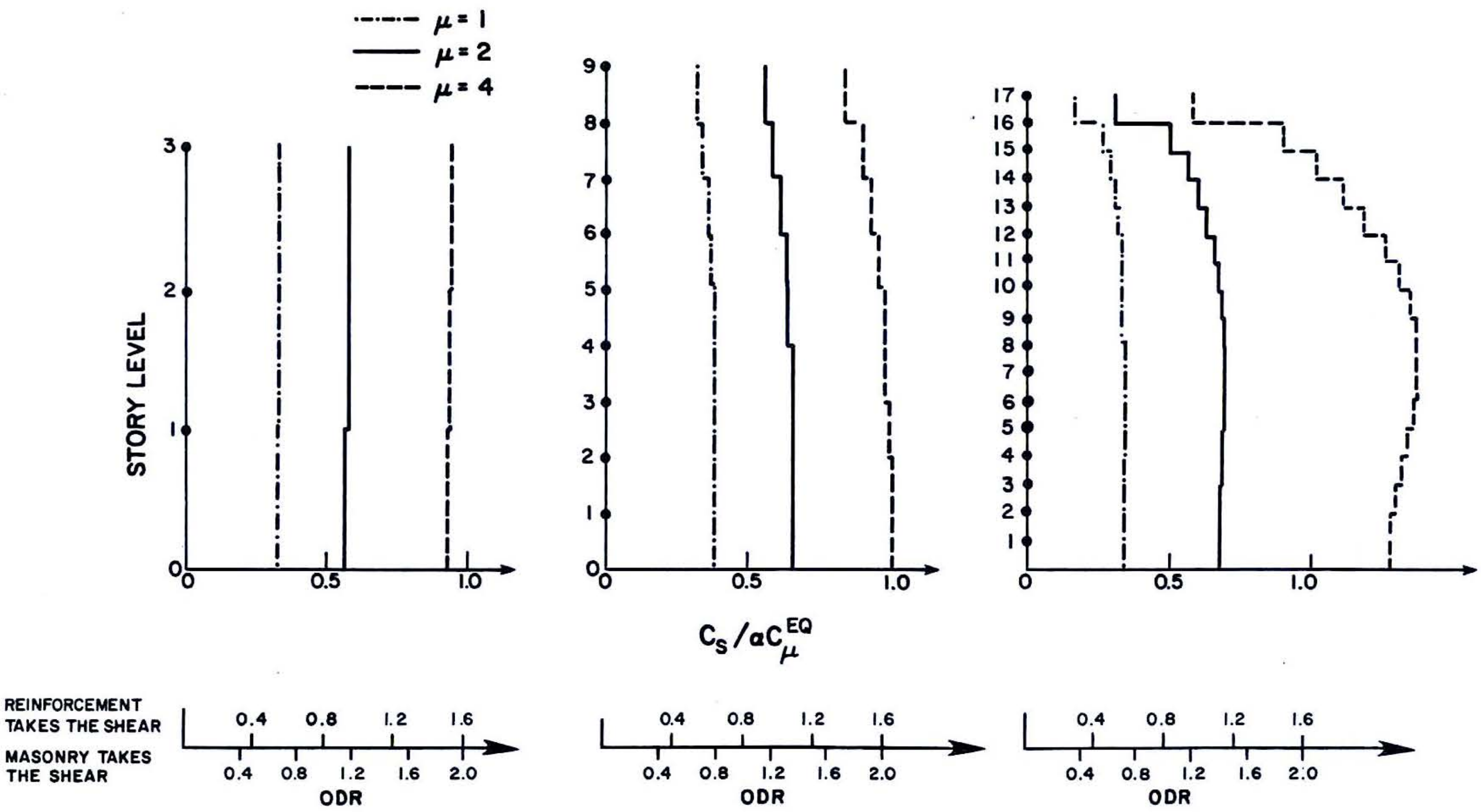

FIG. 4.24 ATC-3-06 STORY BY STORY OVER-DESIGN RATIO FOR THE THREE BUILDINGS; FOR THE ZONE OF HIGHEST SEISMICITY (MAP AREA 7) AND HCBL; M/Vd = 0

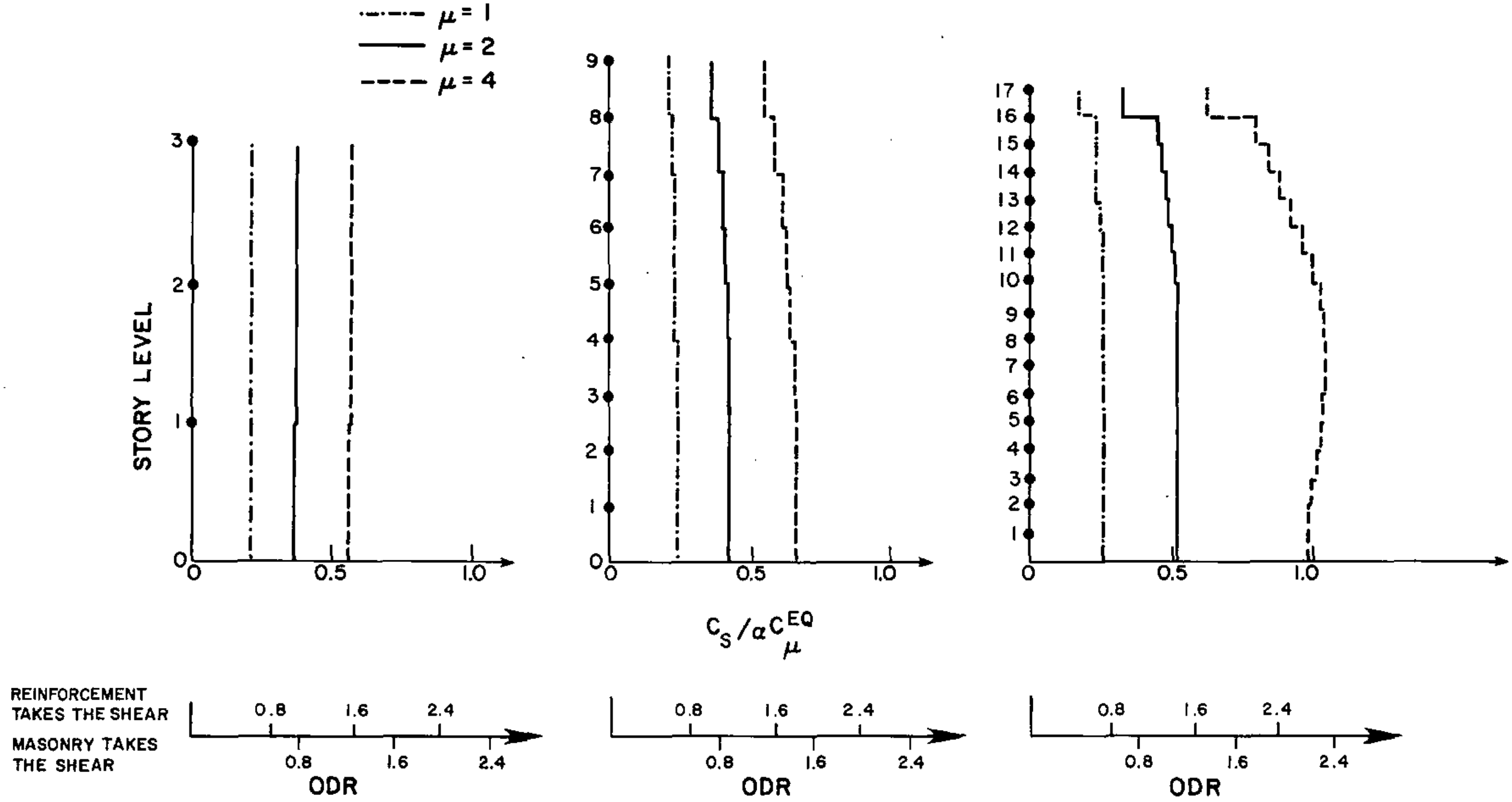

FIG. 4.25 1979 UBC STORY BY STORY OVER-DESIGN RATIO FOR THE THREE BUILDINGS; FOR THE ZONE OF HIGHEST SEISMICITY (Z = 1.0) AND HCBL; M/Vd = 0

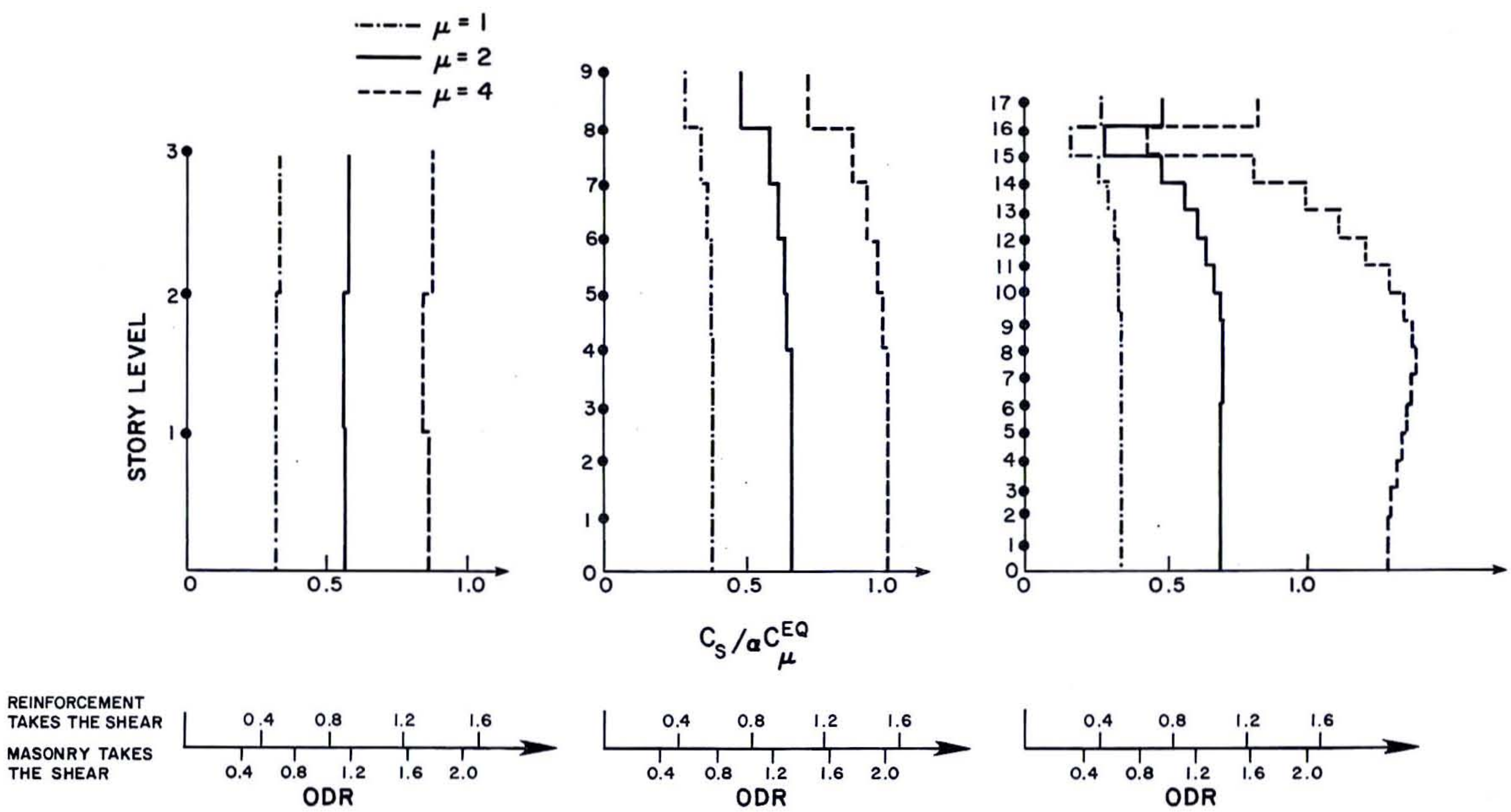

FIG. 4.26 ATC-3-06 STORY BY STORY OVER-DESIGN RATIO FOR W1 OF THE THREE BUILDINGS; FOR THE ZONE OF HIGHEST SEISMICITY (MAP AREA 7) AND HCBL; M/Vd = 0.15

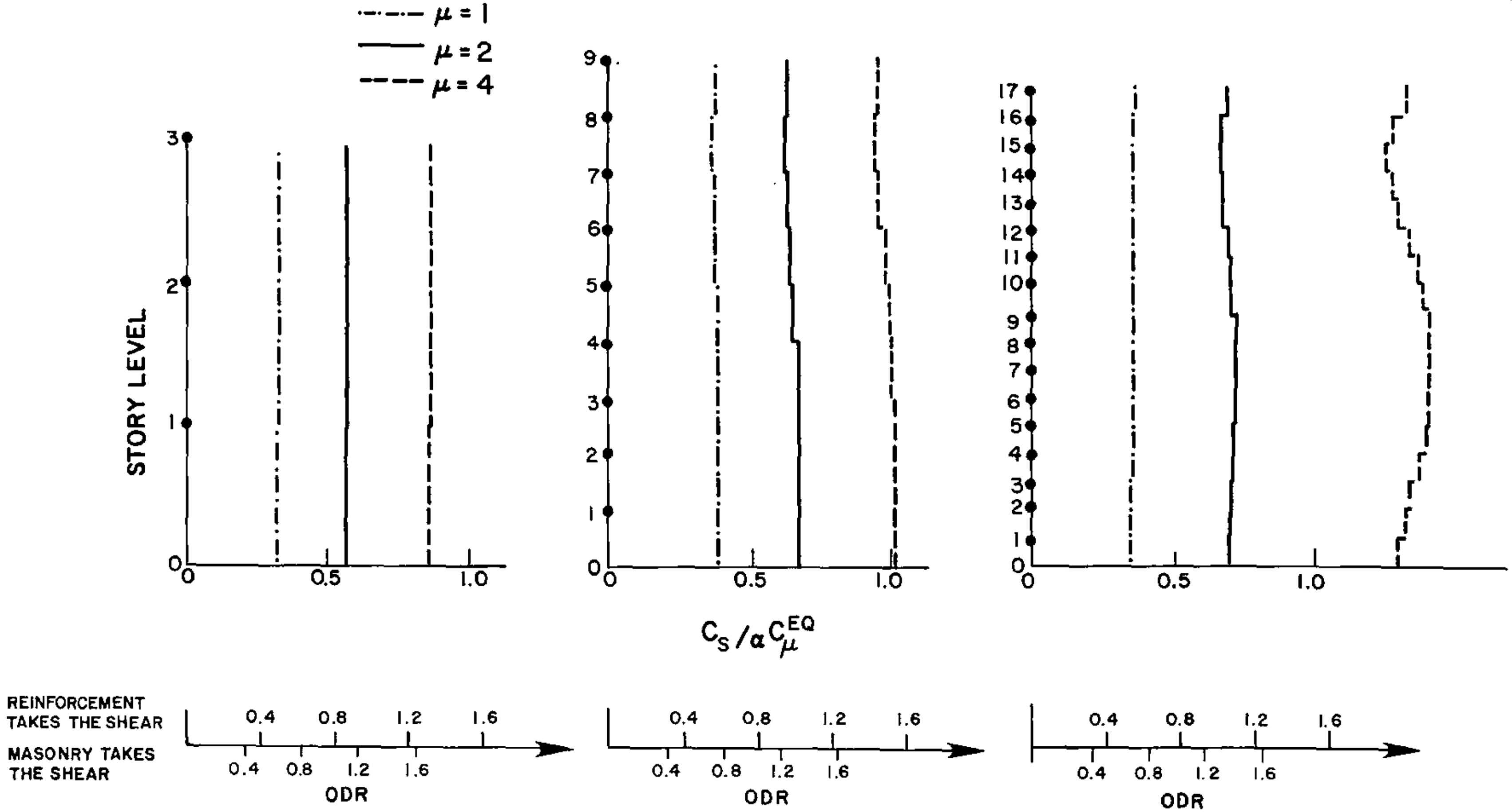

FIG. 4.27 ATC-3-06 STORY BY STORY OVER-DESIGN RATIO FOR W2 OF THE THREE BUILDINGS; FOR THE ZONE OF HIGHEST SEISMICITY (MAP AREA 7) AND HCBL; M/Vd = 0.21

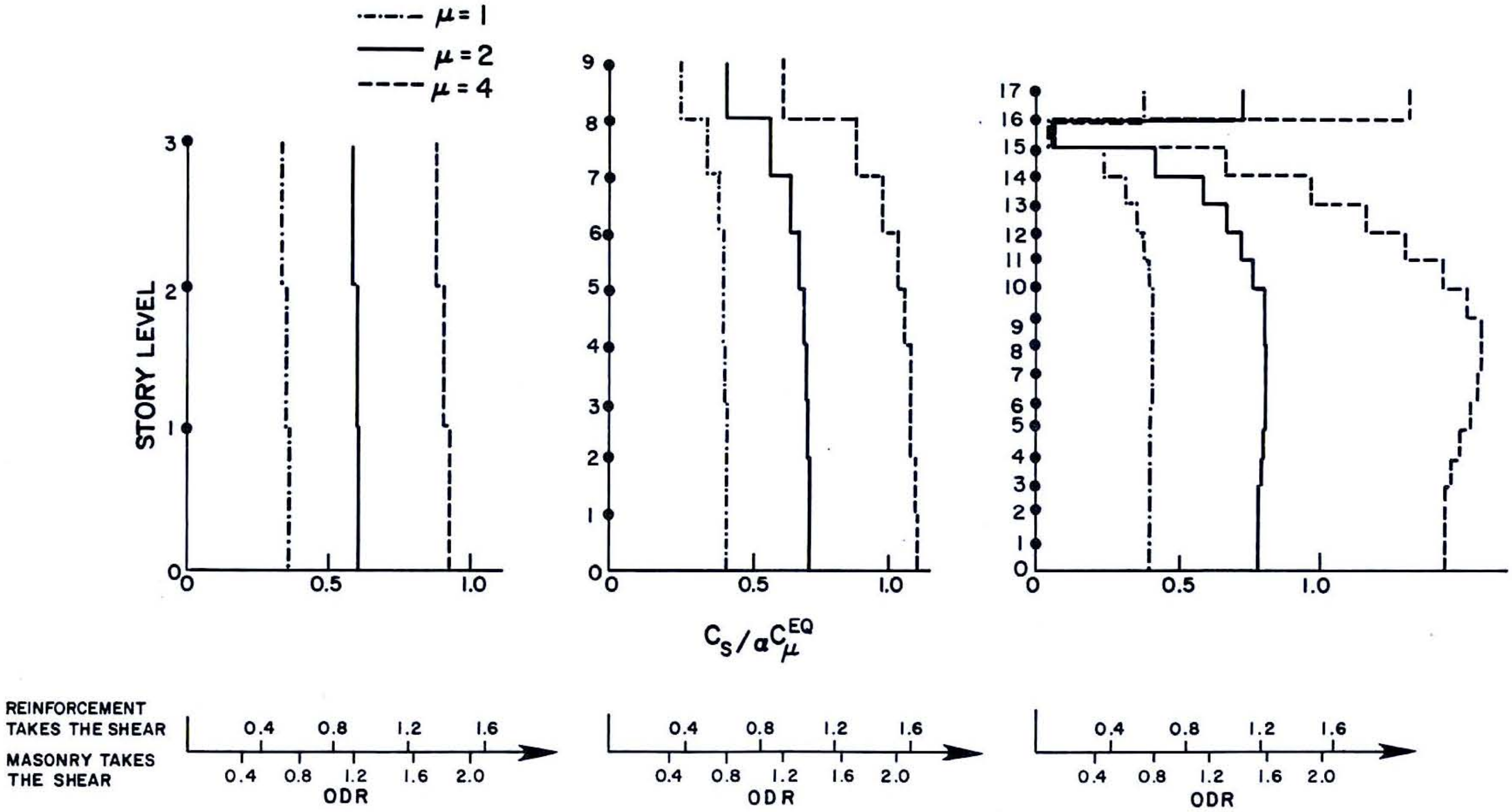

FIG. 4.28 ATC-3-06 STORY BY STORY OVER-DESIGN RATIO FOR W3 OF THE THREE BUILDINGS; FOR THE ZONE OF HIGHEST SEISMICITY (MAP AREA 7) AND HCBL; M/Vd = 0.19

TABLE 4.3

THE OVER DESIGN RATIO; FULLY GROUTED REINFORCED HOLLOW CONCRETE BLOCK MASONRY; f'_m = 3000 psi - REINFORCEMENT TAKES THE SHEAR

Building Type	Force Type	Allowable Stresses and Calculated Loads: W1 Stress (psi) $\frac{M}{Vd} = 0.15$	W1 Load (kips)	W2 Stress (psi) $\frac{M}{Vd} = 0.21$	W2 Load (kips)	W3 Stress (psi) $\frac{M}{Vd} = 0.19$	W3 Load (kips)	Area Required (in^2) W1	W2	W3	ATC3-06 ODR W1	W2	W3	UBC 1979 ODR W1	W2	W3
3 Story	ATC3-06	156.5	197.9	153.4	121.9	154.5	134.0	1265	795	867	1.00	1.00	1.00	--	--	--
	UBC 1979	92.8	129.0	91.0	79.5	91.6	87.4	1390	874	954	--	--	--	1.00	1.00	1.00
	EQ; μ = 1	304.0	608.7	294.1	376.4	297.4	391.5	2002	1280	1316	0.63	0.62	0.66	0.69	0.68	0.72
	EQ; μ = 2	273.6	351.2	264.7	217.2	267.7	225.9	1284	821	844	0.99	0.97	1.03	1.08	1.06	1.13
	EQ; μ = 4	243.2	230.1	235.3	142.3	237.9	148.0	946	605	622	1.34	1.31	1.39	1.47	1.44	1.53
9 Story	ATC3-06	156.5	634.8	153.4	387.8	154.5	435.0	4056	2528	2816	1.00	1.00	1.00	--	--	--
	UBC 1979	92.8	413.7	91.0	252.7	91.6	283.5	4458	2777	3095	--	--	--	1.00	1.00	1.00
	EQ; μ = 1	304.0	1668.5	294.1	1018.6	297.4	1104.3	5488	3463	3713	0.74	0.73	0.76	0.81	0.80	0.83
	EQ; μ = 2	273.6	962.7	264.7	587.8	267.7	637.2	3519	2221	2380	1.15	1.14	1.18	1.27	1.25	1.30
	EQ; μ = 4	243.2	630.7	235.3	385.1	237.9	411.2	2593	1637	1728	1.56	1.54	1.63	1.72	1.70	1.79
17 Story	ATC3-06	156.5	884.5	153.4	548.6	154.5	560.8	5652	3576	3630	1.00	1.00	1.00	--	--	--
	UBC 1979	92.8	678.4	91.0	421.0	91.6	420.5	7310	4626	4591	--	--	--	1.00	1.00	1.00
	EQ; μ = 1	304.0	2565.6	294.1	1587.2	297.4	1456.5	8439	5397	4897	0.67	0.66	0.74	0.87	0.86	0.94
	EQ; μ = 2	273.6	1298.9	264.7	802.7	267.7	734.4	4747	3032	2743	1.19	1.18	1.32	1.54	1.53	1.67
	EQ; μ = 4	243.2	688.0	235.3	424.1	237.9	393.8	2829	1802	1655	2.00	1.98	2.19	2.58	2.57	2.77

W1, W2 and W3 are explained in Fig. 4.13.

TABLE 4.4

THE OVER DESIGN RATIO; FULLY GROUTED REINFORCED HOLLOW CONCRETE BLOCK MASONRY; f'_m = 3000 psi - MASONRY TAKES THE SHEAR

		Allowable Stresses and Calculated Loads						Area Required (in^2)			ATC3-06 ODR			UBC 1979 ODR		
		W1		W2		W3										
		Stress (psi)		Stress (psi)		Stress (psi)										
Building Type	Force Type	$\frac{M}{Vd}$ = 0.15	Load (kips)	$\frac{M}{Vd}$ = 0.21	Load (kips)	$\frac{M}{Vd}$ = 0.19	Load (kips)	W1	W2	W3	W1	W2	W3	W1	W2	W3
3 Story	ATC3-06	48.5	197.9	47.9	121.9	48.1	134.0	4080	2545	2786	1.00	1.00	1.00	--	--	--
	UBC 1979	42.3	129.0	41.4	79.5	41.7	87.4	3050	1920	2096	--	--	--	1.00	1.00	1.00
	EQ; μ = 1	249.2	608.7	239.4	376.4	242.6	391.5	2443	1572	1614	1.670	1.619	1.726	1.25	1.22	1.30
	EQ; μ = 2	199.4	351.2	191.5	217.2	194.1	225.9	1761	1134	1164	2.317	2.244	2.393	1.73	1.69	1.80
9 Story	ATC3-06	48.5	634.8	47.9	387.8	48.1	435.0	13089	8096	9044	1.00	1.00	1.00	--	--	--
	UBC 1979	42.3	413.7	41.4	252.7	41.7	283.5	9780	6104	6799	--	--	--	1.00	1.00	1.00
	EQ; μ = 1	249.2	1668.5	239.4	1018.6	242.6	1104.3	6695	4255	4552	1.955	1.903	1.987	1.46	1.43	1.49
	EQ; μ = 2	199.4	962.7	191.5	587.8	194.1	637.2	4828	3069	3283	2.711	2.638	2.355	2.03	1.99	2.07
17 Story	ATC3-06	48.5	884.5	47.9	548.6	48.1	560.8	18237	11453	11659	1.00	1.00	1.00	--	--	--
	UBC 1979	42.3	678.4	41.4	421.0	41.7	420.5	16038	10169	10084	--	--	--	1.00	1.00	1.00
	EQ; μ = 1	249.2	2565.6	239.4	1587.2	242.6	1456.5	10295	6630	6004	1.771	1.728	1.942	1.56	1.53	1.68
	EQ; μ = 2	199.4	1298.9	191.5	802.7	194.1	734.4	6514	4192	3784	2.800	2.732	3.081	2.46	2.43	2.66

W1, W2 and W3 are explained in Fig. 4.13.

5. EVALUATION OF THE 1979 UBC AND ATC-3-06 SEISMIC DESIGN PROVISIONS

The method used to evaluate the two sets of seismic design provisions is described in Section 2.2 and is based on the Over-Design Ratio (ODR). If the ODR is significantly greater than 1, then the design provisions are considered to be conservative; if it is significantly less than 1, the design provisions are considered to be non-conservative. The accuracy of the ODR values presented in the preceding sections depends on the accuracy of the four variables (L_c, L_{eq}, R_c and R_{eq}) that constitute the ODR. Two of the four factors, the code load (L_c) and code allowable stress (R_c), are specified by the design provisions. The ultimate strength R_{eq}, is determined from test data as described in Section 2.7. Although further testing is necessary, R_{eq} calculated from the test data currently available is considered to be a reasonable estimate of the ultimate shear strength. The greatest degree of uncertainty is in the "realistic" earthquake load, L_{eq}. This is due to uncertainties in earthquake ground motion studies and to the inaccuracies inherent in the use of a ductility reduced elastic spectrum to represent the inelastic response of a masonry building as described in Section 2.5. Nonetheless the ODR provides a reasonable basis for evaluating the adequacy of seismic design provisions at this time.

From the discussion of results presented in Section 3.6, it is clear that the effective allowable shear stresses for seismic loads for the design provisions considered in this study require some adjustments. Care must be exercised as adjustments are made since the results presented here only consider the effects of seismic loads.

Adjustments in the effective allowable shear stresses can be made in various ways depending on the particular design provision. For ATC-3-06 the ϕ factor, the R-factor or the allowable shear stress can be adjusted. For the UBC the K-factor, the 1.5 factor to increase the seismic shear load or the allowable shear stress can be adjusted. However, before adjustments are contemplated to either set of provisions, it is clear that serious consideration should be given to the use of separate allowable shear stresses for different types of reinforced masonry construction. This is currently incorporated in the allowable shear stresses for unreinforced masonry, but reinforced masonry uses the same allowable shear stresses for all types of construction. If this change is not made and the allowable shear stresses are adjusted so the ODRs are approximately equal to or greater than 1, there will then be a considerable amount of conservatism for some materials.

In the ATC-3-06 Tentative Provisions, it is clear that the ODRs for partially reinforced masonry are very conservative. Adjustments for this type of construction can be made either by increasing the R-factor above 1.25 or by using higher allowable shear stresses. For reinforced masonry the effective allowable shear stresses are reasonable for $M/Vd = 0$ for both fully grouted hollow concrete and hollow clay brick construction, but non-conservative for the grouted core clay brick method of construction. For $M/Vd \geq 1$ the effective allowable shear stresses for fully grouted hollow clay brick and grouted core clay brick walls are reasonable when reinforcement takes the shear and conservative when masonry takes the shear. For fully grouted hollow concrete block construction the effective allowable shear stresses are non-conservative for both cases of reinforced masonry.

For the 1979 UBC, the effective allowable shear stresses for partially reinforced masonry are non-conservative for grouted core clay brick and fully grouted hollow concrete block ($M/Vd \geq 1$) methods of construction. For fully grouted hollow clay brick and hollow concrete block ($M/Vd = 0$) the effective allowable shear stresses are conservative. Furthermore, adjustments in these allowable stresses can be made for material type since they are the allowable stresses for unreinforced masonry. It should be noted, however, that the effective allowable shear stresses for unreinforced masonry do not differentiate between walls with different M/Vd ratios.

In the 1979 UBC, for reinforced masonry with $M/Vd = 0$, the effective allowable shear stresses when reinforcement takes the shear are reasonable for both fully grouted hollow concrete block and hollow clay brick. For grouted core clay brick construction they are non-conservative. When masonry takes the shear the effective allowable shear stresses for all three methods of construction are non-conservative. For $M/Vd \geq 1$ the effective allowable shear stresses are reasonable when either masonry or reinforcement takes the shear for fully grouted hollow clay brick and grouted core clay brick construction. For fully grouted hollow concrete block, the effective allowable shear stresses are non-conservative when either masonry or reinforcement takes the shear.

It should be noted that the ultimate strengths and associated ductility factors used here to determine the ODRs were derived from tests on fully grouted hollow concrete and clay brick piers. In the limited number of tests performed on partially grouted piers, the performance of partially grouted hollow concrete block piers has been similar to that of fully grouted piers and, therefore, the conclusions presented here would be applicable to both fully and partially grouted hollow concrete block construction. However, the same situation is not applicable to partially grouted hollow clay brick piers, which have little or no ductile capacity and whose net strength varies between 70% and 100% of that for fully grouted piers. Thus, significantly lower effective allowable shear stresses would have to be used for partially grouted hollow clay brick construction in comparison to those used for fully grouted construction.

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APPENDIX A

HIGHER MODE EFFECTS

As indicated by Eq. 2.22 the total base shear in a building should be found by including at least the first few modes of vibration provided two modes of vibration in the same direction do not have frequencies close to one another. This is usually done in one of two ways, the first of which is a direct summation of the absolute values of each mode so that

$$V = \sum_{i=1}^{N} |V_i| \qquad \text{(A-1)}$$

where

V = the total base shear

V_i = base shear of mode i

N = total number of modes considered.

This approach is a conservative one, because the modal maximums do not in general occur at the same time. The second method, which is frequently used, takes the square root of the sum of the squares or

$$V = \left[\sum_{i=1}^{N} (V_i)^2 \right]^{1/2} \qquad \text{(A-2)}$$

This method also has its defects, but works well for symmetrical buildings (no modal coupling) for which no two modes in the same direction have similar periods of vibration.

Since we used only the fundamental mode in calculating the base shear for comparison with the codes, we now compare this base shear with the appropriate shear when higher modes are taken into account. We compare the two base shears for each of the three buildings described in Chapter 4. For each building the first three modes of each translational direction are combined using the root-mean-square method of ETABS [14]; this corresponds to the second method mentioned above.

For known mode shapes, the maximum elastic force vector in mode i is (see reference [15])

$$\overline{F}_{si,max} = \overline{M}\,\overline{\phi}_i\,\frac{L_i}{M_i}\,S_a(\xi_i, T_i) \tag{A-3}$$

where

$\overline{M}$ = the diagonal mass matrix,

$\overline{\phi}_i$ = the i th mode shape vector,

$L_i \equiv \overline{\phi}_i^T\,\overline{M}\,\{\overline{1}\} = \sum_{j=1}^{N} m_j\,\phi_{ij}$,

$M_i \equiv \overline{\phi}_i^T\,\overline{M}\,\overline{\phi}_i = \sum_{j=1}^{N} m_j\,\phi_{ij}^2$,

$\{\overline{1}\}$ = unit column vector ,

$S_a(\xi_i, T_i)$ = the spectral acceleration for damping ξ_i and period T_i; - units of in./sec^2.

The base shear in each mode can now be obtained from

$$V_i = \{\overline{1}\}^T\,\overline{F}_{si,max} = \frac{L_i^2}{M_i}\,S_a(\xi_i, T_i). \tag{A-4}$$

Comparing Eq. A-4 and Eq. 2.23, we see that the term $\frac{L_i^2}{M_i}$ represents the effective mass vibrating in mode i;

i.e.,

$$M_{i,eff} = \frac{L_i^2}{M_i} = \frac{\left[\sum_{j=1}^{N} m_j\phi_{ij}\right]^2}{\sum_{j=1}^{N} m_j\phi_{ij}^2}, \tag{A-5}$$

as expected.

Table A-1 summarizes the results of the calculations for the three buildings. The base shear for each mode is calculated using Eq. A-4.

The base shear, V, is then calculated using Eq. A-2, and compared with V_T, the value calculated from Eq. 2.23; namely,

$$V_T = \alpha\, S_a(\xi_1, T_1)\, M_{total} \tag{A-6}$$

where

M_{total} is the total mass of the building and α is determined as follows.

The values of α for the 3, 9 and 17-story buildings, calculated from

$$\alpha = \frac{\sqrt{\sum_{i=1}^{N} \left(\frac{L_i^2}{M_i}\right)^2}}{M_{total}}\ , \tag{A-7}$$

are listed in Table A-2. A least squares estimate of α, from these values, assuming a relation of the form

$$\alpha = \frac{a}{T} + b$$

yields

$$\alpha = \frac{0.017}{T} + 0.686 \leq 1.00 \tag{A-8}$$

where T is the fundamental period in the direction considered.

The modal-participation factor gives an estimate of how much of the total weight should be used with the fundamental mode to calculate the basic shear in order to get a close estimate of what the root-mean-square modal combination method would yield.

The base shear V_T is calculated from Eqs. A-6 and A-8, and is then compared with V in the last column of Table A-2. The two methods are shown to be within 4% of each other.

TABLE A-1

MODAL PARAMETERS. 5% DAMPING; $\mu = 1$

Building	Direction	1st Mode $\frac{L_1^2}{M_1}$	1st Mode T_1 (sec)	1st Mode S_a^{Max} (g)	1st Mode V_1 (kip)	2nd Mode $\frac{L_2^2}{M_2}$	2nd Mode T_2 (sec)	2nd Mode S_a^{Max} (g)	2nd Mode V_2 (kip)	3rd Mode $\frac{L_3^2}{M_3}$	3rd Mode T_3 (sec)	3rd Mode S_a^{Max} (g)	3rd Mode V_3 (kip)
3-Story	X-X	9.185	0.098	1.00	3549	1.808	0.030	1.00	699	0.211	0.018	1.00	82
	Y-Y	9.776	0.087	1.00	3777	1.273	0.030	1.00	492	0.154	0.020	1.00	60
9-Story	X-X	25.017	0.409	1.00	9667	6.694	0.109	1.00	2587	2.203	0.053	1.00	851
	Y-Y	26.047	0.315	1.00	10065	6.662	0.093	1.00	2574	1.655	0.049	1.00	639
17-Story	X-X	48.874	0.903	0.648	12237	11.289	0.256	1.00	4362	4.690	0.122	1.00	1812
	Y-Y	48.261	0.732	0.800	14918	13.486	0.201	1.00	5211	4.475	0.100	1.00	1729

TABLE A-2

EFFECTS OF HIGHER MODES 5% DAMPING, $\mu = 1$

Building	Direction	$\frac{L_1^2}{M_1}$	$\frac{L_2^2}{M_2}$	$\frac{L_3^3}{M_3}$	α	α (EQ A-8)	EQ A-6 V_T (kips)	Equation A-4 V_1 (kips)	Equation A-4 V_2 (kips)	Equation A-4 V_3 (kips)	EQ A-2 V (kips)	V_T/V
3-Story	X - X	9.185	1.808	0.211	0.836	0.860	3720	3549	699	82	3618	1.028
	Y - Y	9.776	1.273	0.154	0.881	0.880	3807	3777	492	60	3809	0.999
9-Story	X - X	25.017	6.694	2.203	0.724	0.728	10094	9667	2587	851	10043	1.005
	Y - Y	26.047	6.662	1.655	0.751	0.740	10261	10065	2574	634	10409	0.986
17-Story	X - X	48.878	11.289	4.690	0.706	0.705	12592	12237	4362	1812	13117	0.960
	Y - Y	48.261	13.486	4.475	0.705	0.709	15634	14918	5211	1729	15896	0.984

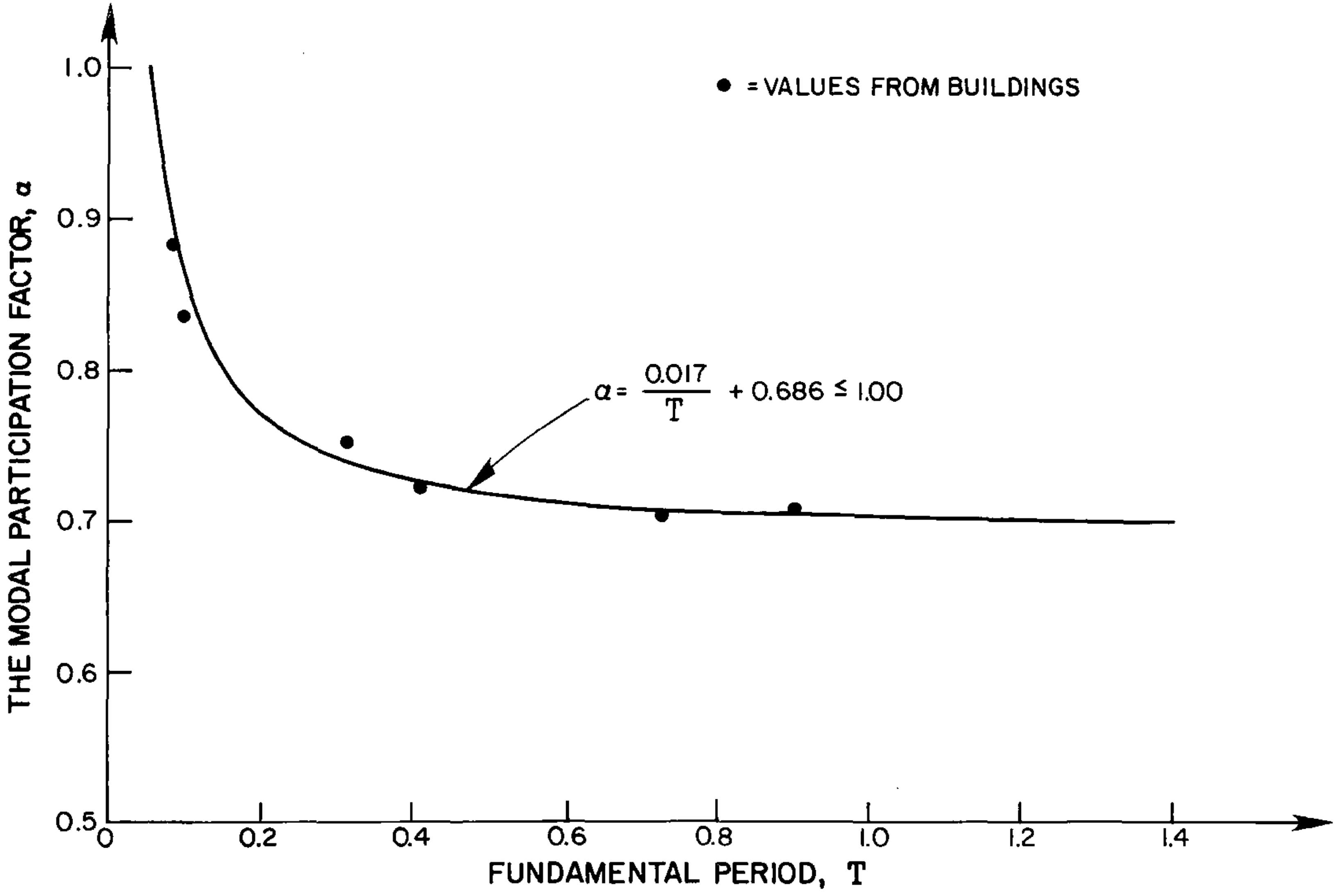

FIG. A-1 MODAL PARTICIPATION FACTOR, α

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