Enclosure 2

FACTORS INFLUENCING DEFLECTIONS IN GROUTED HOLLOW UNIT CONCRETE MASONRY WALLS

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FACTORS INFLUENCING DEFLECTIONS IN GROUTED HOLLOW UNIT CONCRETE MASONRY WALLS

i. PREFACE

This report presents the results of a brief survey on the modulus of rupture and modulus of elasticity of grouted hollow concrete unit masonry. These material properties are significant parameters in assessing the out-of-plane stiffness and deflections of the twelve-inch concrete masonry walls in the PVNGS facility near Pheonix, Arizona.

While many documents were reviewed in the course of this study, the data obtained are not exhaustive. Additional data exists which could not be obtained in the time available.

The observations herein are based upon the data examined. While additional data could influence specific numerical values, dramatic changes in basic relationships would not be expected.

ii. TABLE OF CONTENTS

1.0 INTRODUCTION

This report presents the results of a short study of out-of-plane deflections in grouted hollow unit concrete masonry. Attention was focused on grouted masonry; ungrouted masonry was not emphasized, and clay brick masonry was not considered at all because it is felt that information from tests performed on these materials is not relevant to grouted concrete masonry.

Specifically, this document reports on the basis for code specified procedures for calculating deflections. In the Uniform Building Code (Ref. 1), deflections for slender masonry walls are covered in section 2411(b) as a part of the design method for reinforced masonry slender walls (section 2411, Ref. 1). The provisions of this section were developed from a single experimental research project, the results of which are summarized in <u>Test Report on Slender Walls</u> (Ref 3). Accordingly, a significant portion of this report (section 2) concentrates on the data and conclusions presented in Ref. 3. In particular, the origin of code specified values for the modulus of rupture for concrete masonry was investigated. The sensitivity of deflection calculations to the assumed values of modulus of rupture, modulus of elasticity, and moment of inertia was also investigated.

Following the analysis of the Slender Wall Test Report in section 2.0, the basis for the value of the modulus of rupture for concrete given in the ACI code (4) is discussed in section 3.0. Other pertinent research data and test results related to the modulus of rupture of concrete masonry are collected and summarized in section 4.0. Some observations are presented in section 5.0. A limited bibliography follows in section 6.0.

2.0 DISCUSSION AND ANALYSIS OF THE LOAD-DEFORMATION BEHAVIOR OF SLENDER CONCRETE MASONRY WALLS PER THE UNIFORM BUILDING CODE

The method for calculating the mid-height deflection of a simply supported masonry wall subjected to out-of-plane lateral loads is covered in the UBC in section 2411 (b) as part of the design method for reinforced masonry slender walls. UBC section 2411(b) is reproduced in full in Appendix A. In this section of this report, basic parameters of the UBC deflection calculation and their associated assumptions will be discussed. In section 2.1, the deflection calculations in the UBC are outlined, followed by a description of the Slender Wall Test Program (Ref. 3) from which they are derived. In section 2.2, some detailed results of the Slender Wall Tests (Ref. 3) are presented and analysed in order to clarify the origin of code specified values for modulus of rupture. section 2.3 provides further analysis of the Slender Wall Tests assuming a modified effective section. In section 2.4, load-deformation curves are calculated per UBC (section 2411(b)), and compared to the original test data from the Slender Wall Tests. The sensitivity of these calculations to assumed values of modulus of rupture and modulus of elasticity are also discussed. Finally, in section 2.5, ACI and UBC methods of calculating the cracked load-deformation behavior of reinforced masonry walls are compared.

2.1 Basis for UBC Provisions for Calculation of Deflections in Slender Masonry Walls

According to the UBC, the midheight deflection of a reinforced masonry slender wall is computed by the following formula:

$$
\Delta_s = \begin{cases} \dfrac{5 M_s h^2}{48 E_m I_g} \text{ (for } M_s < M_{cr}) \text{ .... (11-12)} \\ \dfrac{5 M_{cr} h^2}{48 E_m I_g} + \dfrac{5 (M_s - M_{cr}) h^2}{48 E_m I_{cr}} \text{ (for } M_{cr} < M_s < M_n) \text{ .... (11-13)} \end{cases}
$$

WHERE:

h = height of the wall.
M_s = service moment at the midheight of the panel, including $P\Delta$ effects.
E_m = $1000 f'_m$.
I_g, I_{cr} = gross, cracked moment of inertia of the wall cross section.
M_{cr} = cracking moment strength of the masonry wall.
M_n = nominal moment strength of the masonry wall.

The equation numbers used are from the 1985 UBC (Ref. 1). The cracking moment strength of the wall is determined by the formula:

$$M_{cr} = S f_r \text{ .... (11-14)}$$

WHERE:

S = section modulus.
f_r = modulus of rupture

For concrete and masonry, the modulus of rupture, f_r, is defined by Park and Pauley (Ref. 8) as:

$$f_r = K \sqrt{f'_m} \quad \text{where } f'_m = \text{Ultimate compressive masonry stress}, \quad K = \text{Value used to relate } f_r \text{ to } f'_m \qquad (2.1)$$

The UBC lists in section 2411 (b) values for "K" for several types of masonry construction. For concrete masonry units, "K" is given as 2.5. The basis for this value is discussed in section 2.2. For concrete, the UBC has adopted the ACI 318-83 Building Code (Ref. 4) value for "K" of 7.5. The basis for the concrete value is reported in section 3 of this document.

The "Commentary" to Chapter 24 of the Uniform Building Code, 1985" published by the Masonry Society (Ref. 2) gives background

information on the masonry chapter of the UBC. It states that the values for "K" given in the UBC (and, indeed, the entire UBC reinforced masonry slender wall section, 2411) are based upon tests from a comprehensive 1982 reinforced slender wall test program sponsored by the American Concrete Institute (ACI) and the Structural Engineers Association of Southern California (SEASC) (Ref. 3).

In the Slender Wall Test Program reported in Ref. 3, thirty 24 foot tall concrete, concrete masonry, and brick masonry walls of various thicknesses were tested. The walls were laterally loaded by an airbag as well as vertically by an eccentric and relatively moderate dead load. Some test results are shown in Table 2.1. For concrete masonry, three tests were performed on each of three wall thicknesses (nominally 6, 8, and 10 inches). The load-deflection curves for each thickness of concrete masonry wall are plotted in Figs. 2.1 through 2.3, and the averages of the three curves for each thickness are plotted together in Fig. 2.4. The averaged load-deflection results from the four different thicknesses of concrete panels are given in Fig. 2.5.

2.2 Analysis of Slender Wall Test Results (Ref. 3)

In order to understand the basis for the design recommendations made by the Slender Wall Report (Ref. 3) (subsequently accepted by the UBC (Ref. 1)), it is important to determine what and why certain assumptions (for example, material properties) and conclusions (for example, "K" = 2.5 for concrete masonry) were made.

In addition to the Slender Wall Test Report, the original test data for the entire Test Program of Ref. 3 were obtained to determine the basis for the information and conclusions of that Report

(Ref. 32). An analysis of these data is described in detail below. The results of the analysis are summarized in Tables 2.3 through 2.5. An explanation of each column in Tables 2.3 through 2.5 of this document is given in notes following each table.

2.2.1 Compressive Strength and Elastic Modulus of Masonry

In the Slender Wall Test Program (Ref. 3), prisms were built for each wall thickness and tested at the age of 28 days. In addition, at the end of the program, prisms were cut from the test walls and tested at an age of over one year. The results of these two sets of prism tests are reported on page 2-3 of the Report. There was some discrepency between the results of the two tests, and hence some difficulty in selecting values of f'm and E to use in this analysis.

According to the original test data (Ref. 32), the results reported on page 2-3 of Ref. 3 were obtained from three prism tests for each wall thickness made at 28 days. The reported elastic tangent modulus was measured during one prism test of each wall type. The elastic modulus was calculated by the authors of the Slender Wall Test Report (Ref. 3) as the slope between two subjectively selected points just above the origin on the stress-strain curve. After one year, one prism was cut from each wall and compressive strength and stiffness data was taken. This data is reported in Fig. 2-4 of Ref. 3, however the data shows greater scatter than Fig. 2-4 implies. Refering to the original test data (Ref. 32), the results of each prism test are collected and reported in Table 2.2. The prism test results appear to be inconclusive as to the actual material properties of the walls tested in the ACI-

TABLE 2.1
(Ref. 3)
SLENDER WALLS TEST RESULTS

Wall No. and Type		Thick-ness, t, in.	f'_c or f'_m, psi	h/t Ratio	Vert. Load, plf	Lat. Load at f_y, psf	Defl. at Yield, in.	Max. Lat. Defl., in.	Date Tested
CMU	1	9.63	2460	30	320	94	5.5	17.1	3- 9-81
	2	9.63	2460	30	860	82	5.5	8.0	2-25-81
	3	9.63	2460	30	860	73	6.3	19.0	2-18-81
	4	7.63	2595	38	860	75	6.5	11.2	3-10-81
	5	7.63	2595	38	860	75	7.5	10.3	3-12-81
	6	7.63	2595	38	320	71	5.8	14.8	4-21-81
	7	5.63	3185	51.2	320	46	9.0	17.7	4-22-81
	8	5.63	3185	51.2	320	38	---	15.9	4-30-81
	9	5.63	3185	51.2	320	46	9.8	11.0	5- 1-81
Br	10	9.6	3060	30.3	320	94	---	15.6	4-20-81
	11	9.6	3060	30.3	320	89	9.3	16.8	4-17-81
	12	9.6	3060	30.3	320	74	9.0	14.6	5-11-81
	13	7.50	3440	38.4	320	40	12.0	19.6	5- 8-81
	14	7.50	3440	38.4	320	54	14.0	15.9	5- 7-81
	15	7.50	3440	38.4	320	66	10.5	14.8	5- 6-81
HBr	16	5.50	6243	52.4	320	57	8.0	19.3	4-15-81
	17	5.50	6243	52.4	320	48	8.2	18.2	4-16-81
	18	5.50	6243	52.4	320	55	7.9	11.1	5- 4-81
Con	19	9.50	4000	30.3	320	87	7.3	9.9	5-14-81
	20	9.50	4000	30.3	320	83	5.3	7.0	5-12-81
	21	9.50	4000	30.3	320	83	7.5	12.3	4-27-81
	22	7.25	4000	39.7	320	57	5.4	12.2	4-28-81
	23	7.25	4000	39.7	320	52	7.4	11.8	4-29-81
	24	7.25	4000	39.7	860	57	7.6	11.8	4-14-81
	25	5.75	4000	52.4	860	51	8.1	13.2	3-14-81
	26	5.75	4000	52.4	860	42	7.2	11.1	3-18-81
	27	5.75	4000	52.4	320	42	8.5	12.4	3-23-81
	28	4.75	4000	60.6	320	32	11.6	13.0	5- 5-81
	29	4.75	4000	60.6	320	34	12.6	19.2	5-15-81
	30	4.75	4000	60.6	320	34	13.1	15.2	5-14-81

Note: CMU = Concrete Masonry Unit; Br = Two-Wythe Brick
HBr = Hollow Brick; Con = Concrete

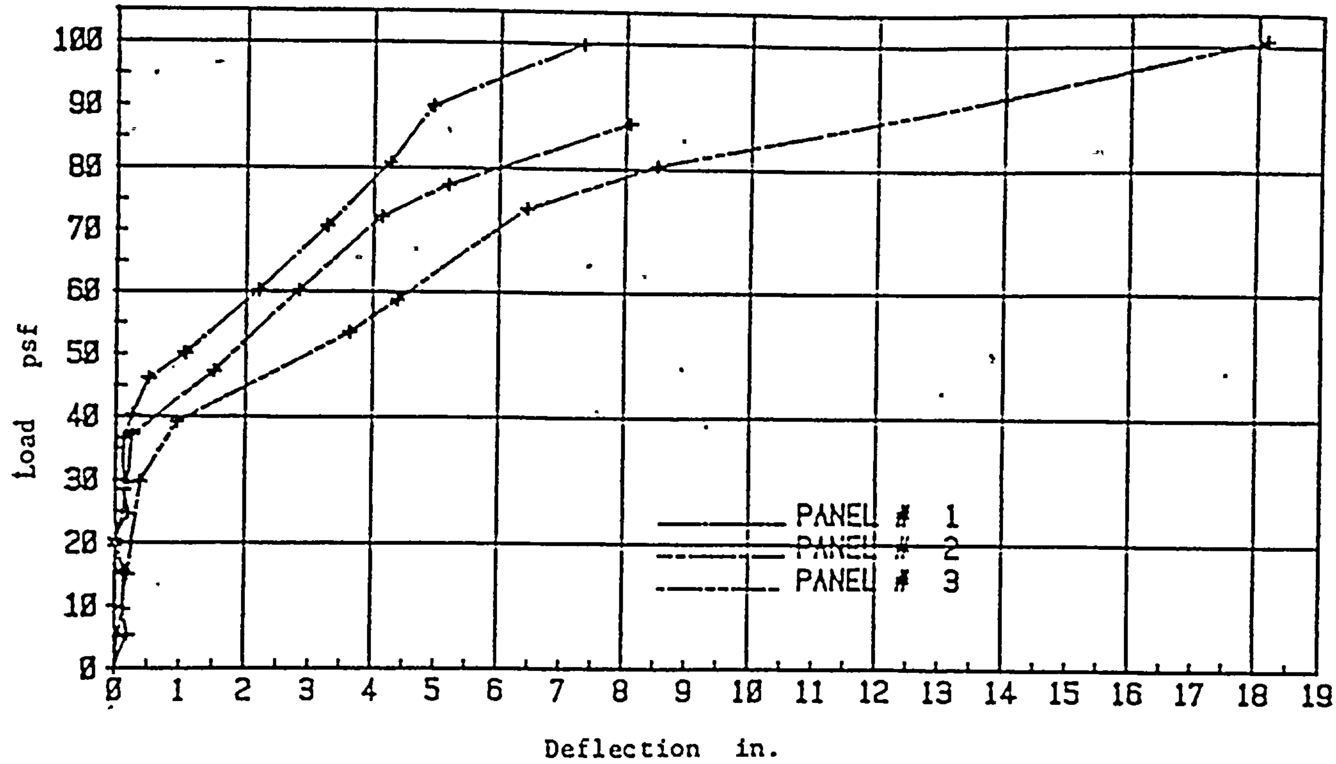


Fig. 2.1 Load - Deflection Curves, 10" Concrete Block Masonry (Ref. 3)

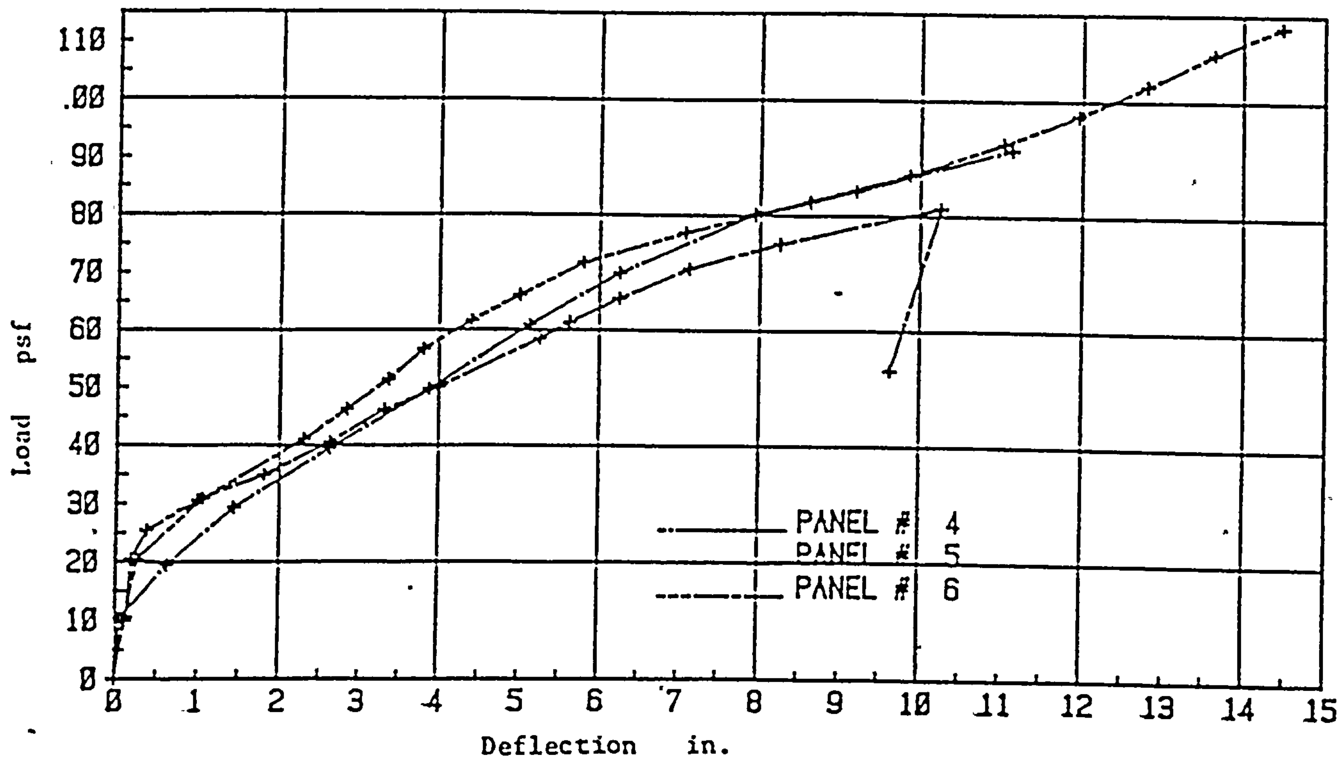


Fig. 2.2 Load - Deflection Curves, 8" Concrete Block Masonry (Ref. 3)

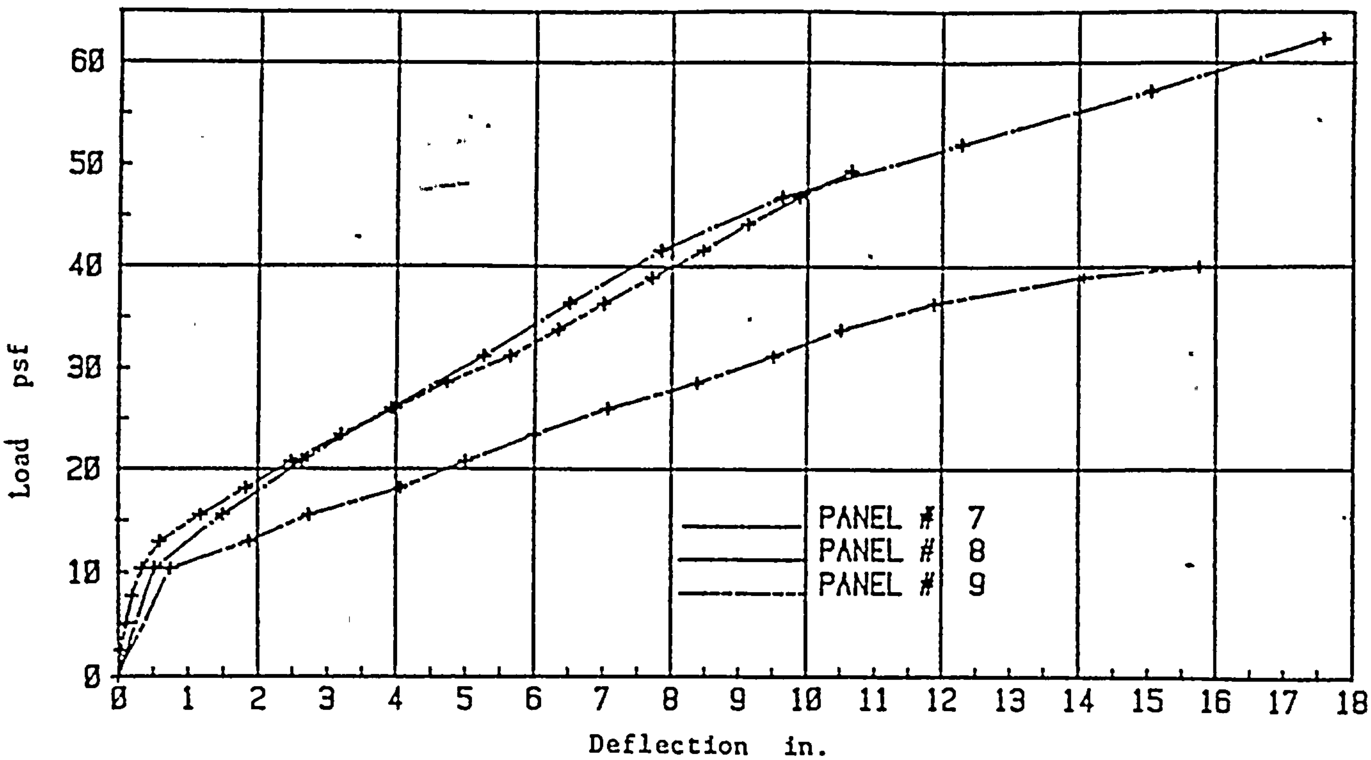


Fig. 2.3 Load - Deflection Curves, 6" Concrete Block Masonry (Ref. 3)

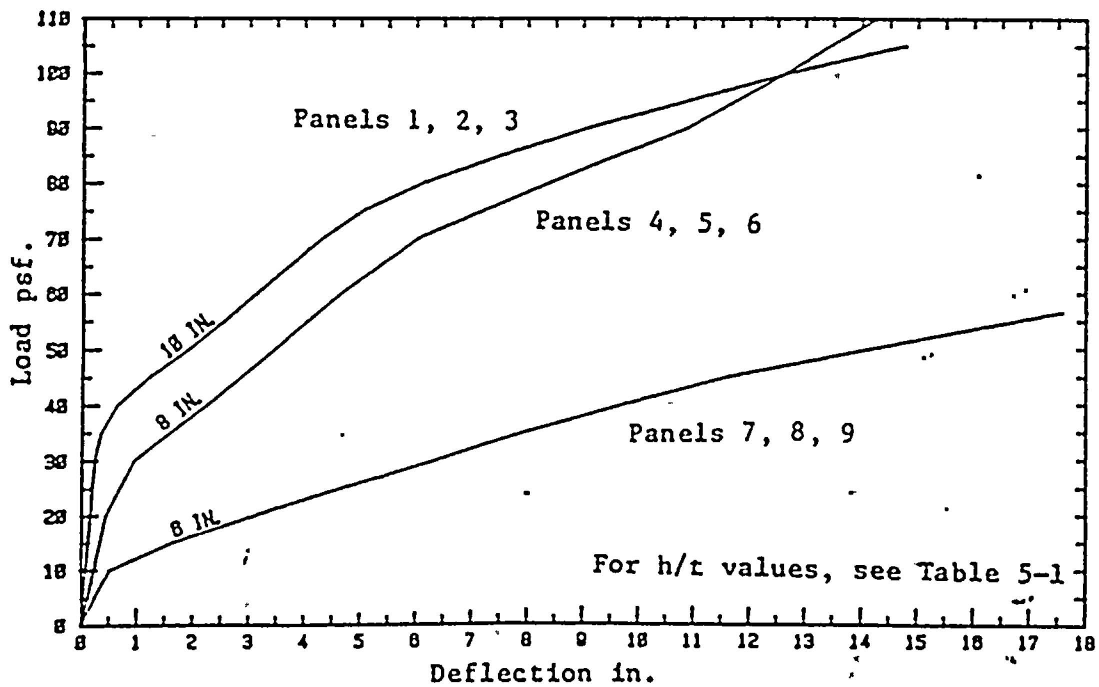


Fig. 2.4 Deflection at Mid-Height: 6", 8", 10" Concrete Block Masonry (Ref. 3)

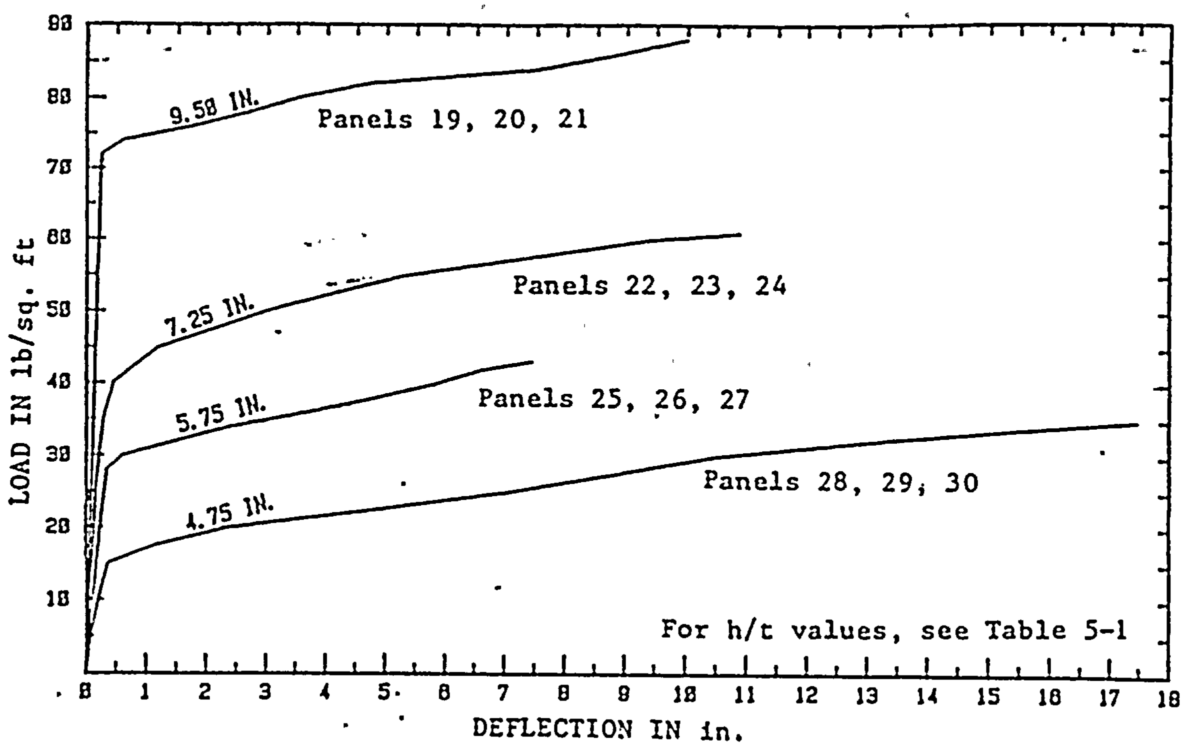


Fig. 2.5 Deflection in Inches at Mid-Height: Concrete Panels. (Ref. 3)

TABLE 2.2

PRISM TESTING DATA (REF. 32)

Wall Type	FIELD BUILT PRISMS (28 day) Comp. Strength (psi)	FIELD BUILT PRISMS (28 day) Elastic Modulus (x 10^6)	WALL CUT PRISMS (1 year) Comp. Strength (uncorrected)	WALL CUT PRISMS (1 year) Comp. Strength (corrected) (3)	WALL CUT PRISMS (1 year) Elastic Modulus (x 10^6)
6" CMU	3045		N.A.	N.A.	2.49
	3115		4600	5290	N.A.
	3395	1.59	3382	3890	1.82
AVERAGE	(3185)		(3990)	(4590)	(2.16)
8" CMU	2535		4409	4365	2.03
	2490		3914	3405	1.29
	2760	1.72	4098	3935	1.03
AVERAGE	(2595)		(4140)	(3900)	(1.45)
10" CMU	2270		4460	4330	1.75
	2530		4805	4325	1.75
	2570	2.17	3828	3560	1.28
AVERAGE	(2460)		(4365)	(4070)	(1.59)

NOTES:

(1) This table is an accumulation of information from the original test data.

(2) The elastic modulus was calculated by the authors of Reference 3 as the slope between two subjectively selected points just above the origin on the stress-strain curve.

(3) The corrected prism strengths were obtained by multiplying the uncorrected prism strengths by the appropriate h/t correction factors as per 1979 UBC. Both uncorrected and corrected values were presented in ref. 32, and are duplicated here for clarity.

TABLE 2.3

REINFORCED CONCRETE MASONRY WALL PANEL DATA (Ref. 3)

Wall Type	(1) f'_m Test (psi)	(2) Lateral load at Cracking	(3) Defl. at Cracking (in.)	(4) Cracking Moment (lb-in)	(5) f_r (from test) (psi)	(6) "K" from Test
6" CMU	3185	8.7 psf	0.7	8748	138	2.5
8" CMU	2595	26 psf	0.5	24876	214	4.2
10" CMU	2460	42 psf	0.3	38580	208	4.2

(1) The compressive strength of the actual masonry used in the test program (Ref. 3).

(2) The lateral load at initial cracking as calculated from cracking information given on Page 6-3 of Ref. 3 and yielding information given in Table 5-1 of Ref. 3.

(3) The deflection at cracking used to calculate the cracking moment in Col. (4). This value was read from Figs. 2.1 through 2.3 (Ref. 3).

(4) The initial cracking moment at mid-height was calculated including $P - \Delta$ effects as outlined in Ref. 3 (page 7-16).

(5) The modulus of rupture from the tests as calculated by UBC Eqn. 11-14 (See page 5 of this report).

(6) The value "K" as defined by Eqn. 2.1 of this report as calculated from test data.

TABLE 2.4

REINFORCED TILT-UP CONCRETE WALL PANEL DATA (Ref. 3)

Wall Thickness (in.)	(7) f'_c Test (psi)	(8) Lateral load at Cracking	(9) Defl. at Cracking (in.)	(10) Cracking Moment (lb-in)	(11) f_r (from test) (psi)	(12) "K" from Test
4.75	4000	20 psf	0.35	18340	371	5.9
5.75	4000	31 psf	0.40	29590	337	5.3
7.25	4000	39 psf	0.45	35470	447	7.0
9.50	4000	76 psf	0.20	66970	406	6.4

(7) The compressive strength of the actual concrete used in the test program (Ref. 3).

(8) The lateral load at initial cracking as calculated from cracking information given on Page 6-4 of Ref. 3 and yielding information given in Table 5-1 of Ref. 3.

(9) The deflection at cracking used to calculate the cracking moment in Col. (8). This value was read from Fig. 2.5 (Ref. 3).

(10) The initial cracking moment at mid-height was calculated including $P-\Delta$ effects as outlined in Ref. 3 (page 7-16).

(11) The modulus of rupture from the tests as calculated by UBC Eqn. 11-14 (See page 5 of this report).

(12) The value "K" as defined by Eqn. 2.1 of this report as calculated from test data.

TABLE 2.5

REINFORCED CONCRETE MASONRY WALL PANEL DATA
(with tensile face shell removed)

Wall Type	(13) Depth w/o one face shell (in)	(14) f_r (psi)	(15) "K"
6" CMU	4.375	228	4.0
8" CMU	6.375	306	6.0
10" CMU	8.125	292	5.9

(13) The thickness of the concrete masonry wall assuming one face shell has been removed. (Fig. 2.6)

(14) The modulus of rupture calculated assuming the cracking moment was resisted by the one face shell in compression and the grout core.

(15) The value for "K" using the modulus of rupture calculated in Col. (14).

SEASC test program (Ref. 3), particularly with regard to the elastic modulus. For lack of better information, the compressive strength of the walls was taken to be the average of the 28 day tests in accordance with the UBC (Ref. 1). These are the values listed in Column 1 of Table 2.3.

2.2.2 Cracking Load and Cracking Moment

The Report (Ref. 3) states (in the section in which the concrete block wall results are discussed on page 6-3) that "cracked performance [for the 10, 8, and 6 inch walls] started at approximately 50%, 35%, and 20% of the yield, respectively". Test values for the lateral load on the walls at yielding are given in Table 2.1 of this report (reprinted from Ref. 3). The cracking load was obtained by taking the specified percentage of the reported yield load, and is reported for each thickness of wall in Column 2 of Table 2.3.

The deflection at cracking can be read directly from the averaged load-deformation plot (Fig. 2.4). With this information and the lateral load at cracking (Column 2 of Table 2.3), the cracking moment, including $P-\Delta$ effects, can be calculated using Equation 11-7 from the UBC reprinted below:

$$M_u = \frac{w_u h^2}{8} + P_{ou}\frac{e}{2} + (P_{wu} + P_{ou})\,\Delta_u \qquad (11\text{-}7)$$

WHERE:

w_u = factored distributed lateral load.
h = height of the wall between points of support.
P_{wu} = factored weight of the wall tributary to the section under consideration.
Δ_u = horizontal deflection at midheight under factored load; $P\Delta$ effects shall be included in deflection calculation.
P_{ou} = factored load from tributary floor or roof loads.
e = eccentricity of P_{ou}.
P_u = axial load at midheight of wall, including tributary wall weight.

The cracking moments calculated from the slender wall tests of Ref. 3 for each concrete masonry wall thickness are shown in Column 4 of Table 2.3.

2.2.3 Modulus of Rupture ("K" values)

The values for the modulus of rupture and the resulting value for "K" are not given explicitly in the Slender Wall Test Report (Ref. 3), however, they can be derived from the text and from the load-deformation plots given in the Report. The modulus of rupture is calculated using Equation (11-14) of this document and the "K" value is calculated using equation 2.1. These values are given in Columns 5 and 6 of Table 2.3 respectively.

The method used to obtain values for "K" and the modulus of rupture for the concrete masonry walls of Ref. 3 was also used to analyze the reinforced concrete tilt-up panel test results given in that reference. (See Table 2.1) Twelve panels with four different thicknesses were tested. All had the same amount of reinforcing, and thus different steel ratios. Averaged load-deformation results are given in Fig. 2.5. Values for "K" were calculated as for concrete masonry, and the results are summarized in Table 2.4. In each case, the "K" value fell short of the ACI (Ref. 4) recommended value of 7.5.

The relationship of the compressive strength of masonry to its modulus of rupture (Equation 2.1 of this document) was probably modelled for simplicity after a similar relationship for concrete as presented in the ACI code (Ref. 4). However, the masonry relationship between modulus of rupture and compressive strength does not appear to be based on any reported statistical correlation between

the two properties. Even the original ACI equation (9-9) after which it is modeled is based on a very poor correlation, as will be seen in section 3 of this report. Apparently, a closer look at the "K" value is warranted.

2.3 Analysis of Slender Wall Tests (Ref. 3) Assuming the Face Shells Have no Tensile Strength

In some codes (Refs. 1 and 32), the tensile strength of an reinforced masonry wall is assumed to be zero. In this spirit, the Slender Walls of Ref. 3 were again analyzed as described above assuming the face shells of the concrete masonry units carry no tensile stresses. Basically, the modulus of rupture is recalculated based on the section properties of a wall without one face shell as shown in Fig. 2.6. The assumption is made with this analysis that, before the wall section is fully cracked, the tensile force required to resist bending is provided primarily by the grout. The modulus of rupture and "K" values resulting from this analysis is summarized in Table 2.5.

The values for the modulus of rupture resulting from the analysis of the concrete masonry assuming the face shells resist no tension are given in Col. 14. These values are similar to tensile strength values reported in the literature for grout. (The compressive strength of the grout used in the slender wall test program of Ref. 3 was 3106 psi. Ref. 18 reports the tensile strength of grout is about 9% of its compressive strength, while Ref. 23 reports 8%.)

The values for "K" resulting from the analysis of the concrete masonry using modified section properties (given in Col. 15 of Table 2.5) are more aligned with values from the concrete walls. This is illustrated in Fig. 2.7 which plots the relationship of the "K"

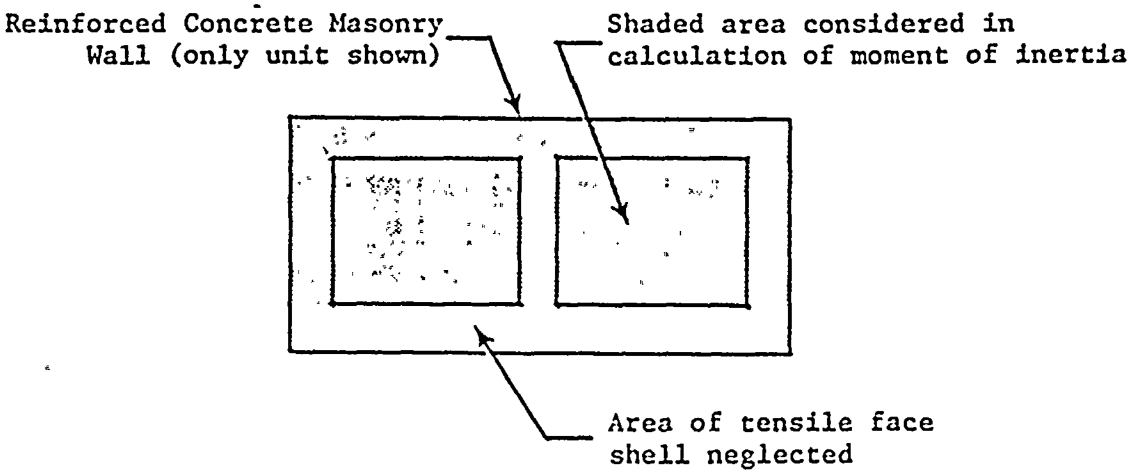
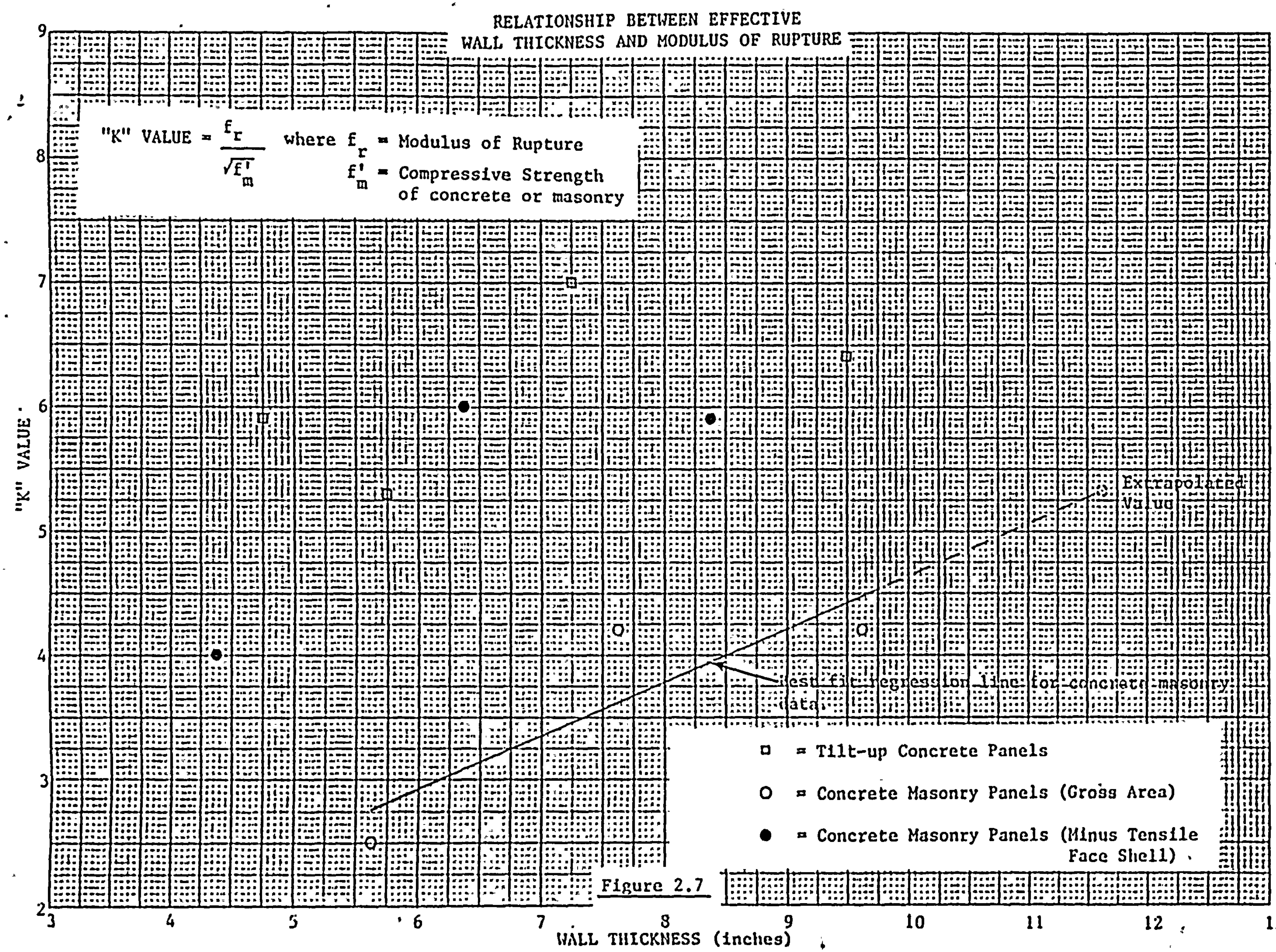


Figure 2.6
Calculation of Moment of Inertia
Neglecting Face Shell in Tension


RELATIONSHIP BETWEEN EFFECTIVE
WALL THICKNESS AND MODULUS OF RUPTURE
"K" VALUE = $\frac{f_r}{\sqrt{f'_m}}$ where f_r = Modulus of Rupture
f'_m = Compressive Strength of concrete or masonry
Extrapolated Value
Best fit regression line for concrete masonry data
□ = Tilt-up Concrete Panels
○ = Concrete Masonry Panels (Gross Area)
● = Concrete Masonry Panels (Minus Tensile Face Shell)
"K" VALUE
WALL THICKNESS (inches)
2
3
4
5
6
7
8
9
10
11
12
13
Figure 2.7

value to the effective thickness of the concrete or concrete masonry walls used in the slender wall test (Ref. 3). The "K" value-wall thickness relation for concrete masonry based on the gross area is also given in Fig. 2.7 along with an extrapolated value for 12 inch concrete masonry.

2.4 Comparison of UBC Slender Wall Load-Deformation Predictions to the Tests From Which They Were Derived

Except for the "K" value calculated for the 6" concrete masonry walls, "K" values calculated from the Test data (Ref. 3) are higher than the value recommended by the UBC. This is because the UBC recommended "K" value for concrete masonry of 2.5 was derived solely from the three 6" concrete masonry wall tests of Ref. 3. Thicker walls (with proportionately more grout) were not included. Figure 2.8 shows analytical load-deformation curves for K = 2 and K = 3 superimposed upon the 6" concrete masonry wall test data from Ref. 3. It is not clear from the text of Ref. 3 or the original test data (Ref. 32) which values for the compressive strength or the elastic modulus were used in the development of the curves in Fig. 2.8. It is apparent, after graphical interperataion, that the value of modulus of elasticity used in Fig. 2.8 was approximately equal to the test value from the wall-cut prisms (2160 ksi).

Since the material properties used in the development of the curves in Fig. 2.8 are not clearly defined, and the results obtained from 28 day tests in Ref. 3 were dissimilar to those obtained from prisms cut from the walls themselves, particularly the values for the elastic modulus, the influence of these properties on the load-deformation curves of Ref. 3 (and thusly the slender masonry wall design equations of the UBC) was investigated.

To develop the predicted load-deformation curves shown in Fig. 2.8, an iterative procedure must be used, because, when P $-\Delta$ effects

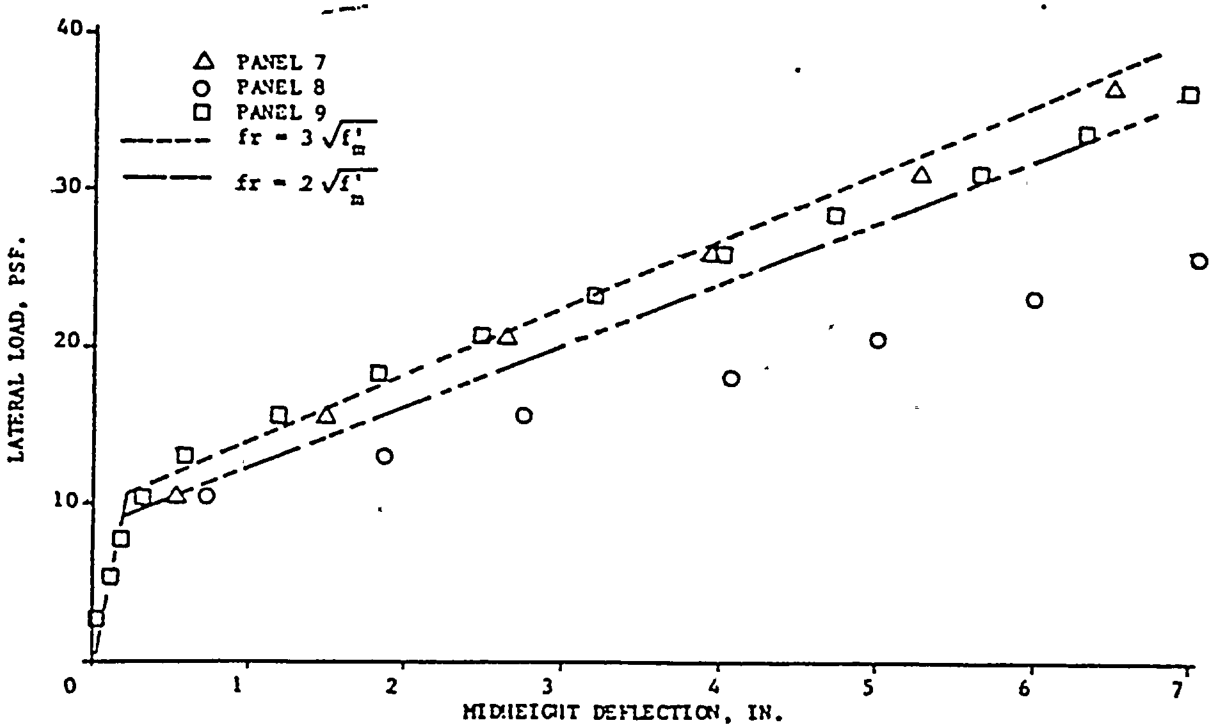


Figure 2.8

Calculated Load-Deflection Relations for 6" Concrete Masonry (Ref. 3)

are included, the moment and resulting deflection are interdependent. This procedure is outlined in section 7.4.3 of the slender wall test report (Ref. 3). A short computer program was written to perform this iterative procedure. Material properties required to calculate the load-deformation are the elastic modulus and the compressive strength of the masonry (required to calculate the modulus of rupture per Equation 2-1 of this report). The modulus of rupture (as represented by the "K" value) influences the point of cracking on the load-deformation curve. (The point of cracking is the knee in the load-deformation curve just after the inital elastic portion.) If the modulus of rupture (or the value for "K") increases, the load required to crack the wall will increase. The elastic modulus affects the slope of the load-deformation curve both before and after cracking. If the elastic modulus increases, both the initial elastic load-deformation slope and the cracked slope will be stiffer.

In Fig. 2.9, the predicted load-deformation curve for 6" concrete masonry walls are plotted using the value for the elastic modulus obtained in the 28-day prism test (See Table 2.2 of this report) and K = 2.5. The value for the cracked moment of inertia was taken from Table 7-2 of Ref. 3. Agreement is quite good. Figs. 2.10 and 2.11 show the predicted load-deformation results for 8" and 10" concrete masonry, respectively, using the same material properties specified above. Also included are the test results from the Slender Wall Test Program (Ref. 3). The UBC prescribed "K" values are too low to accurately predict the point of cracking for the 8" and 10" concrete masonry walls. From the calculations of Table 2.3, a more reasonable "K" value for both the 8" and 10" walls would be

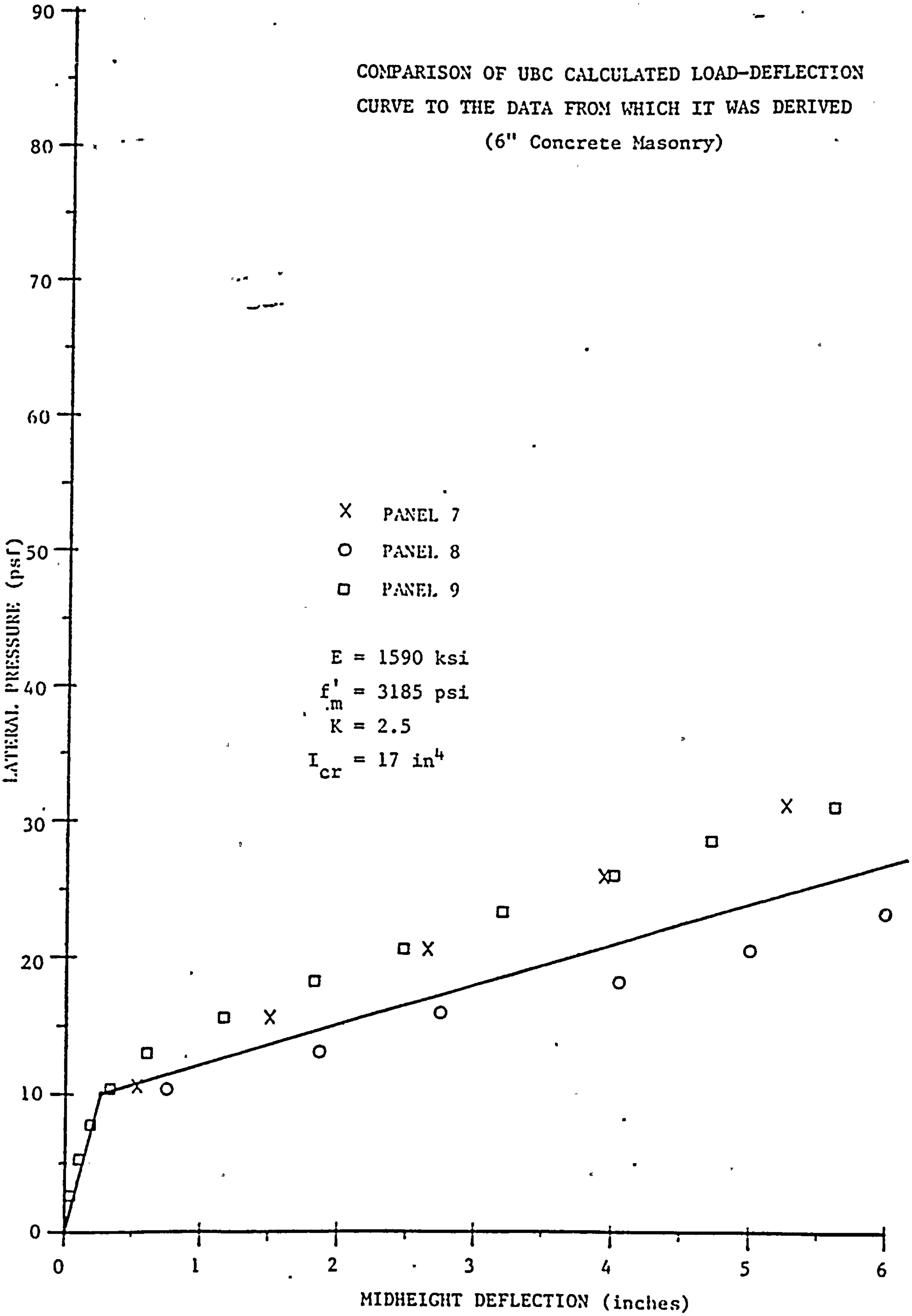
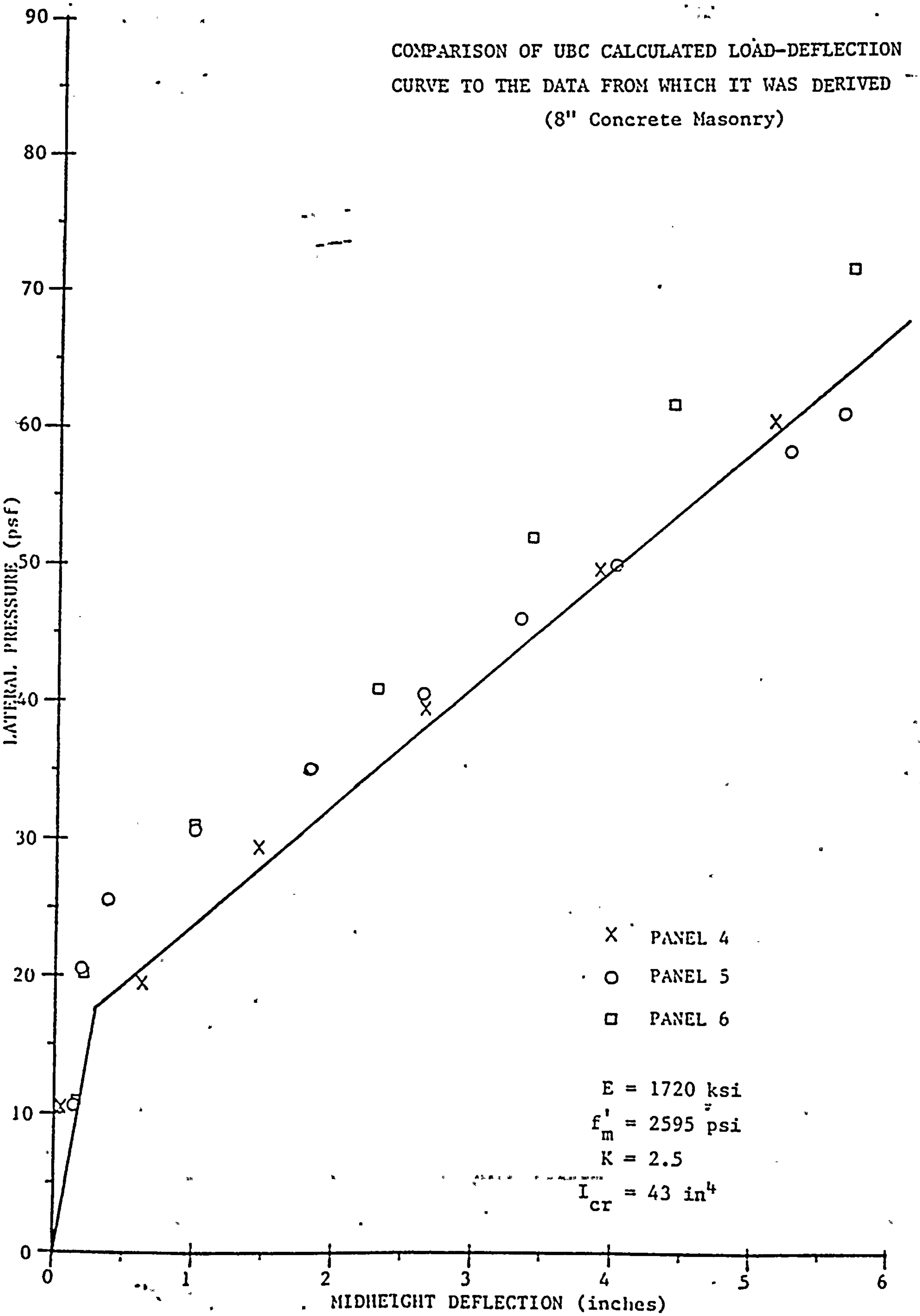

COMPARISON OF UBC CALCULATED LOAD-DEFLECTION
CURVE TO THE DATA FROM WHICH IT WAS DERIVED
(6" Concrete Masonry)
X PANEL 7
O PANEL 8
□ PANEL 9
E = 1590 ksi
f'm = 3185 psi
K = 2.5
Icr = 17 in4
LATERAL PRESSURE (psf)
0
10
20
30
40
50
60
70
80
90
MIDHEIGHT DEFLECTION (inches)
0
1
2
3
4
5
6

Figure 2.9



COMPARISON OF UBC CALCULATED LOAD-DEFLECTION
CURVE TO THE DATA FROM WHICH IT WAS DERIVED
(8" Concrete Masonry)
LATERAL PRESSURE (psf)
MIDHEIGHT DEFLECTION (inches)
0
10
20
30
40
50
60
70
80
90
1
2
3
4
5
6
X PANEL 4
O PANEL 5
□ PANEL 6
E = 1720 ksi
f'm = 2595 psi
K = 2.5
Icr = 43 in4

Figure 2.10

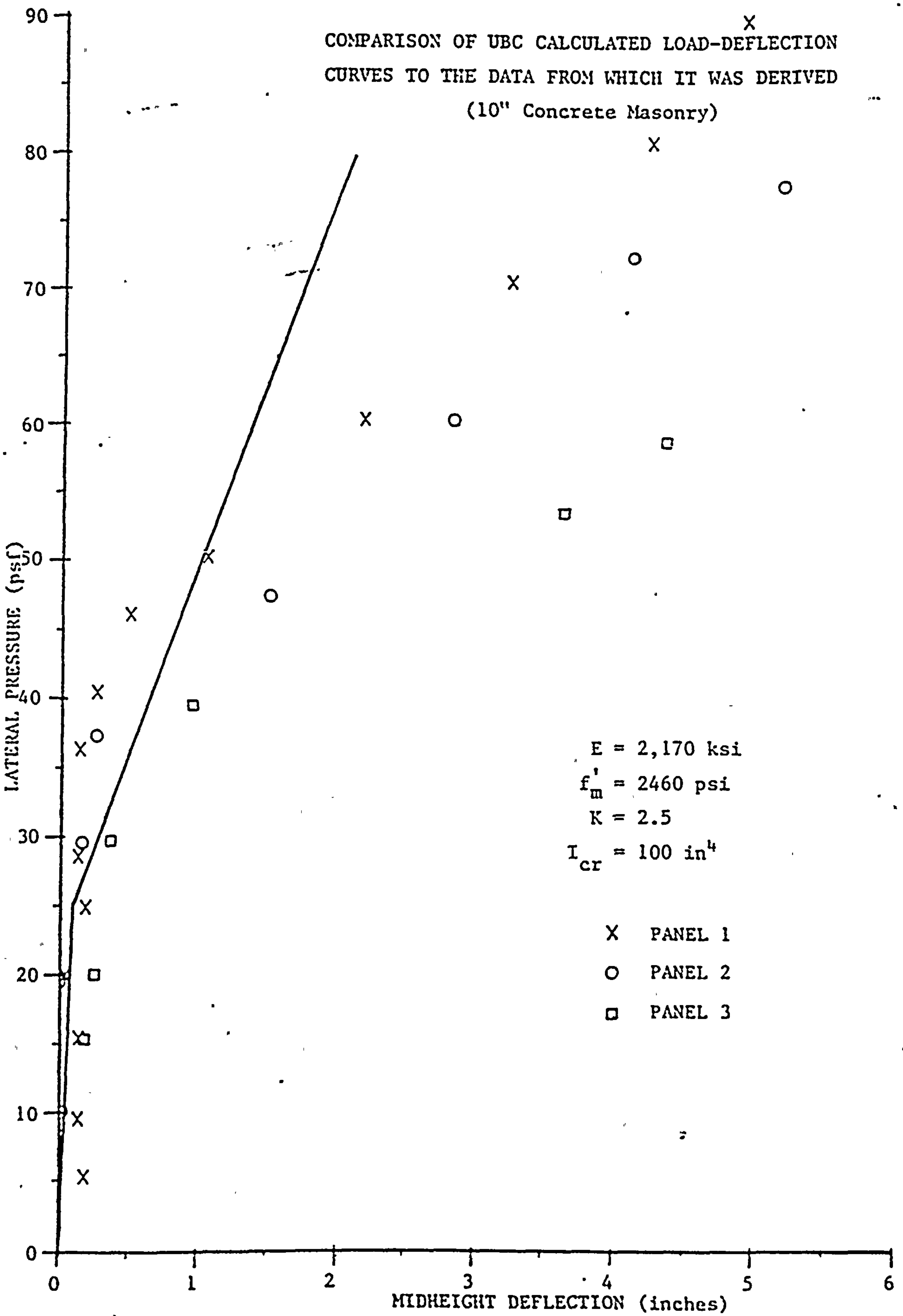

COMPARISON OF UBC CALCULATED LOAD-DEFLECTION
CURVES TO THE DATA FROM WHICH IT WAS DERIVED
(10" Concrete Masonry)
LATERAL PRESSURE (psf)
MIDHEIGHT DEFLECTION (inches)
E = 2,170 ksi
f'm = 2460 psi
K = 2.5
Icr = 100 in4
PANEL 1
PANEL 2
PANEL 3
0
10
20
30
40
50
60
70
80
90
1
2
3
4
5
6

Figure 2.11

4.2. For the 10" wall, the elastic modulus obtained from the 28 day prism tests is much higher than the modulus measured from the wall-cut prisms. The curve shown in Figure 2.11 is based on the elastic modulus from the 28-day prism tests, and the resulting load-deformation curve under estimates the measured deflections in the 10" Slender Wall tests. (See also section 2.4.2 and Fig. 2.15.)

2.4.1 Effect of the Modulus of Rupture on the Predicted Load-Deformation Curves

The value of the modulus of rupture controls the load which can be sustained before cracking occurs. The influence of the modulus of rupture (as represented in Eqn. 2.1 of this report by the value "K") is illustrated in Figure 2.12 in which a 68% increase in the "K" values resulted in a 68% increase in the load required to crack the wall. The slope of the load-deformation curve, both before and after cracking, is not dependent on the value of "K".

2.4.2 Effect of the Elastic Modulus on the Predicted Load-Deformation Curves

To determine the influence that the assumption for the elastic modulus has on the predicted load-deformation response of slender masonry walls per the UBC, the elastic modulus was varied while all other parameters were held constant. The resulting load-deformation plots for 6", 8", and 10" concrete masonry are plotted in Figs. 2.13 through 2.15 respectively. It would appear from these figures that the average wall cut prism elastic modulus values predict the load-deformation behavior of the walls more accurately than the 28 day prism values.

One could also conclude from this test data that the UBC relationship stating that the elastic modulus is 1000 times the compressive strength of the masonry provides an upper bound for the

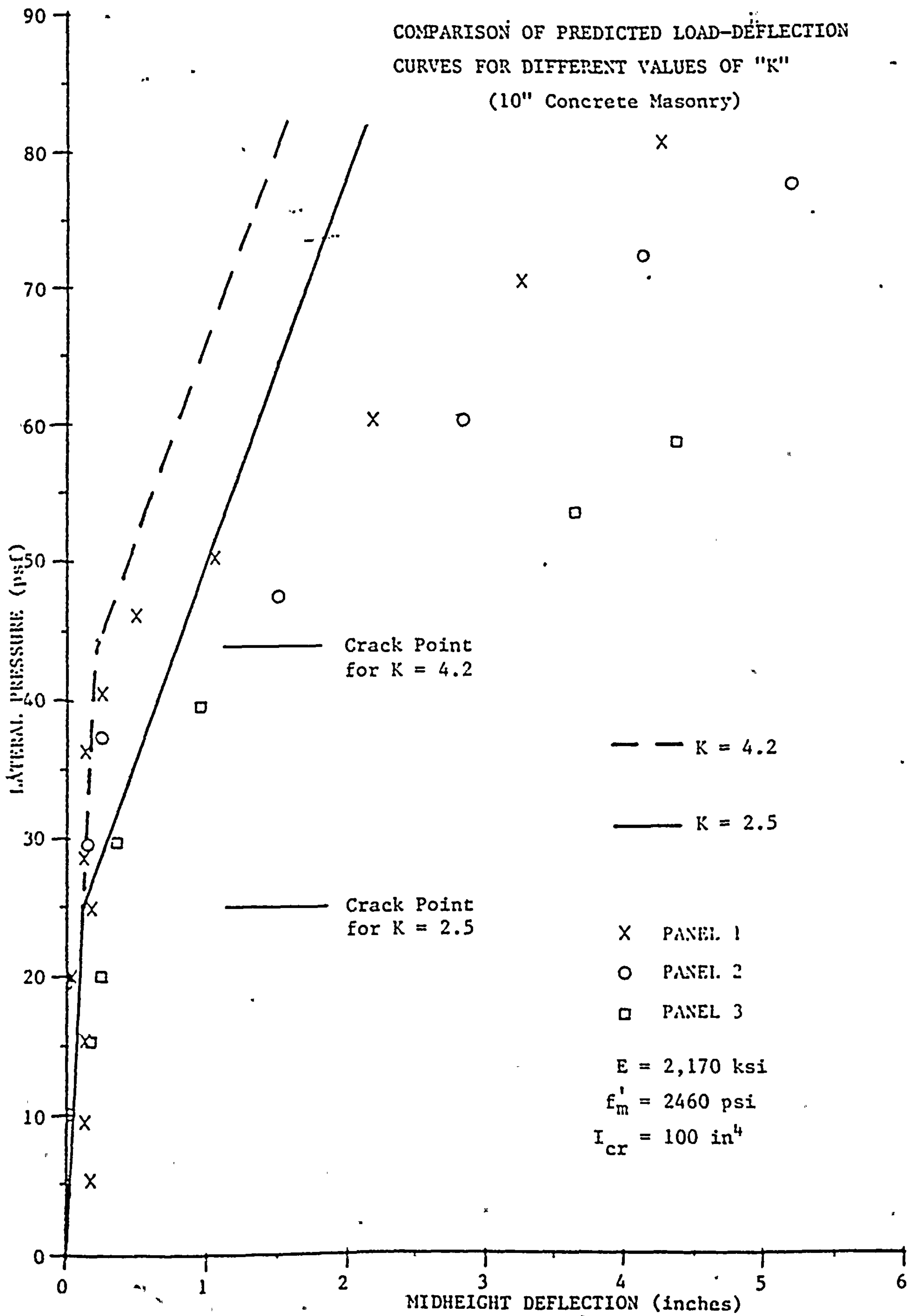
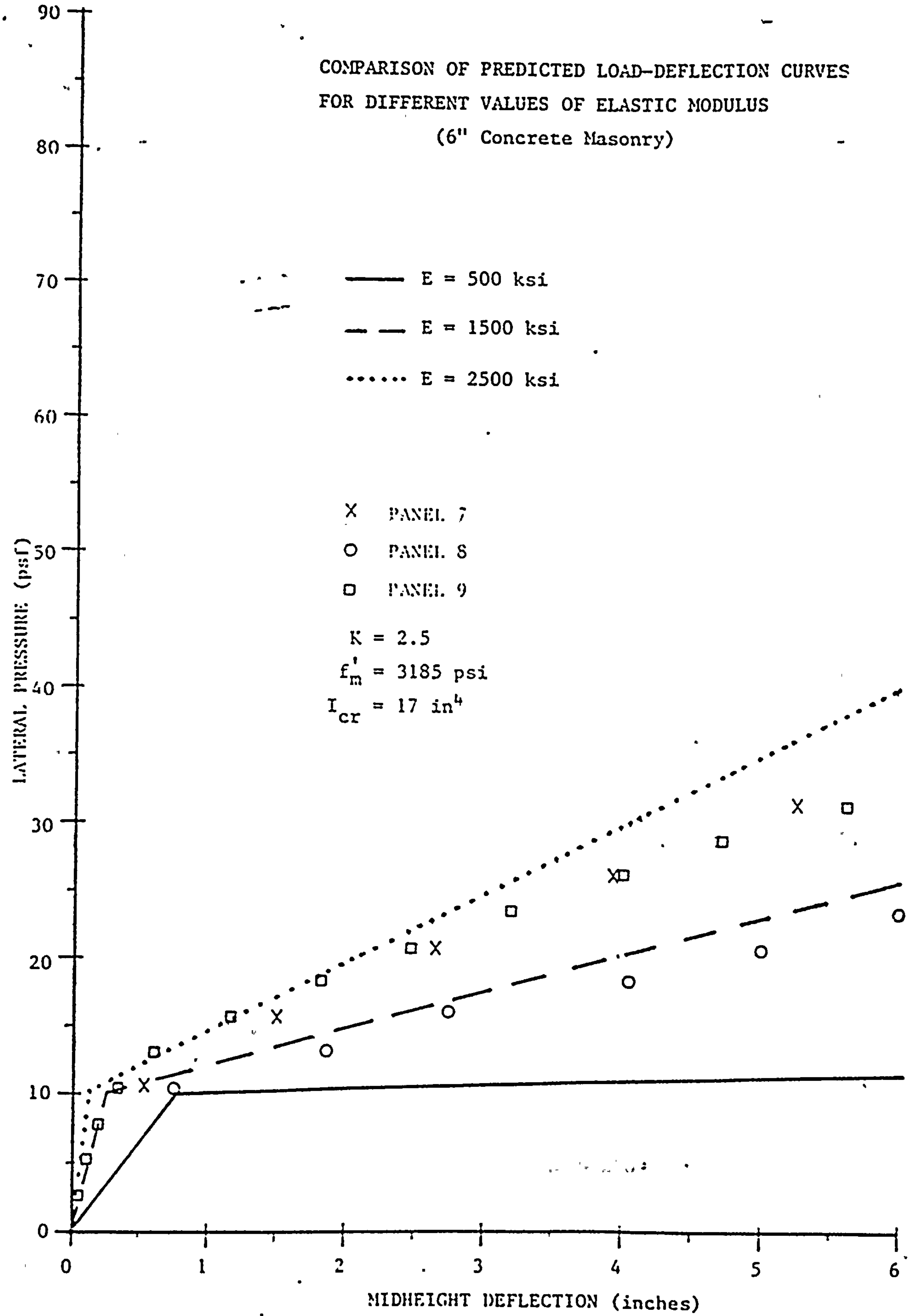

COMPARISON OF PREDICTED LOAD-DEFLECTION
CURVES FOR DIFFERENT VALUES OF "K"
(10" Concrete Masonry)
LATERAL PRESSURE (psf)
90
80
70
60
50
40
30
20
10
0
Crack Point
for K = 4.2
Crack Point
for K = 2.5
K = 4.2
K = 2.5
PANEL 1
PANEL 2
PANEL 3
E = 2,170 ksi
f'_m = 2460 psi
I_{cr} = 100 in^4
0
1
2
3
4
5
6
MIDHEIGHT DEFLECTION (inches)

Figure 2.12



COMPARISON OF PREDICTED LOAD-DEFLECTION CURVES
FOR DIFFERENT VALUES OF ELASTIC MODULUS
(6" Concrete Masonry)
E = 500 ksi
E = 1500 ksi
E = 2500 ksi
X PANEL 7
O PANEL 8
□ PANEL 9
K = 2.5
f'm = 3185 psi
Icr = 17 in4
LATERAL PRESSURE (psf)
MIDHEIGHT DEFLECTION (inches)
0
10
20
30
40
50
60
70
80
90
1
2
3
4
5
6

Figure 2.13

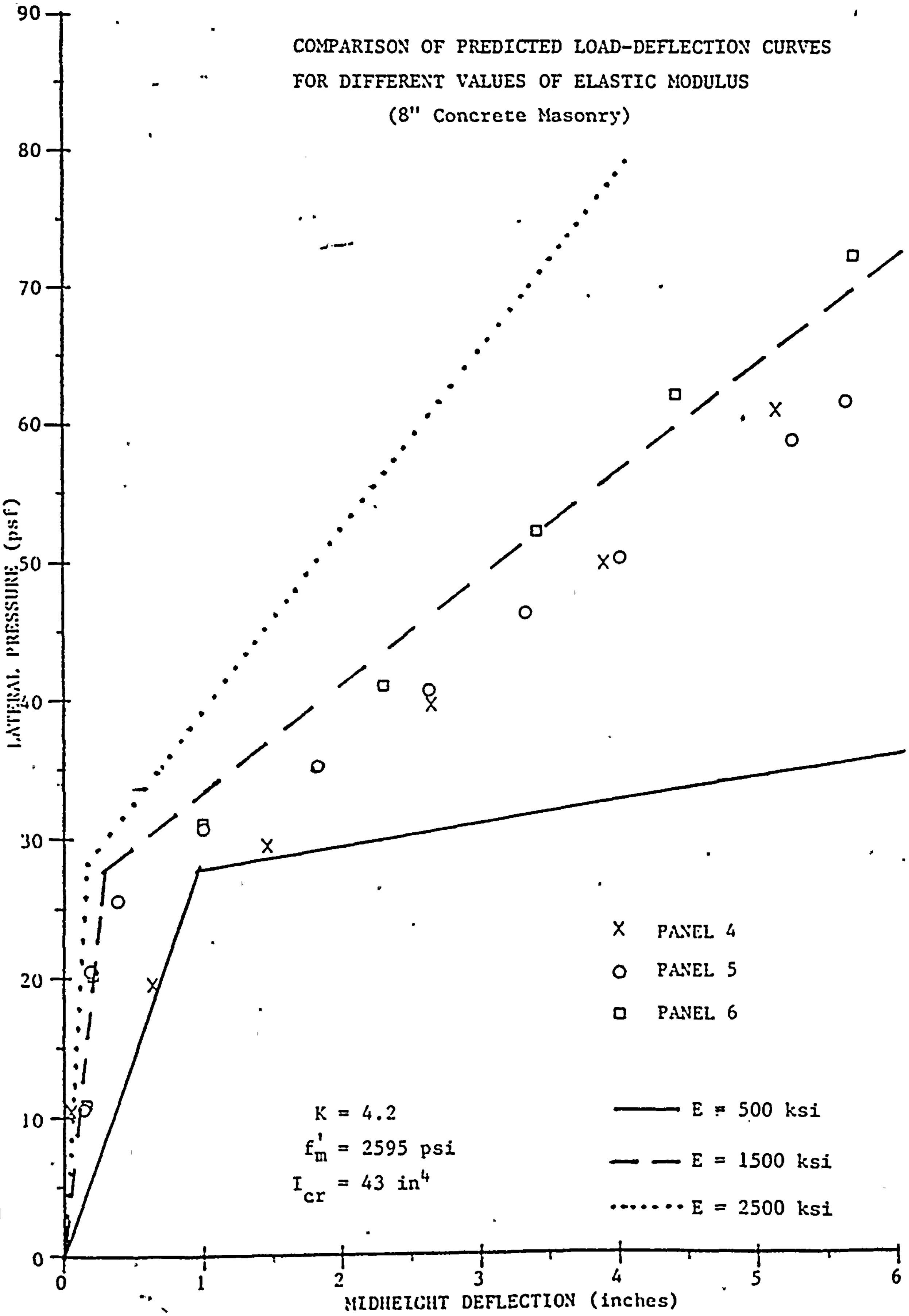
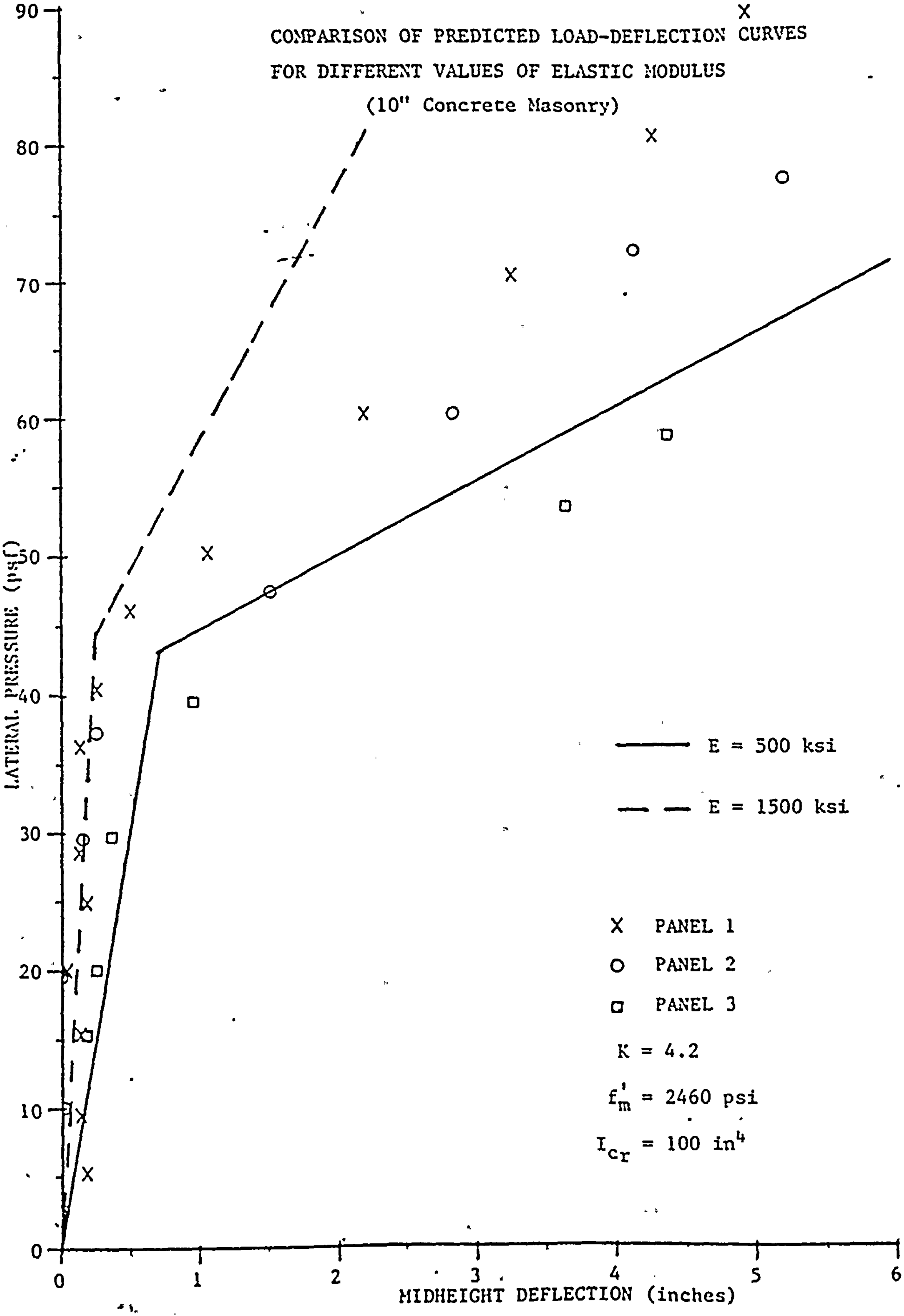

COMPARISON OF PREDICTED LOAD-DEFLECTION CURVES
FOR DIFFERENT VALUES OF ELASTIC MODULUS
(8" Concrete Masonry)
LATERAL PRESSURE (psf)
90
80
70
60
50
40
30
20
10
0
1
2
3
4
5
6
MIDHEIGHT DEFLECTION (inches)
X PANEL 4
O PANEL 5
□ PANEL 6
K = 4.2
f'm = 2595 psi
Icr = 43 in4
E = 500 ksi
E = 1500 ksi
E = 2500 ksi

Figure 2.14



COMPARISON OF PREDICTED LOAD-DEFLECTION CURVES
FOR DIFFERENT VALUES OF ELASTIC MODULUS
(10" Concrete Masonry)
LATERAL PRESSURE (psf)
MIDHEIGHT DEFLECTION (inches)
0
10
20
30
40
50
60
70
80
90
0
1
2
3
4
5
6
E = 500 ksi
E = 1500 ksi
X PANEL 1
O PANEL 2
□ PANEL 3
K = 4.2
f'_m = 2460 psi
I_{cr} = 100 in^4

Figure 2.15

measured value of elastic modulus. A report from Atkinson and Kingsley (Ref. 33) reported data from twenty compression tests on grouted concrete masonry. For prism compressive strengths ranging from 3037 psi to 4084 psi, the ratio of the elastic modulus to the compressive strength varied from 502 to 646. It should be pointed out, however that the elastic modulus as calculated from a stress-strain curve is highly sensitive to the method by which it is calculated. The elastic modulus values were calculated by Atkinson and Kingsley as a secant modulus to 50% of the compressive strength. Values for the elastic modulus calculated as an initial tangent modulus can be much higher.

2.5 Comparison of ACI and UBC Methods of Calculating the Cracked Load-Deformation Behavior

For purposes of calculating deflections, both the ACI and UBC codes call for using the gross area of a concrete or masonry wall when calculating the uncracked moment of inertia. After the wall has cracked, the method for calculating the wall deflection of each code differs. Figures 2.16 and 2.17 illustrate the effect of each method on the theoretical code-calculated load deflection curves for 6" and 10" concrete masonry walls. The experimental values from the Slender Wall Test Program (Ref. 3) are also included for comparison.

The UBC states in section 2411 (b) 4 that after cracking, the moment of inertia immediately changes from the uncracked to the cracked value. This results in a bilinear load-deflection curve for the walls as seen in Figures 2.16 and 2.17.

The ACI code (Ref. 4) uses Eqn. 9-7 (shown below) to calculate an effective moment of inertia which replaces the elastic moment of inertia in the deflection calculation.

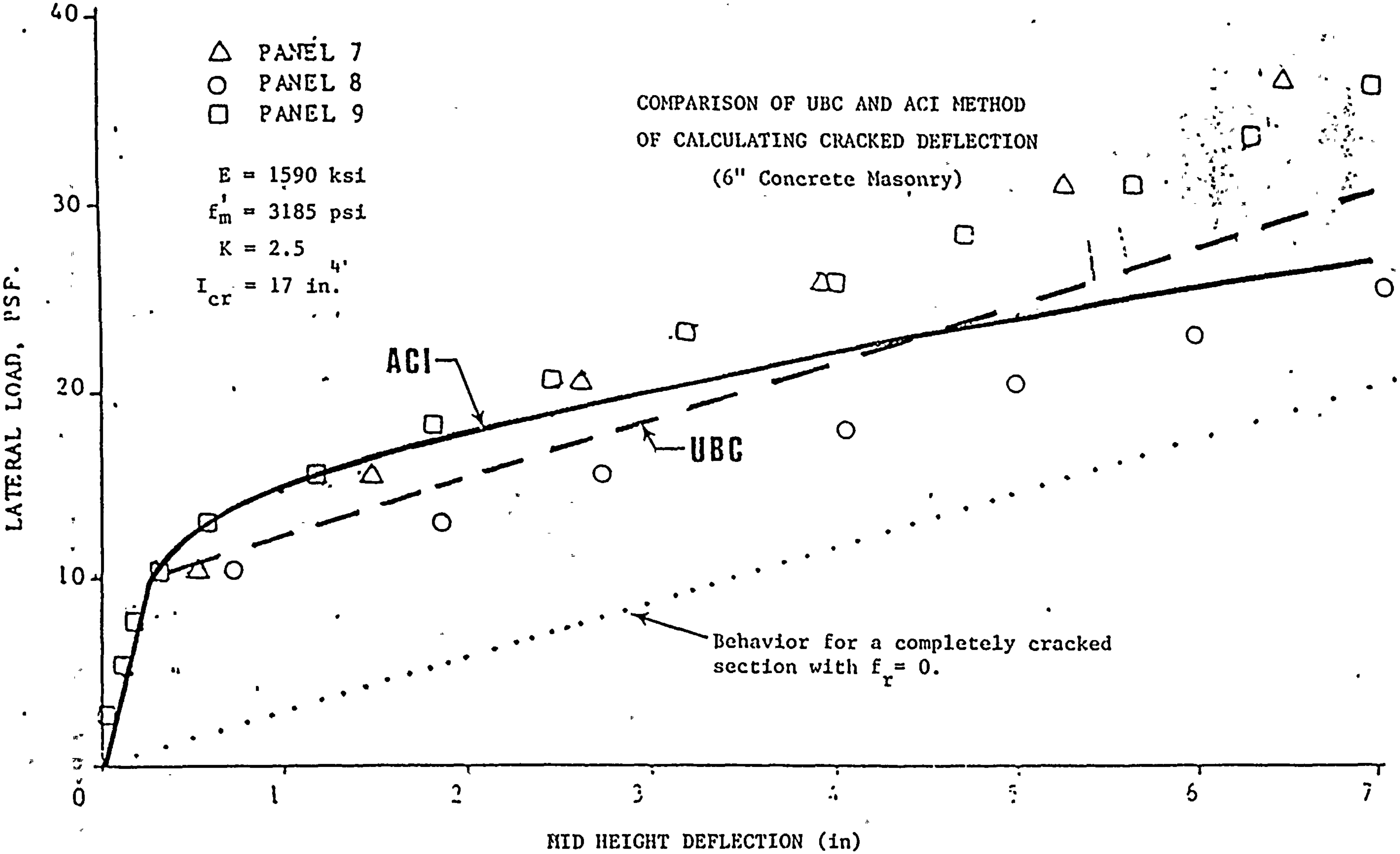

PANEL 7
PANEL 8
PANEL 9
E = 1590 ksi
f'm = 3185 psi
K = 2.5
Icr = 17 in.4
COMPARISON OF UBC AND ACI METHOD
OF CALCULATING CRACKED DEFLECTION
(6" Concrete Masonry)
ACI
UBC
Behavior for a completely cracked
section with fr = 0.
LATERAL LOAD, PSF.
40
30
20
10
0
1
2
3
4
5
6
7
MID HEIGHT DEFLECTION (in)

Figure 2.16

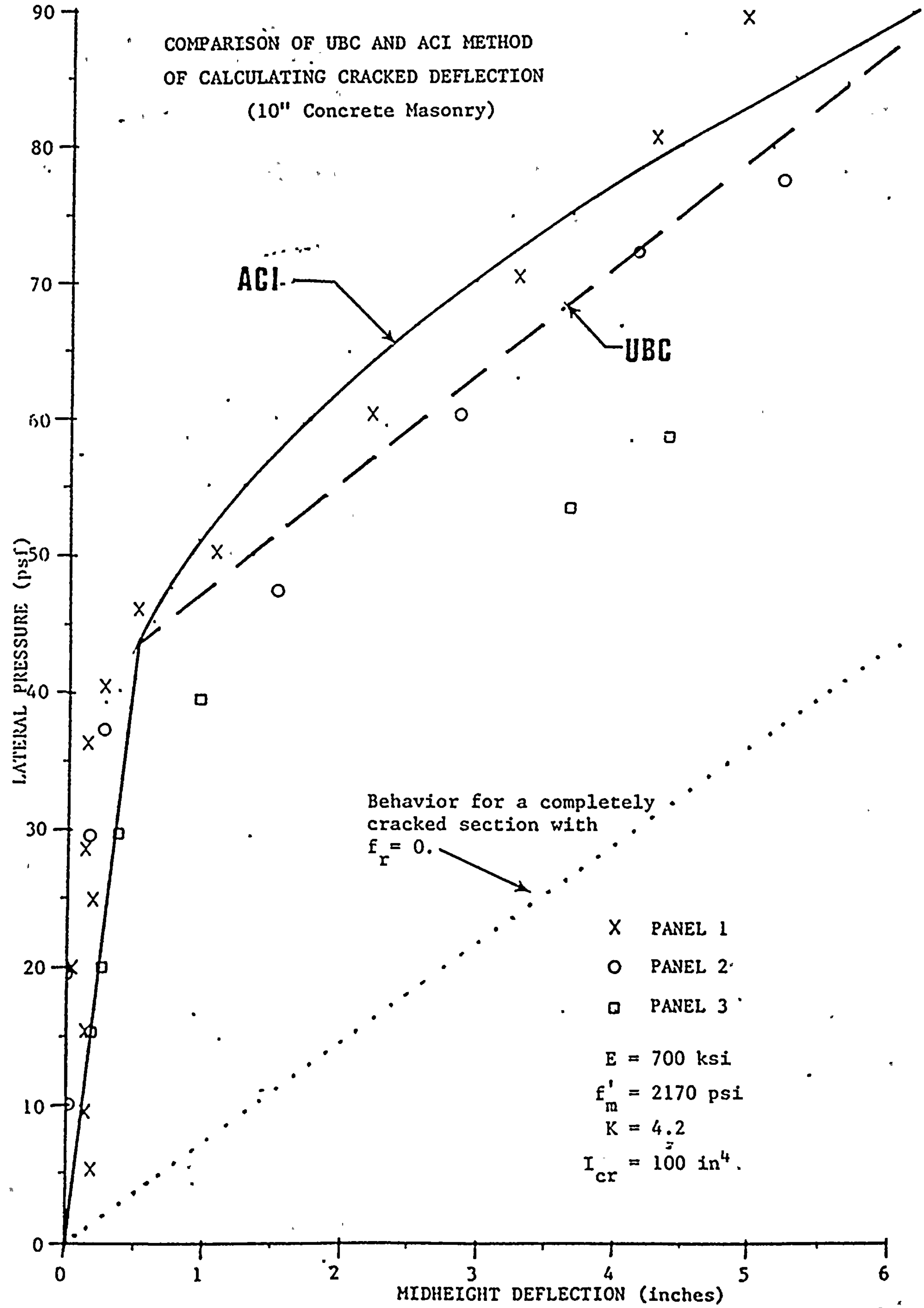

COMPARISON OF UBC AND ACI METHOD
OF CALCULATING CRACKED DEFLECTION
(10" Concrete Masonry)
ACI
UBC
Behavior for a completely
cracked section with
$f_r = 0.$
X PANEL 1
O PANEL 2
□ PANEL 3
$E = 700$ ksi
$f'_m = 2170$ psi
$K = 4.2$
$I_{cr} = 100\ in^4$
LATERAL PRESSURE (psf)
MIDHEIGHT DEFLECTION (inches)
0
10
20
30
40
50
60
70
80
90
1
2
3
4
5
6

Figure 2.17

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \quad \text{.............. (9-7)}$$

This results in a gradual transition from the uncracked to the cracked value as seen in Figure 2.16 and 2.17. As the moment gets much larger than the cracked moment, the load-deflection curve asymptotically approaches the line, (shown as a dotted line in Figures 2.16 and 2.17), representing the behavior of a wall with no tensile strength, (f_r = 0).

For deflections in the range of usual engineering interest, it appears that both methods provide reasonably accurate predictions of the load-deformation data reported in the Slender Wall Test Report.

3.0 DISCUSSION OF THE MODULUS OF RUPTURE OF CONCRETE PER THE ACI 318-83 CONCRETE CODE

In section 9.5.2.3 of the ACI code (4), equation 9-9 gives a value for the modulus of rupture of normal weight concrete for use in calculating deflections in nonprestressed one-way construction. Equation 9-9 states:

$$f_r = 7.5 \sqrt{f'c} \quad (f'c \text{ in psi})$$

Other values are given for lightweight concrete. While the modulus of rupture of concrete is not a function of compressive strength alone, the value in Equation 9-9 is widely accepted for purposes of design (5,6,7,8,9,10).

Several different values of this equation using different K values have been suggested. Most appear to be based on the data of Branson. Branson (6), (citing references 12 and 13), recommends the following equation for f , which includes the concrete weight, w:

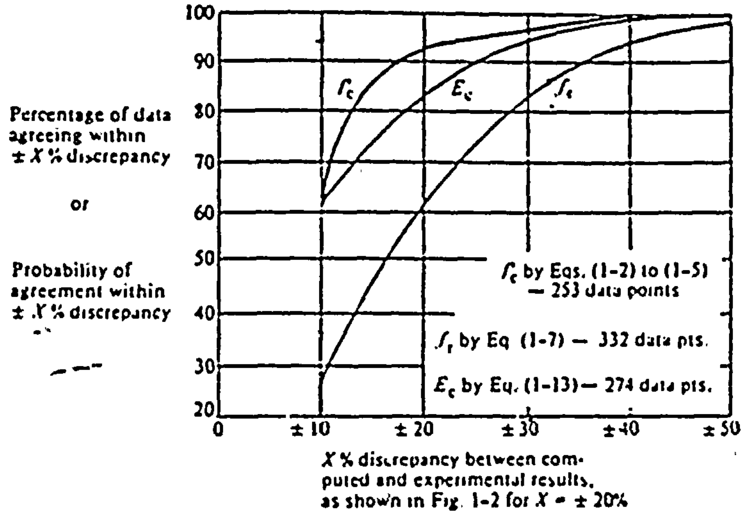
$$f_r = 0.65 \sqrt{w\ f'c} \quad (w \text{ in pcf, } f'c \text{ in psi})$$

For normal weight concrete, (w=145 pcf), this gives:

$$f_r = 7.84 \sqrt{f'c} \quad (f'c \text{ in psi}) \qquad \text{(Eq. 3.2)}$$

The value of K = 7.84 results in values of f_r 4%-5% greater than equation 9-9 (in which K = 7.5). Figures 3.1, 3.2, and 3.3, (from ref. 6), compare Equation 3.2 to results of 332 tests on various weight concretes (11). In Figure 3.3, the data scatter is considerable, and the correlation to equation 3.2 is very poor. For example, from Figure 3.2, only 27% of the test data fell within +- 10% of the computed results using equation 3.2. Note also that Equation 3.2 represents a mean value, and not a lower bound of test results.

It is relevant to note that the K values listed in Table 2.2,



Curves showing the probability of agreement with experimental data involving different weight conc for calculated results of f_c', f_t, and E_c.

Figure 3.1

Tabulated values from Fig. 1-3 showing the probability of agreement with experimental data involving different weight concrete, for calculated results of f_c', f_t, and E_c

	Percentage of data agreeing within ± X% discrepancy or Probability of agreement within ± X% discrepancy		
± X% discrepancy	f_c' by Eqs. (1-2) to (1-5)—253 data pts.	f_t by Eq. (1-7) —332 data pts.	E_c by Eq. (1-13) —274 data pts.
±10%	62%	27%	62%
±20%	93%	61%	83%
±30%	97%	83%	95%
±40%	100%	94%	99%
±50%	100%	98%	100%

Figure 3.2

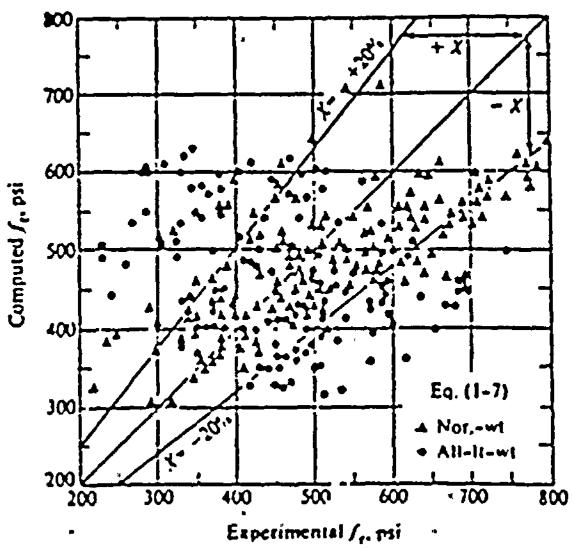


Figure 3.3

column 16 (of this report) for concrete walls vary from 4.9 to 5.8. These are significantly lower than the value K=7.5 recommended by the ACI 318-83 code (4).

4.0 ADDITIONAL PERTINENT DATA ON MODULUS OF RUPTURE

4.1 NCMA

The "Specification for the Design and Construction of Load-Bearing Concrete Masonry" (14), published by the National Concrete Masonry Association, specifies, in section 3.3.2, values for the allowable tension in flexure for concrete masonry. For tension normal to the bedjoints and type M or S mortar, the allowable stress for hollow unit masonry is 23 psi, and for grouted or solid masonry, 39 psi. The derivation of these values is traced in Reference 15. Tables 4.1 and 4.2, reprinted from Reference 15, show the pertinent test data. Test specimens were uniformly loaded walls tested in accordance with ASTM E 72.

The design values for grouted masonry were <u>not</u> obtained from tests on grouted walls, but from tests on six composite walls composed of 4-inch concrete bricks and 4-inch hollow blocks (Table 4.1). The gross area of the walls was greater than 75% solid, so the walls were designated as solid masonry. Furthermore, it is assumed that solid and grouted masonry have identical behavior, a point that has been disputed by Hamid (18). The reported mean value of modulus of rupture was 157 psi (based on the gross area), and a factor of safety of 4 was applied to obtain the design value of 39 psi.

The design values for hollow unit masonry were obtained from the 27 tests, from three sources, shown in Table 4.2. The mean modulus of rupture for types M and S mortar (14 specimens) was 93 psi on the net area. Applying a factor of safety of 4 gives the allowable value of 23 psi.

TABLE 4.1 FLEXURAL STRENGTH, VERTICAL SPAN CONCRETE MASONRY WALLS FROM TESTS AT NCMA LABORATORY

ASTM Mortar Type*	Wall: Nominal Thickness in.	Wall: Max. Uniform Load psf.	Wall: Net Section Modulus in 3/ft	Modulus of Rupture: Gross Area, psi	Modulus of Rupture: Net Mortar Bedded Area, psi
Monowythe Walls of Hollow Units					
M	8	85.15	80.97	61.74	88.73
M	8	87.10	80.97	63.15	90.76
M	8	91.00	80.97	65.97	94.82
M	8	103.35	80.97	74.93	107.69
S	8	62.40	80.97	45.24	69.47
S	8	72.15	80.97	52.31	75.18
S	12	183.3	164.64	57.11	93.94
S	12	161.2	164.64	50.22	82.62
Composite Walls of Concrete Brick & Hollow CMU					
S	8	222.3	103.82	161.16	180.67
S	8	219.7	103.82	159.29	178.55
S	8	187.2	78.16	135.72	202.09
S	8	228.8	103.82	165.88	185.95
S	8	218.4	78.16	158.34	235.77
S	8	223.6	78.16	162.11	241.38
S	12	171.6	139.83	53.46	103.55
S	12	150.8	139.83	46.98	91.00
S	12	156.0	139.83	48.60	94.14
S	12	213.2	139.83	66.42	128.66
Cavity Walls					
S	10	98.8	50.36	158.62	165.55
S	10	156.0	50.36	250.44	261.38
S	10	88.4	48.16	141.91	154.88
S	10	119.6	50.36	192.01	200.40
S	10	114.4	50.36	183.66	191.68
S	10	109.2	48.16	175.30	191.32
S	12(4-4-4)	145.6	50.36	233.73	243.94
S	12(4-4-4)	145.6	50.36	233.73	243.94
S	12(6-2-4)	135.2	77.80	127.38	146.63
S	12(6-2-4)	119.6	77.80	112.68	129.70

* Mortar type by proportion requirements.
** Air space not included in gross area of cavity walls.

TABLE 4.2
FLEXURAL STRENGTH--SINGLE WYTHE WALLS OF HOLLOW UNITS--
UNIFORM LOAD--VERTICAL SPAN

Mortar Type Proportion ASTM C 270	Modulus of Rupture psi, Net Area	Reference
M	110	17
M	108	NCMA
M	102	17
M	97	17
M	95	NCMA
S	94	NCMA
M	91	NCMA
M	89	NCMA
N	88	16
S	84	17
S	83	NCMA
S	81	17
S	75	NCMA
S	69	NCMA
N	67	16
N	62	16
S	60	17
N	58	16
N	45	16
O	60	17
O	41	16
O	36	16
O	36	16
O	33	16
O	32	16
O	30	17
O	27	16

4.2 UNIFORM BUILDING CODE (1)

Section 2406(c) 4. specifies allowable tensile stresses for walls in flexure. For tension normal to bed joints and concrete units, the allowable stress is 40 psi for solid units and 25 psi for hollow units. The Commentary to Chapter 24 of the Uniform Building Code (2) contains an extensive discussion of factors affecting tensile bond strength in masonry, (including mortar proportions, mortar air content, unit initial rate of absorption, unit moisture content, joint geometry, mortar joint thickness, workmanship, and curing conditions), however, there is no discussion of the origin of the given allowable values. No specific data is cited, and no safety factors are given. The commentary includes, without comment, the allowable values given in Reference 14 for concrete units, (discussed in section 4.1 above), and the values provided in Reference 19 for solid clay units.

4.3 ACI 531-79

The ACI Code for reinforced concrete masonry (21) also specifies allowable values for masonry in tension. The allowable stresses are for both axial tension and tension in flexure, and they are related to mortar strength. For tension normal to bed joints, the allowable stresses are $0.5\sqrt{m_o}$ (maximum 25 psi) for hollow unit masonry, and $1.0\sqrt{m_o}$ (maximum 40 psi) for solid or grouted masonry. m_o is the specified 28-day minimum required compressive strength of mortar per ASTM C 270, psi.

ACI 531-79 states that stresses can be increased by 1.33 for earthquake design.

4.4 CMI STANDARD 301-76

The 1976 CMI Standard allows 0 psi for the tensile strength of masonry.

4.5 AUSTRALIAN STANDARD CA47-1969

The Australian Standard allows 10 psi for tensile stress normal to bed joints.

4.6 DRYSDALE, HAMID, AND TONEFF, 1983 (20)

Drysdale et. al. discuss allowable tensile stresses in masonry in reference (20). The authors find the allowable tensile stresses in North American codes (21,26) to be unjustified and unconservative based on test data on small scale specimens (18,24,25). They also point out, however, that "tests on full scale walls clearly indicate that, at least where walls are supported on more than two edges, capacities exceed by a considerable amount the predictions using strengths from small wall specimens and elastic analysis". The authors state the difficulty of identifying trends in the research data and the allowable stresses reported in the codes due to the large scatter in most data, (coeficients of variation within groups of apparently identical specimens are often on the order of 0.30). They do acknowledge, however, that "grouted blockwork has been shown to have very much improved tensile strength normal to the bed joints" (18,28,29). As a result, "for a safety factor of 2.0 between the characteristic strength .... and the allowable stress, the allowable stresses for grouted blockwork would be conservative ...".

Table 4.6.1 is included from Reference 20 for reference. Note

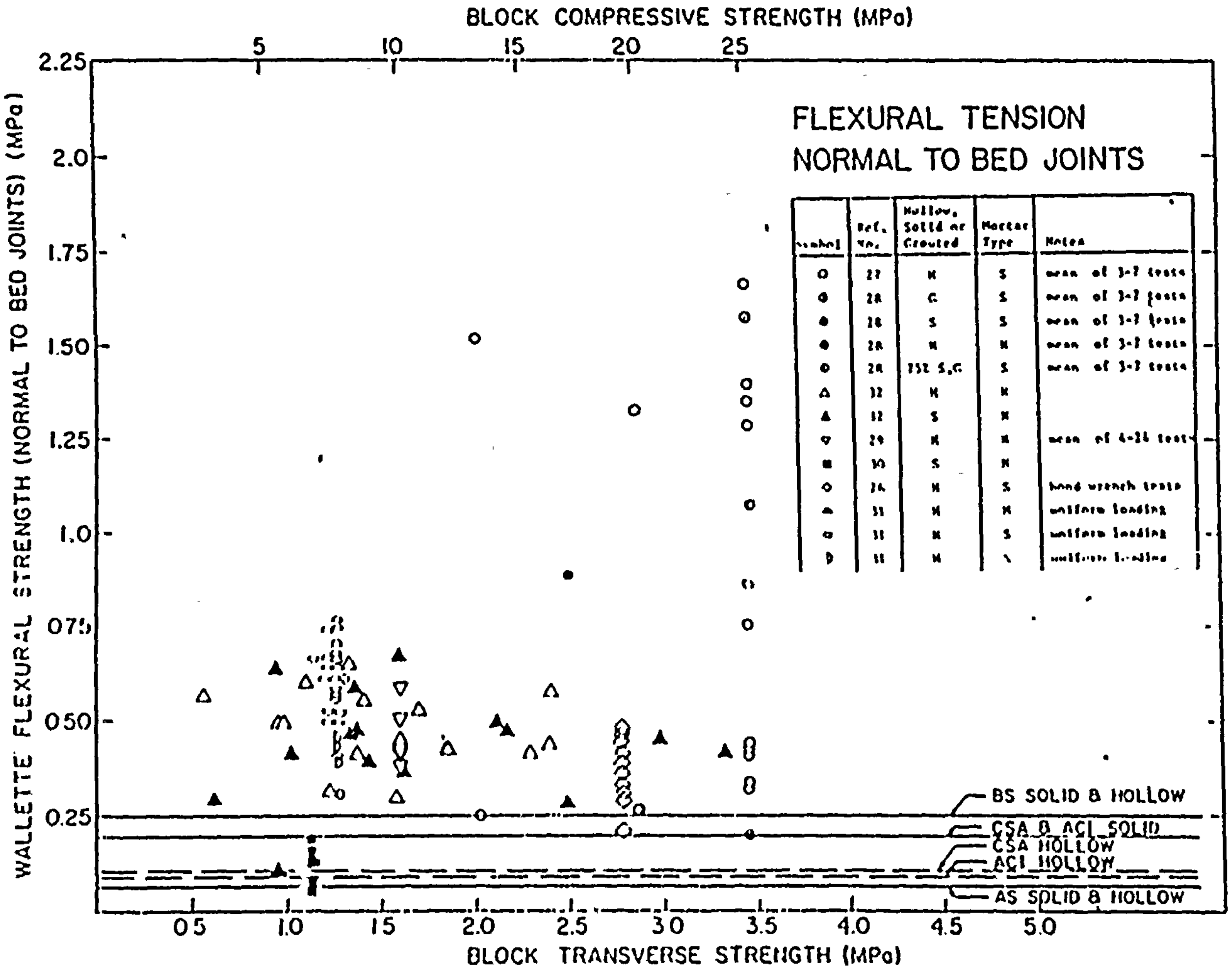


Figure 4.6.1

DATA AND ALLOWABLE STRESSES FOR FLEXURAL TENSION NORMAL TO THE BED JOINTS

that the data for grouted masonry falls above the data for ungrouted concrete masonry.

4.7 MILLER, NUNN, HEGEMIER, 1978 (Ref. 30)

Reference (30) reports the results of a study on the effect of various grouting strategies on the strength and elastic moduli of concrete masonry. Modulus of rupture tests were not performed; however, some prisms were tested in direct tension normal to the bed joint in addition to standard compression tests of prisms. For concrete, the split cylinder tensile strength usually ranges from 50% to 75% of the modulus of rupture (8), so it is likely that direct tension tests would represent a lower bound of modulus of rupture values for grouted masonry. Results of the compression and tension tests are reprinted in tables 4.7.1 and 4.7.2. ("Field Practice" prisms were built using standard constuction site procedures, and "controlled slump" prisms were built in carefully controlled laboratory conditions.) If, as is the practice with the modulus of rupture, the tensile strength is related to $\sqrt{f'm}$ by the equation

$$f_t = K \sqrt{f'm}$$

these results yield a mean K factor of 2.3 for the "field practice" prisms, and 2.5 for the "controlled slump" prisms.

4.8 DRYSDALE AND HAMID, 1984 (Ref. 18)

Drysdale and Hamid report on an experimental study of the effect of grouting on the flexural tensile strength of concrete block masonry in Ref. 18. Their results are summarized in table 4.8.1. The tests directly relating to grouted concrete masonry are shaded. Three different strengths of grout were used, however,

Table 4.7.1

Compressive Strength of 4-Course Field Practice Prisms

	STD	STD VIBR	ADM	ADM VIBR
	1348	2274	2181	2001
	1449	2426	2350	2282
	1398	2089	2324	2173
Failure Stress f_c	1887	2434	2308	2148
(psi)	1490	2358	2450	2468
mean	1524	2316	2323	2215
std. dev.	212	142	96	173

Tensile Strength of 3-Course Field Practice Prisms

	STD	STD VIBR	ADM	ADM VIBR
	69.7	116.3	92.5	94.8
	111.2	91.8	110.7	98.6
Failure Stress f_t	111.2	127.3	118.3	116.3
(psi)	84.3	113.0	110.4	143.2
	69.9	89.9	95.8	107.4
mean	89.3	107.7	105.5	112.1
std. dev.	20.9	16.3	10.9	19.3

Table 4.7.2

Compressive Strength of 4-Course Controlled Slump Prisms

	STD	STD VIBR	ADM	ADM VIBR
	2140	2123	1685	2359
	1702	2140	1735	2584
Failure Stress f_c	2079	2241	1574	2595
(psi)	2072		1735	2746
	1928		1634	2544
mean	1984	2171	1673	2566
std. dev.	176	60.4	69.2	139

Tensile Strength of 3-Course Controlled Slump Prisms

	STD	STD VIBR	ADM	ADM VIBR
	74.9	99.8	91.8	160.0
	82.1	93.8	81.1	150.0
Failure Stress f_t	105.0	114.8	92.4	161.0
(psi)	119.7	131.3	70.4	178.6
	88.2	133.4		
	95.4			
	107.1			
	93.5			
mean	95.7	114.6	83.9	162.4
std. dev.	14.5	17.9	10.4	11.9

TABLE 4.8.1
FLEXURAL TENSION TEST RESULTS

Group	Block Type	Grout Type	Mortar Strength (N/mm^2)	Flexural Tensile Strength[a]						
				Normal to Bed Joints				Parallel to Bed Joints		
				No. of Tests	f_{tn} (N/mm^2)	(psi)	V^d (%)	No. of Tests	f_{tp} (N/mm^2)	V^d (%)
1	NORMAL	–	16.5	5	0.43	62	37.8	3	1.00	18.4
2	NORMAL	GN	17.4	7	1.40	203	16.8	6	1.71	11.2
3	NORMAL	GN	16.1	6	1.36	197	12.3	–	–	–
4	NORMAL	GS	16.7	6	1.67	242	9.7	–	–	–
5[b]	NORMAL	GN	17.8	5	0.77[c]	112	6.0	–	–	–
6	SOLID	–	18.1	5	0.88	98	21.4	4	1.25	8.1

a) Stresses are based on minimum face shell area for ungrouted specimens and on gross area for grouted specimens

b) Only central cell grouted in two and a half block long wall (5 cells)

c) Based on minimum face shell area

d) Coefficient of variation

within the range of commonly used grout, the compressive strength of the grout was found to have a minimal effect on the modulus of rupture. They found that grouting concrete masonry causes a 233% increase in flexural tensile strength over hollow concrete masonry. Grouted concrete masonry gave an average tensile strength of at least 197 psi. No prism strength values were given making calculation of "K" impossible. The flexural capacities of their assemblages were greater than predicted when superimposing tensile bond strength and grout tensile strength. An analysis in which failure was controlled by the extreme fiber of grout only slightly underestimated strength, however, the analysis showed sensitivity to "subjective assumptions".

4.9 DRYSDALE AND HAMID, 1982 (Ref. 29)

In Ref. 29, Drysdale and Hamid report on the in-plane tensile strength of concrete masonry. Tensile strength was measured using splitting tension tests of masonry assembleges. (Note that for concrete splitting tensile strength is usually 50% to 75% of the modulus of rupture, (Ref. 8)). Results are summarized in Table 4.9.1 with shaded values relating specifically to grouted concrete masonry. Grouting was found to have a profound effect on the tensile strength normal to the bed joint.

Table 4.9.1

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. Summary of splitting test results

Series No.	Block Type(a)	Grout(a) Type	Splitting Tensile Strength: Parallel to Bed Joints: No. of Specimens	Parallel: $f_{tp}^{(b)}$ (MPa)	Parallel: $f_{tp}^{(c)}$ (MPa)	Parallel: V(%)	Normal to Bed Joints: No. of Specimens	Normal: $f_{tp}^{(b)}$ (MPa)	Normal: $f_{tn}^{(c)}$ (MPa)	Normal: V(%)
1	190 mm. NB Hollow	-	4	0.55	1.60 (0.89)(d)	6.5	3	0.20	0.59	23.3
2		CN	5	0.85	-	9.2	5	0.80	-	13.0
3		CN(e)	4	0.70	-	9.1	2	0.75	-	-
4		CN	3	0.90	-	20.5	3	0.76	-	10.3
5		CS	2	0.82	-	-	3	1.00	-	12.1
6	190 mm. NB 75% Solid	-	3	0.74	1.20	2.9	3	0.30	0.54	5.4
7		CN	2	1.00	-	-	3	0.98	-	8.1
8	190 mm. NB 100% Solid	-	5	0.99	0.99	9.7	3	0.70	0.70	14.4
9	190 mm. WB Hollow	-	3	0.52	1.52 (0.88)(d)	3.1	3	0.21	0.61	24.1
10		CN	8	0.90	-	15.9	3	0.85	-	8.2
11	190 mm. SB Hollow	-	2	0.73	2.14 (1.18)(d)	-	-	-	-	-
12		CN	3	1.16	-	9.7	-	-	-	-
13	190 mm. LB Hollow	-	3	0.55	1.60 (0.89)(d)	4.6	-	-	-	-
14		CN	4	0.88	-	13.2	2	1.07	-	-
15	190 mm. NB(f) Hollow	-	1	0.22	0.64	-	1	0.20	0.58	-
16		CN	3	0.63	-	15.8	3	0.77	-	6.7
17	140 mm. NB Hollow	CN	3	0.62	-	9.0	-	-	-	-
18	240 mm. NB Hollow	CN	2	0.70	-	-	2	0.64	-	-

(a) for grout and block properties, See Tables 1 and 2.

(b) based on gross area for all specimens.

(c) based on minimum face shell thickness.

(d) based on face shell thickness adjacent to the central web.

(e) empty bed joints.

(f) stack pattern.

5.0 OBSERVATIONS

The data and information presented herin were compiled in a very short time and cannot be considered an exhaustive treatment of the subject. A problem encountered was a general lack of standardization of the data reported. For example, the use of different symbols or terms to identify the same physical quantity. Also differences in experimental procedure may qualify some of the results. Further investigation would be required to resolve some of these issues.

No experimental values for modulus of rupture were found for 12-inch (nominal) grouted concrete block masonry. Based primarily on the results of the slender wall program (3):

1) it appears that K values in $f_r = K \sqrt{f'm}$ increases with thickness,

2) it appears that K values for grouted concrete block masonry, assuming zero tensile strength of the mortar bed joint, are of the same order as the K values for concrete panels,

3) it appears that the K values for concrete panels are less than the value of 7.5 given by ACI 318-83 (4), and

4) it appears that K value of 2.5 for concrete masonry is a lower bound. K = 2.5 was derived from 6" wall tests only, and the Slender Wall tests seem to indicate that the value of K increases with wall thickness.

The effect of the value of masonry elastic modulus on out-of-plane deflection calculaions is significant. For this reason, care must be taken in assuming elastic modulus values based on

compressive strength data. Specific information on the numerical value of the elastic modulus (E) for masonry is difficult to derive from the literature, since no convention has been consistently followed in the definition of E for nonlinear material behavior. This difficulty, combined with the inherent variability of masonry material properties, makes the prediction of E values difficult. The UBC recommends taking the value of E as 1000 f'm. Test data, using measured values of compressive strength and elastic modulus, indicates that this may be an upper bound on masonry stiffness rather than a mean value. Assuming a lower ratio of E/f'm may be appropriate. Average measured values of elastic modulus for grouted hollow concrete masonry prisms (Refs. 3,30,33) range from 1500 ksi to 2600 ksi.

Different techniques for calculating the moment of inertia after initial cracking can also affect deflection calculations. The ACI technique for calculating an effective moment of inertia results in a slightly stiffer section, and thus smaller deflections, than if the section is assummed to be completely cracked as per the UBC. Both techniques provide a reasonable prediction of out-of-plane deflections within the range of engineering interest, based on data from the Slender Wall Test Report (Ref. 3). (Both ACI and UBC deflection calculations shown in Figures 2.16 and 2.17 were based on measured modulus of rupture values: f_r = 138 psi for 6" masonry, f_r = 208 psi in 10" wall)

Further time and effort would be required to fully document existing data on the modulus of elasticity and the modulus of rupture for grouted hollow unit concrete masonry. Data exists in

various places which would require time to acquire. Because of different methods used by the various researchers to present and analyze data, time and effort would be required to convert all data to a common form for analysis.

It is apparent, however, based upon the experience of this study and the experience of Atkinson-Noland and Associates in many previous masonry studies, that it would be very difficult to base precise conclusions regarding the values of the modulus of rupture and the modulus of elasticity for a specific wall or walls upon data in the literature. Present standards and methods for determining material properties, fabrication of specimens, and recording data are not sufficiently uniform to support precise comparisons of results, however, they may be satisfactory at this time to establish design values. It is the authors' opinion that values required for analysis of specific masonry are best obtained by tests of samples of the specific masonry or of samples built to replicate the specific masonry.

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Design, Reinforced Masonry Slender Wall

Sec. 2411. (a) General. In lieu of the procedure prescribed in Section 2409, the procedures set forth in this section, which considers the slenderness of walls by representing effects of axial forces and deflection in calculation of moments, may be used when the vertical load stress at the location of maximum moment does not exceed $0.04 f'_m$ as computed by the following formula:

$$\frac{P_w + P_o}{A_g} \leq 0.04 f'_m \quad \text{.......................(11-1)}$$

WHERE:

P_o = load from tributary floor or roof area.

P_w = weight of the wall tributary to section under consideration.

f'_m = ultimate compressive masonry stress as determined by Section 2406 (b). The value of f'_m shall not exceed 6000 psi.

A_g = gross area of wall:

(b) Slender Wall Design Procedure. 1. Maximum and minimum reinforcement. The reinforcement shall not exceed the following ratios, p_g, of reinforcement area to gross masonry area:

	$f_y = 40$ ksi	$f_y = 60$ ksi
Concrete Block Masonry	0.0048	0.0032
Hollow Brick Masonry and Two-wythe Brick Masonry	0.0060	0.0040

Minimum reinforcement shall be provided in accordance with Section 2407 (h) 4 B for all seismic areas when using this method of analysis.

2. **Moment and deflection calculations.** All moment and deflection calculations in Section 2411 (b) are based on simple support conditions top and bottom. Other support and fixity conditions, moments and deflections shall be calculated using established principles of mechanics.

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3. **Strength design. A. Load factors.** Factored loads shall be based on:

$$U = 1.4D + 1.7L, \text{ or} \quad \text{(11-2)}$$

$$U = 0.75(1.4D + 1.7L + 1.87E), \text{ or} \quad \text{(11-3)}$$

$$U = 0.75(1.4D + 1.7L + 1.7W), \text{ or} \quad \text{(11-4)}$$

$$U = 0.9D + 1.43E, \text{ or} \quad \text{(11-5)}$$

$$U = 0.9D + 1.3W \quad \text{(11-6)}$$

WHERE:

D = dead loads, or related internal moments and forces.
E = load effects of earthquake, or related internal moments and forces.
L = live loads, or related internal moments and forces.
U = required strength to resist factored loads, or related internal moments and forces.
W = wind load, or related internal moments and forces.

B. Required moment. Required moment and axial force shall be determined at the midheight of the wall and shall be used for design. The moment strength, M_u, shall be at least equal to:

$$M_u = \frac{w_u h^2}{8} + P_{ou}\frac{e}{2} + (P_{wu} + P_{ou})\,\Delta_u \quad \text{(11-7)}$$

WHERE:

w_u = factored distributed lateral load.
h = height of the wall between points of support.
P_{wu} = factored weight of the wall tributary to the section under consideration.
Δ_u = horizontal deflection at midheight under factored load; $P\Delta$ effects shall be included in deflection calculation.
P_{ou} = factored load from tributary floor or roof loads.
e = eccentricity of P_{ou}.
P_u = axial load at midheight of wall, including tributary wall weight.

$$P_u = P_{wu} + P_{ou} \quad \text{(11-8)}$$

C. Design strength. Design strength provided by the reinforced masonry wall cross section in terms of axial force and moment shall be computed as the nominal strength (see Section 2602, Definitions) multiplied by a strength reduction factor, ϕ, as set forth in Formula (11-9).

$$M_u \leq \phi M_n \quad \text{(11-9)}$$

WHERE:

M_n = nominal moment strength found for cross sections subjected to combined flexure and given axial load.

The strength reduction factor for flexure, ϕ, shall be as follows:

	Special Inspection Yes	Special Inspection No
Concrete Block and Hollow Brick (Designated 6-inch thickness or greater) and two-wythe brick masonry	0.8	0.5
Hollow Brick Masonry (Designated 5-inch or less thickness)	0.6*	0.4*

*EXCEPTION: $\phi = 0.8$ for special inspection and $\phi = 0.5$ for noncontinuous inspection when vertical reinforcing bars are held in position at the top, bottom and at intervals not farther apart than 192 bar diameters.

D. **Design assumptions for nominal strength.** Nominal strength of singly reinforced masonry wall cross sections to combined flexure and axial load shall be based on applicable conditions of equilibrium and compatibility of strains. Strain in reinforcement and masonry walls shall be assumed directly proportional to the distance from the neutral axis.

Maximum usable strain at extreme masonry compression fiber shall be assumed equal to 0.003.

Stress in reinforcement below specified yield strength f_y for grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.

Tensile strength of masonry walls shall be neglected in flexural calculations of strength, except when computing requirements for deflection.

Relationship between masonry compressive stress and masonry strain may be assumed to be rectangular as defined by the following:

(i) Masonry stress of $0.85 f'_m$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = 0.85\,c$ from the fiber of maximum compressive strain.

(ii) Distance c from fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

4. **Deflection design.** The midheight deflection, Δ_s, under service lateral and vertical loads (without load factors) shall be limited by the relation

$$\Delta_s = 0.007h \qquad \text{(11-10)}$$

EXCEPTION: For hollow brick masonry designated 5 inches or less in thickness where vertical reinforcement bars are not held in position at the top, bottom and at intervals not farther apart than 192 bar diameters, use

$$\Delta_s = 0.005h \qquad \text{(11-11)}$$

The midheight deflection shall be computed with the following formula:

$$\Delta_s \begin{cases} \dfrac{5 M_s h^2}{48 E_m I_g} \text{ (for } M_s < M_{cr}) \text{ ......(11-12)} \\[2ex] \dfrac{5 M_{cr} h^2}{48 E_m I_g} + \dfrac{5 (M_s - M_{cr}) h^2}{48 E_m I_{cr}} \text{ (for } M_{cr} < M_s < M_n) \text{ ......(11-13)} \end{cases}$$

WHERE:

h = height of the wall.

M_s = service moment at the midheight of the panel, including $P\Delta$ effects.

E_m = $1000f'_m$.

I_g, I_{cr} = gross, cracked moment of inertia of the wall cross section.

M_{cr} = cracking moment strength of the masonry wall.

M_n = nominal moment strength of the masonry wall.

The cracking moment strength of the wall shall be determined from the formula:

$$M_{cr} = Sf_r \text{ ......(11-14)}$$

WHERE:

S = section modulus.

f_r = modulus of rupture shall be assumed as follows for calculating deflection:

	f_r psi
Concrete Masonry Units	$2.5\sqrt{f'_m}$ psi
Hollow Brick Units	$2.5\sqrt{f'_m}$ psi
Two-wythe Brick Walls	$2.0\sqrt{f'_m}$ psi