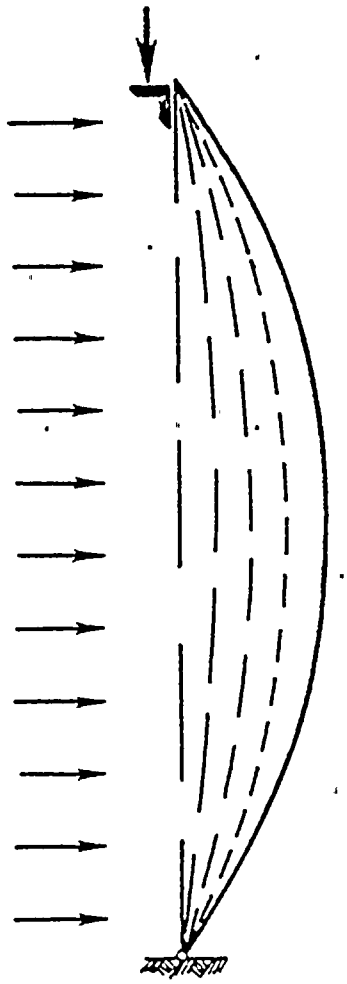
Enclosure 3

TEST REPORT on SLENDER WALLS



Southern California Chapter
American Concrete Institute

Structural Engineers Association
of Southern California

ACI-SEASC Task Committee on Slender Walls ©

American Concrete Institute, Southern California Chapter

and the

Structural Engineers Association of Southern California

REPORT OF THE
TASK COMMITTEE ON SLENDER WALLS

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February 1980 - September 1982
Los Angeles, California

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Design engineers or architects using the information in this report are cautioned to exercise judgment in its application to individual buildings. Specific conditions such as openings in walls, expansion or contraction of the concrete, roof, floor or foundation details, the dynamic effects of seismic loads, as well as job site controls must all be considered.

This report is proposed for guidance only, and the Structural Engineers Association of Southern California and the Southern California Chapter of the American Concrete Institute, including the members of the Task Committee on Slender Walls, take no responsibility for the application of any statements or principles included in this report.

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SECTION 1

GENERAL

The increased use of concrete and masonry walls for commercial and industrial buildings is due principally to their economy, fire safety, architectural appearance, and ease of construction. Along with this increased usage, there has been a trend toward making these walls more slender in the interest of further cost savings.

The proper design of these walls for strength and safety has been an important task for the structural engineer, and a number of concepts have been developed for these walls to resist seismic or wind forces. In conjunction with the design process, building codes have limited the ratio of height to thickness of these walls.

1.1 COMMITTEE FORMATION AND GOALS

To confirm these design concepts and to evaluate the slenderness limitations, a volunteer committee called the ACI-SEASC Task Committee on Slender Walls was formed. The committee consisted of representatives from the Structural Engineers Association of Southern California and the Southern California Chapter of the American Concrete Institute.

The goal of the Task Committee was to test slender concrete and masonry walls to determine their behavior when subjected to eccentric axial and lateral forces that simulated gravity loads, along with wind or seismic pressures.

1.2 TEST SPECIMENS AND RESULTS

A total of 30 full-size panels were tested. These panels were constructed of tilt-up concrete, concrete block, clay brick, and clay blocks. The panels were tested in a special frame

capable of simulating eccentric roof loads as well as lateral forces, both applied at the same time. Horizontal deflections of the wall panels were measured under varying increments of load to determine the ultimate capacity of each panel.

The test results were dramatic and very informative. The tests showed excellent behavior of all panels under severe loading conditions, and most importantly, showed that the arbitrary and fixed limitation of height to thickness ratio is inappropriate and control should be based on strength and deflection considerations. These tests proved that thin walls of this construction can handle all specified code loadings for vertical and lateral forces with reserve deflection capacities far in excess of service requirements.

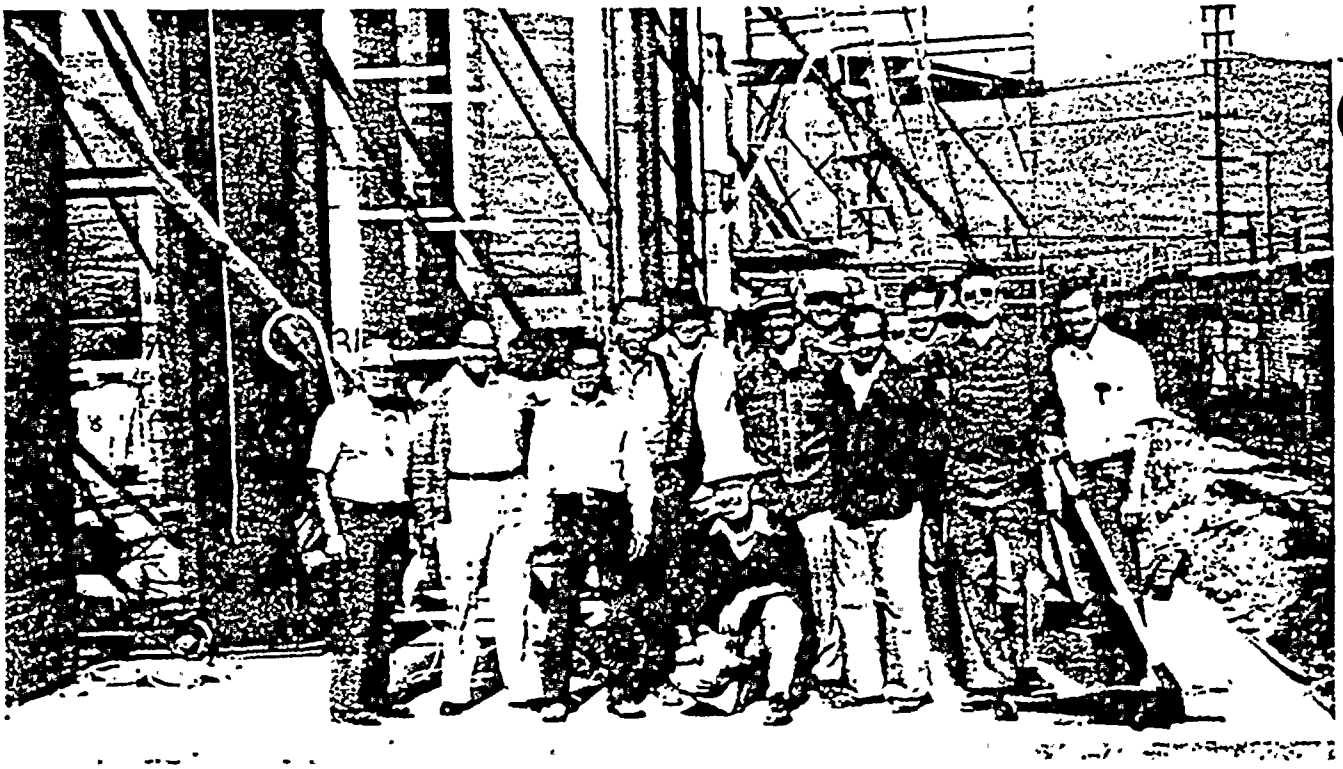


Fig. 1-1. Members of the Task Committee on Slender Walls. Ralph S. McLean (Proj. Director), James E. Amrhein, James S. Lai, William M. Simpson (Chairman), Robert E. Tobin, Joseph A. Dobrowolski (kneeling), Lawrence G. Selna, James R. Johnson, Samy A. Adham, John Coil, Donald E. Lee, and Ullrich A. Foth

1.3 REPORT OF THE TASK COMMITTEE

This report on the testing program follows, for the most part, the experimental process. Section 2 lists the materials and properties as they applied to the test program. The succeeding two chapters describe the construction of the test specimens and explain the equipment and methods used to perform the testing. Test results and an interpretation of these results are given in Sections 5 and 6. The development of design methods includes design requirements and procedures, analysis methods, and load/deflection relations; these are presented in Chapter 7. This is followed by design examples in Section 8. Conclusions and recommendations are listed in Section 9.

The final section of the report is an expression of gratitude to all who contributed their time and resources to make this project a success. Section 10 also includes a bibliography.

1.4 INFORMATION AVAILABLE

Original test data, laboratory reports on materials, analytical procedures, and presentational photos, slides and film of the project are available through the Office of the Structural Engineers Association of Southern California, 2550 Beverly Boulevard, Los Angeles, CA 90057. This information can be obtained with the approval of the Board of Directors and upon payment of costs for duplication and administration.

SECTION 2

MATERIALS AND MATERIAL PROPERTIES

2.1 INTRODUCTION

The materials in this test program were the normal materials used in average construction in Southern California. Concrete was supplied by a local ready mix firm using a typical mix proportion for tilt-up concrete. The concrete block and brick were from local manufacturers and were typical of the basic quality found in the area. No particular effort was made to use high strength materials, although it appears from the results that the materials were of higher strength than generally specified.

The materials and their properties are described below.

2.2 CONCRETE

Concrete was supplied by Consolidated Rock Products Co. from their Irwindale plant and was a five-sack per cu yd mix with a 0.67 water/cement ratio. The mix design by Conrock consisted of the following quantities per cubic yard:

Portland cement	– 470 lb (5 sacks)
Washed concrete sand	– 1420 lb (~14 cu ft)
1-in. gravel	– 1815 lb (~18 cu ft)
Water	– 317 lb (38 gal)

Compressive strength results measured by Conrock were:

7-day compressive strength test	– 2282 psi
28-day compressive test	– 3181 psi

During the placing of the concrete tilt-up wall specimens, 16 cylinders and 6 concrete beams were made. Test results by Twining Laboratories for laboratory specimens were as follows:

7-Day Test

Compression	–	2300 psi
Splitting tensile (average)	–	270 psi

28-Day Test

Compression (average)	–	3,225 psi
Modulus of elasticity	–	3,360,000 psi
Splitting tensile strength	–	355 psi
Modulus of rupture beam specimens	–	695 psi

167-Days (job-cured)

Compressive strength	–	4,009 psi
Modulus of elasticity	–	3,540,000 psi
Modulus of rupture beams	–	520 psi

The concrete tilt-up panels were provided with conventional tilt-up lifting inserts. Two were placed in each edge and two at the top end of each panel. All panels were cast October 3, 1980, and lifted October 15, 1980, and stored on edge with air space between panels. The panels were lifted by the insert in the long edges to assure that the panels would not be damaged in lifting.

2.3 CONCRETE BLOCK

The concrete masonry units (CMU) were supplied by Angelus Block Co., Inc. of Sun Valley, California, and were Grade N, Type II, medium weight and normal strength. Tested prisms of

concrete masonry units made at the time of construction demonstrated the following properties:

CMU	Compressive Strength, f'_m	Modulus of Elasticity, E_m
10"	2460 psi	2.17×10^6 psi
8"	2595 psi	1.72×10^6 psi
6"	3185 psi	1.59×10^6 psi

2.4 CLAY BRICK

The clay brick units Grade SW were supplied by Higgins Brick Co. of Redondo Beach, California. Properties of brick prisms, shown in Figure 2-1, were as follows:

Clay Brick	Compressive Strength, f'_m	Modulus of Elasticity, E_m
9.6"	3060 psi	1.12×10^6 psi
7.5"	3440 psi	1.42×10^6 psi

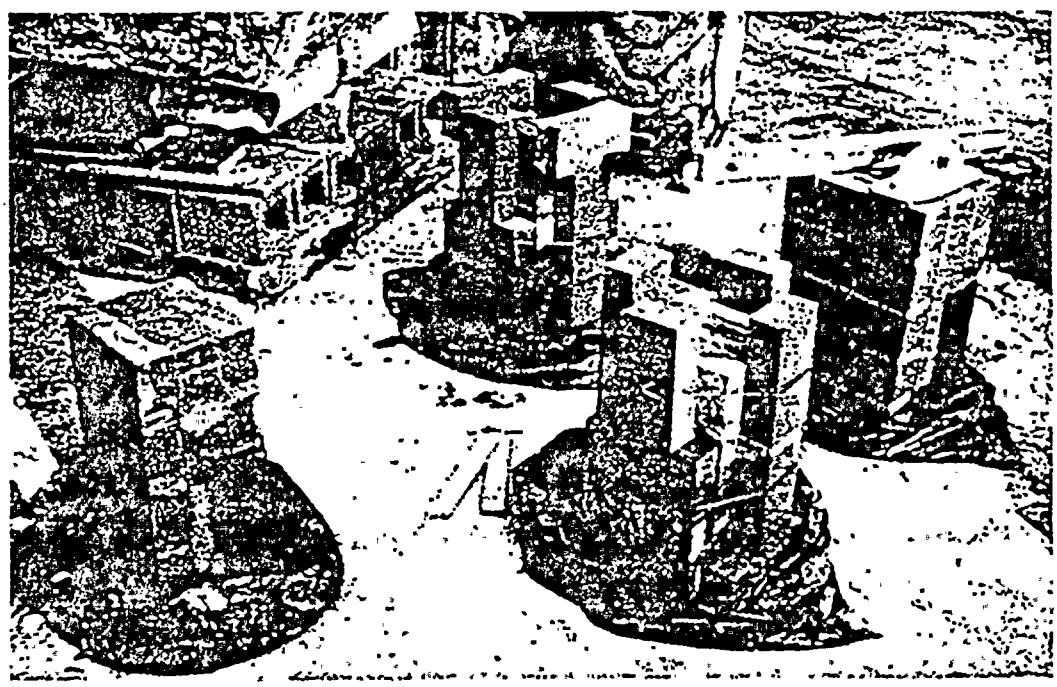


Fig. 2-1 Construction of 2-Wythe 9.6" Thick Grouted Brick Prisms.

2.5 CLAY BLOCK

The 5-1/2 in. thick hollow brick units, clay block, were supplied by Mutual Materials Co. of Bellevue, Washington, and had the following properties:

Clay Block	Compressive Strength, f'_m	Modulus of Elasticity, E_m
5.5"	6243 psi	2.33×10^6 psi

2.6 MORTAR

The mortar was Type S of the following proportions: one part portland cement, one-half part lime, and four parts sand by volume; all measurements were made with a cubic foot box initially. After the job progressed, shovel measurements were used.

Mortar, Type S 2" x 4" Cylindrical Specimens	Compressive Strength, f'_m	Standard Deviation
7-day	2348 psi	498 psi
28-day	3361 psi	614 psi

The 7-day strengths ranged from a low of 1305 psi to a high of 3365 psi. The 28-day strengths ranged from a low of 2420 psi to a high of 4710 psi.

2.7 GROUT

The grout was mixed on the job in the proportion of one part portland cement, three parts sand, and two parts pea gravel, with enough water for a slump of 8 to 10 in. All materials were measured by volume and were mixed in a drum mixer. Seven-day average strengths of 43 grout specimens 4" x 4" x 8" were 2014 psi with a standard deviation of 400 psi; these ranged from

a low of 1190 psi to a high of 2630 psi. The 28-day strengths for 58 specimens 4" x 4" x 8" averaged 3106 psi with a standard deviation of 474 psi; these ranged from a low of 1965 psi to a high of 4000 psi.

2.8 REINFORCING STEEL

All vertical steel was Grade 60 furnished by Bethlehem Steel and came from the same heat. The mill reported yield strength of 72,250 psi and ultimate tensile strength of 102,750 psi. Twining Laboratories reported the yield strength to be 67,500 psi and the ultimate tensile strength to be 102,000 psi. The average yield strength was 70,000 psi. Elongation in 8 in. equaled 17%. The modulus of elasticity was measured to be 28.6×10^6 psi. All vertical bars were in full length without splices. Steel properties on typical stress/strain tests by Twining Laboratories and the Structural Laboratory of UCLA are shown in Figure 2-2. Note

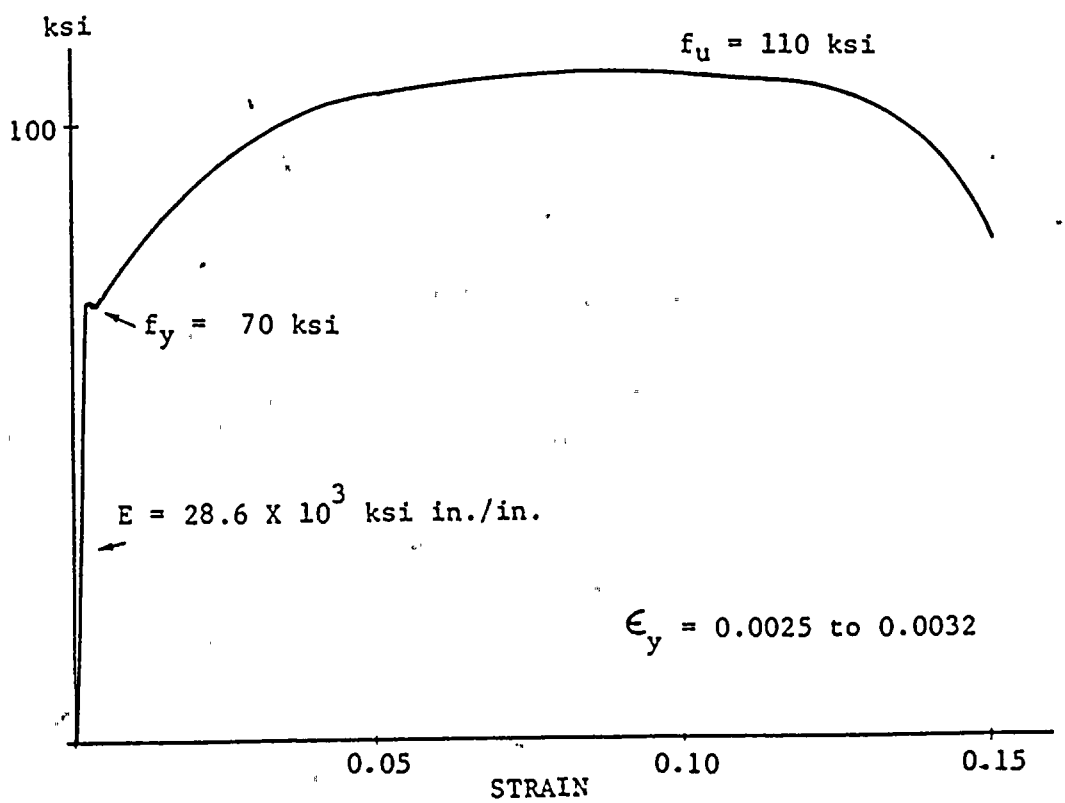


Fig. 2-2 Stress-Strain Curve for Reinforcing Steel.

that the reinforcement yield plateau strain of 0.0025 to 0.0032 in. per in. is a small part of the overall steel elongation.

Horizontal bars were #3, Grade 40, with the mill report stating yield strength of 52,730 psi and ultimate tensile strength of 75,910 psi. Twining Laboratories reported the #3 bars had yield strength of 52,000 psi and ultimate tensile strength of 79,100 psi. The elongation in 8 in. was measured at 18% and had a modulus of elasticity of 28.0×10^6 psi.

All reinforcing steel met the requirements of ASTM A615-78 Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement.

2.9 FIELD CUT PRISMS AND CORES

Upon completion of the deflection test program, prisms were sawed from all of the concrete masonry, brick, and clay block panels. In addition, cores were drilled from all of the concrete tilt-up panels. These samples were made over a year after the panels were first built. Figure 2-3 shows these cut samples ready to be transported to the testing laboratory.

A graphical comparison of the f'_m compressive strengths of the original prisms tested at 28 days with those cut from the walls over one year later is shown in Figure 2-4. As expected, the compressive strengths were greater for the year-old prisms. However, the moduli of elasticity of the older specimens were both above and below the 28-day values. This may have been due to differences in procedure since the tests were conducted originally at one laboratory and later at another laboratory. In either case these differences are not significant since the values were not used in the original design calculations.

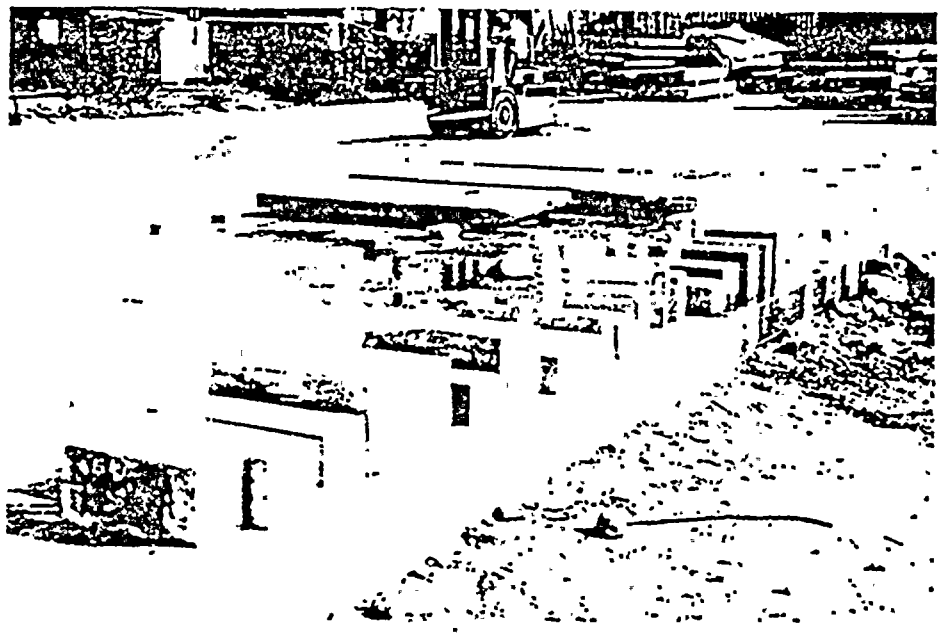


Fig. 2-3 Prisms and Cores from Tested Wall Panels.

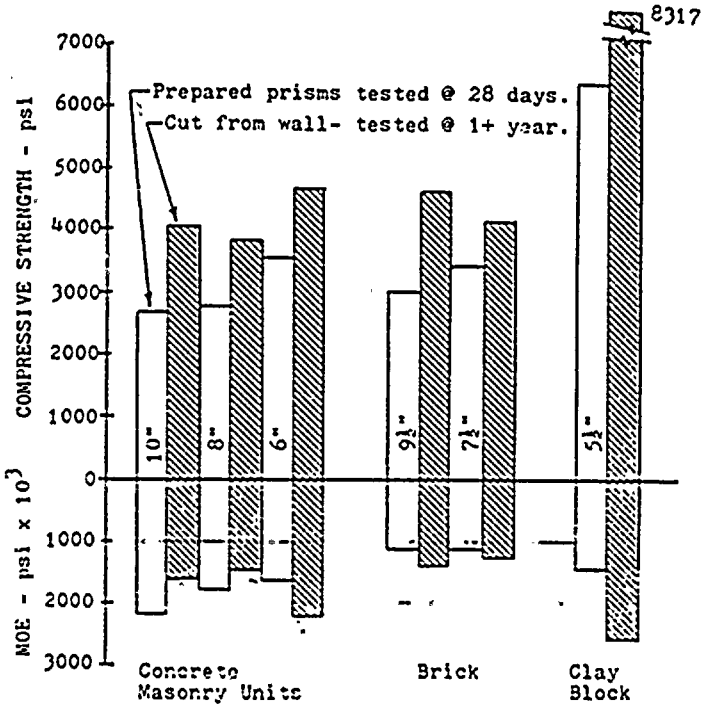


Fig. 2-4 Tests of Masonry Prisms.

Compression tests of 3.70-in.-dia. concrete cores varied between 4217 and 4862 psi with an average of 4607 psi. These compared with 28-day average strengths of 3200 psi. Modulus of elasticity tests were not made on these cores. On the basis of these compression values, the concrete had gained 44% in strength over a one-year period. The f'_m strength of the concrete block increased 43% in one year, the clay brick 35%, and the clay block 33%. The exact percentages are not as important as the fact that all of the walls, without any exceptions, improved with age.

Continuous strain measurements were taken on saw-cut prisms from all masonry panels. The maximum strain levels reached prior to compressional failure are as follows:

	Maximum Strain
Concrete Masonry Units	0.0032 in./in.
Brick Masonry	0.0042 in./in.
Hollow Brick Masonry	0.0038 in./in.

SECTION 3

CONSTRUCTION OF TEST SPECIMENS

3.1 GENERAL

In order to obtain realistic results, full-scale specimens were tested so that proper evaluations of the slenderness effect, the PΔ effect, and the eccentric moment (Pe) effect could be made.

All panels--concrete, clay brick, clay block, and concrete block--were built in the construction yard of Sanchez and Hernandez in Irwindale, California.

3.2 PANEL DESCRIPTIONS

All panels were 4'0" wide and 24'8" high, and were 24'0" between horizontal points of support (Fig. 3-1). The panel height of 24'8" was selected because it represented current construction trends and allowed for an evaluation of the slenderness ratios from 30 to 60 with standard materials of construction. Many industrial and commercial buildings and storage and manufacturing facilities are designed using approximately this height. A total of 30 walls were built for the test program.

Twelve walls were concrete tilt-up panels, three each with thicknesses of 4-3/4", 5-3/4", 7-1/4", and 9-1/2" resulting in nominal h/t ratios of 60, 50, 40, and 30 respectively. Nine walls, three of each thickness, were concrete masonry of 6", 8", and 10" nominal thickness with an actual thickness of 5-5/8", 7-5/8", and 9-5/8", for slenderness h/t ratios of 51, 38, and 30. Six walls were clay brick masonry, three each of 7.5" thick and 9.6" thick with h/t ratios of 38 and 30. Three walls were 5-1/2" hollow clay block units with an h/t ratio of 52.

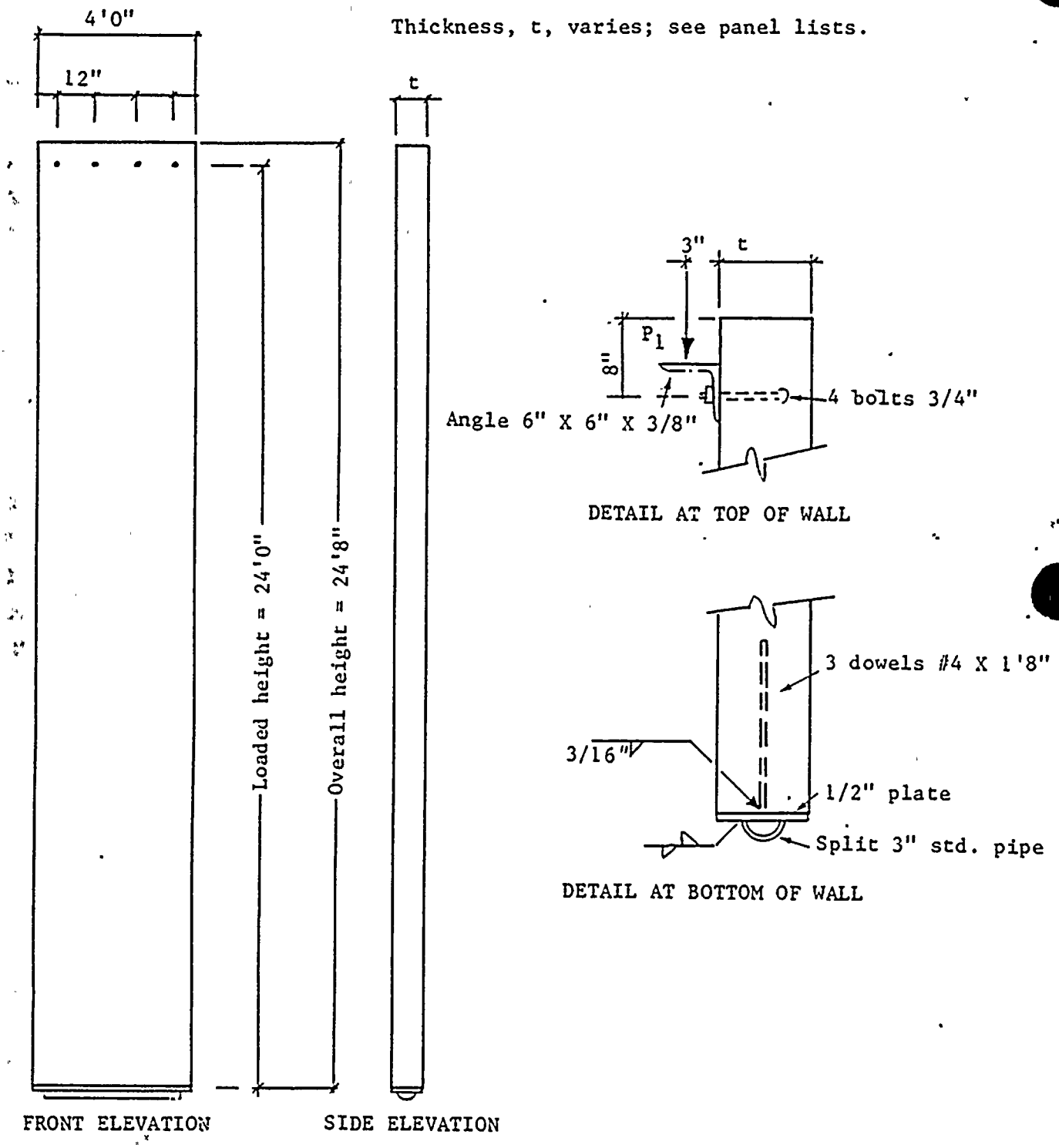


Fig. 3-1 Typical Panel Details: Concrete and Masonry.

The panels were built on half-inch base plates and supported on one-half of a 4" pipe underneath (Fig. 3-2). This provided a true pin connection for free rotation at the base. This pinned end simplified calculation, as there was zero moment at the bottom. The top support allowed the panels to rotate, and the support could move vertically to allow freedom of movement and rotation but yet prevent lateral translation.

3.3 CONCRETE PANEL CONSTRUCTION

The concrete tilt-up panels, as shown in Figure 3-3, were cast with the exterior face down and with ledger bolts protruding from the exposed face. Each panel contained four continuous 1/2" (#4) bars in the vertical direction. Number 3 horizontal bars spaced 2' o.c. were used in the 4-3/4" thick and 5-3/4" thick panels. Number 4 horizontal bars spaced 2' o.c. were used in the 7-1/4" thick and 9-1/2" thick panels. After casting, panels were lifted and stored on edge, as shown in Figure 3-4, for a period of 160 days before they were again lifted to the final vertical position on the test slab.

3.4 MASONRY PANEL CONSTRUCTION

All masonry panels were built in-place similar to normal construction methods, as shown in Figure 3-5. The masonry walls were built on 1/2" base plates supported on one-half of a 4" pipe. On the 1/2" plate, five 1/2" reinforcing bar dowels were welded and extended 24" into the panel. The panel reinforcing steel, five #4 bars, Grade 60, lapped the steel dowels on the base plate. The base plate was stabilized with wedges and dry-packed with mortar to secure it from moving during construction. The masonry panels were solid grouted in 4' lifts using the techniques of delayed consolidation: After the water was absorbed into the masonry, the grout was consolidated, which caused it to compact completely and bond firmly to the masonry.

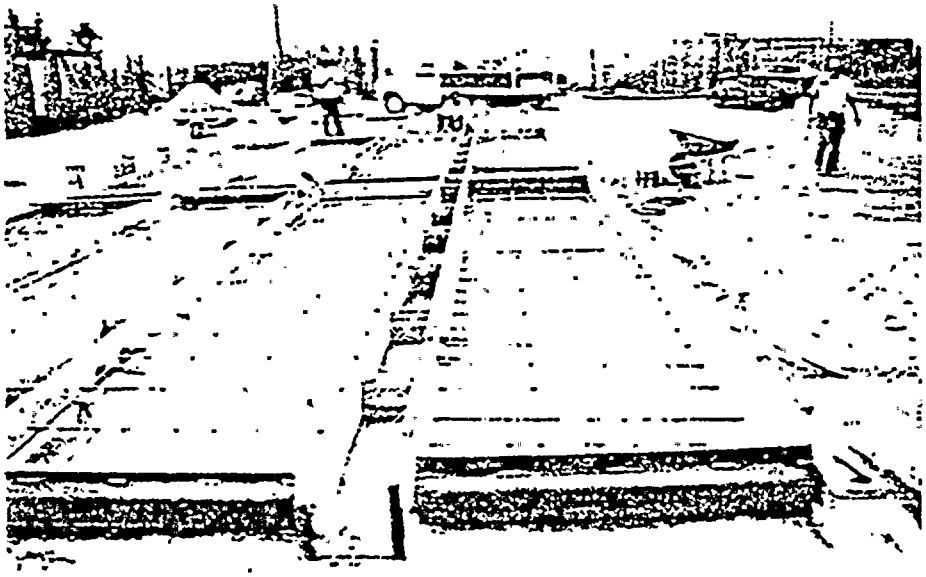


Fig. 3-2 Bottom of Tilt-Up Panel on Half-Round 4" Pipe.

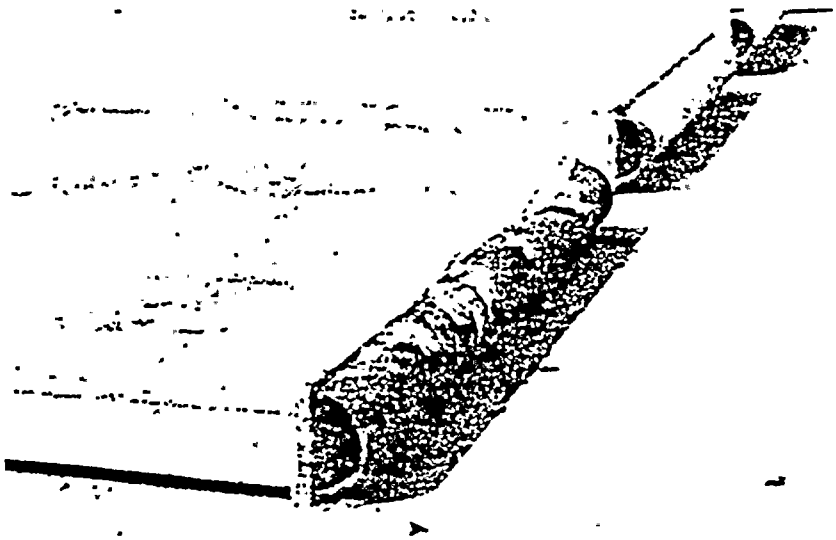


Fig. 3-3 Forming of Tilt-Up Panels.

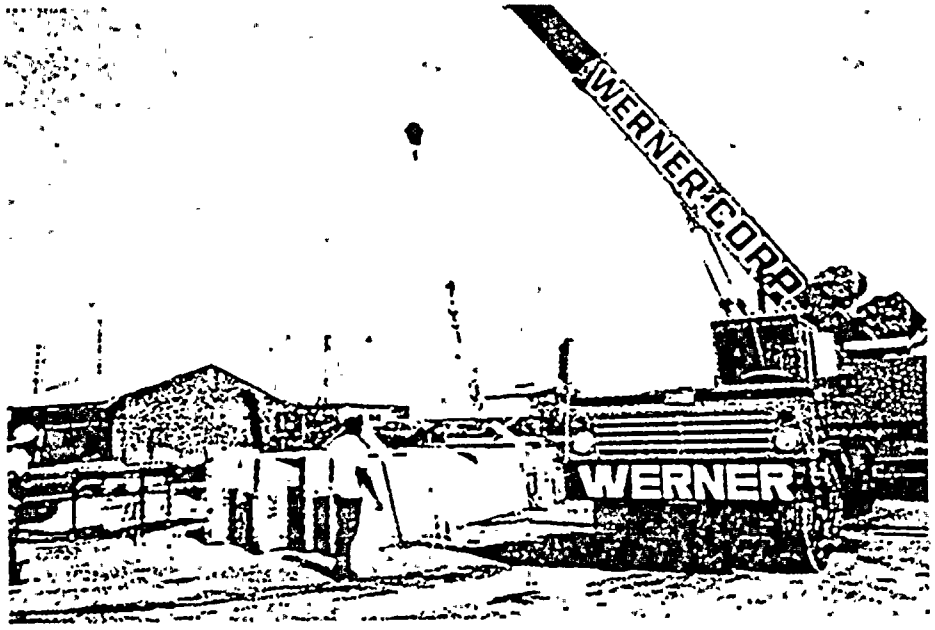


Fig. 3-4 Moving Tilt-Up Concrete Panels to Storage Area.

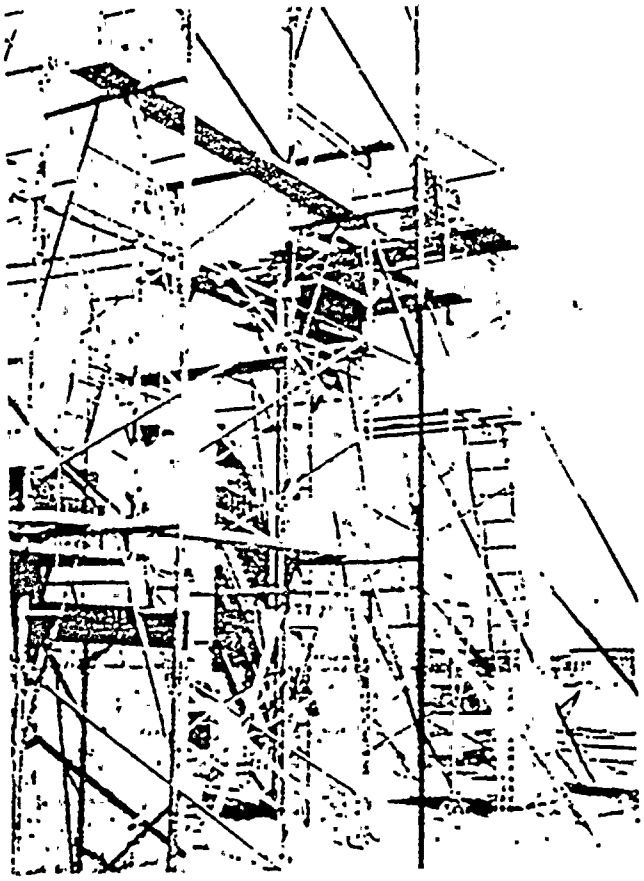


Fig. 3-5 Construction of Masonry Panels In-Place.

All brick masonry panels were constructed with continuous vertical reinforcement consisting of five #4 bars, and horizontal reinforcement with #3 bars spaced 4'0" apart vertically.

The panels were strengthened by reinforcing steel rods, which were embedded in the connecting grout for the clay brick and in the core grout for the concrete and clay blocks. Figure 3-6 shows the details for vertical steel placement in alternate rows of masonry. Figure 3-7 shows the completed masonry walls prior to testing. All walls were braced temporarily to prevent tipping.

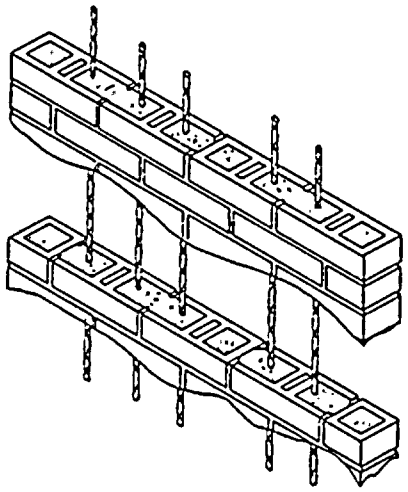
3.5 PLACEMENT OF STEEL

The specimens were designed to have steel reinforcement located in the center of the panel between the outer faces. After testing, the panels were broken apart, approximately in the middle third, and the location of the reinforcing steel was measured in relation to the loading face. Table 3-1 presents the measurements and accuracy of the steel placement.

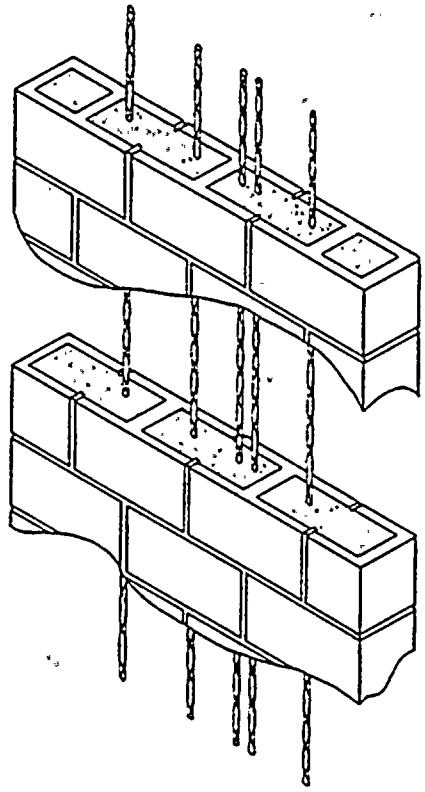
The last column of Table 3-1 indicates the percentage of the panel width that the reinforcing steel is off center line. For example, for the 9-1/2" thick concrete tilt-up panels, reinforcement placement remained good, within 3% of the center line location. But for the 7-1/4" thick and the 4-3/4" thick tilt-up concrete panels, larger d distances were measured to as much as 23% greater than the specified d.

For the concrete masonry wall panels, the steel placement ranged from 4% off center line to as much as 20% off. This meant the steel was kept within 1/2" of the specified location.

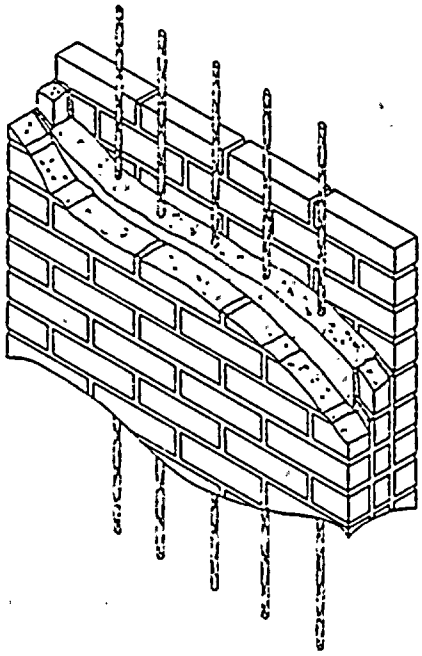
The brick wall steel placement was 19% and 22% off center line in two panels, but only 6% off in the third, which meant



Hollow brick



Concrete block



2-Wythe brick

Fig. 3-6 Arrangement of Reinforcing Steel in Concrete Block, Clay Block, and Two-Wythe Brick Panels.

that the greatest discrepancy from the specified location was more than 3/4".

Placement of steel away from design location may increase or decrease the capacity to resist forces depending upon where the steel is located and from which direction forces are considered. Analysis of the results reflect this variation in the location of the steel, and design parameters have been adjusted accordingly, amounting to what is known among engineers as the ϕ factor.

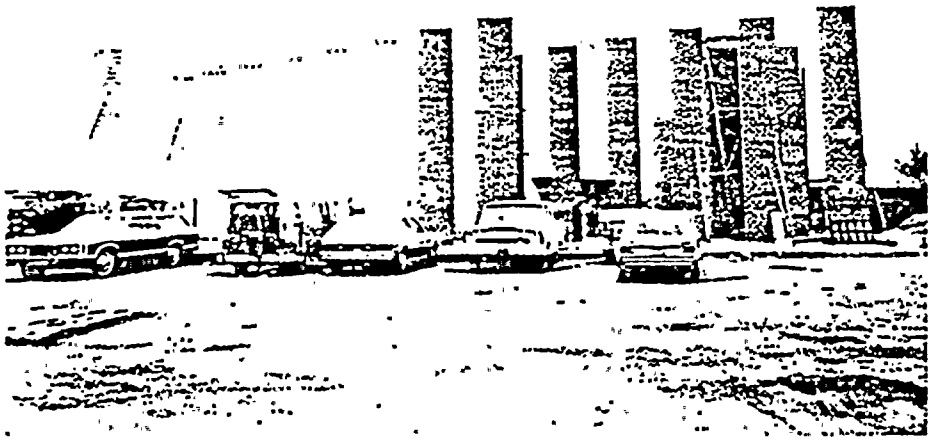


Fig. 3-7 Concrete Masonry and Brick Masonry Panels In-Place Ready to be Tested.

TABLE 3-1. PLACEMENT OF STEEL REINFORCEMENT

Panel	No.	Thickness (t), in. Nominal	Thickness (t), in. Measured	Distance (d) from Outer Face of Wall to Center Line ℄ of Steel, in. Bar #1	Bar #2	Bar #3	Bar #4	Bar #5	Ave. d	Variation in d* %	Variation in d* in.
Concrete Block	1	10	9.69	5.19	5.19	4.94	5.06	4.94	5.06	4.5%	0.21 in.
	2	10	9.69	4.19	4.19	4.69	4.19	4.69	4.39	9%	0.45 in.
	3	10	9.69	5.25	5.13	5.25	5.25	5.38	5.25	8%	0.40 in.
	4	8	7.63	4.13	4.13	4.13	4.24	3.88	4.10	7%	0.28 in.
	5	8	7.63	4.7	4.4	4.3	4.4	4.4	4.44	16%	0.62 in.
	6	8	7.63	3.3	3.3	3.2	3.2	3.1	3.22	16%	0.59 in.
	7	6	5.63	2.83	2.95	3.19	3.19	3.19	3.07	9%	0.25 in.
	8	6	5.63	2.18	2.24	2.22	2.16	2.41	2.24	20%	0.57 in.
	9	6	5.63	3.00	3.15	2.99	2.99	3.07	3.04	8%	0.22 in.
Clay Brick	10	9.6	9.63	5.38	5.63	5.88	5.88	6.01	5.76	19.5%	0.94 in.
	11	9.6	9.56	5.81	5.62	5.69	5.69	5.56	5.67	19%	0.89 in.
	12	9.6	9.56								
	13	7.5	7.38	4.4	4.4	4.7	4.5	4.6	4.52	22.5%	0.83 in.
	14	7.5	7.63	3.1	3.1	3.2	3.2	3.0	3.12	18%	0.69 in.
	15	7.5	7.63	3.7	3.8	4.2	4.3	4.3	4.06	6%	0.24 in.
Clay Block	16	5.5	5.50	3.40	3.34	3.27	3.08	3.19	3.26	17%	0.51 in.
	17	5.5	5.50	3.12	3.11	3.12	3.11	3.02	3.10	11%	0.35 in.
	18	5.5	5.50	3.31	3.34	3.49	3.32	3.29	3.35	20.5%	0.60 in.
Concrete Tilt-Up	19	9.5	9.6	4.67	5.24	4.48	4.24	**	4.66	3%	0.14 in.
	20	9.5	9.4	4.76	4.59	4.70	4.76		4.70	0%	0 in.
	21	9.5	9.5	4.4	4.6	4.7	4.8		4.63	3%	0.12 in.
	22	7.25	7.4	3.88	4.0	4.13	4.38		4.10	10%	0.40 in.
	23	7.25	7.34	2.85	3.35	3.48	3.48		3.29	10%	0.38 in.
	24	7.25	7.38	4.8	4.7	4.3	4.3		4.53	22%	0.84 in.
	25	5.75	6.13	3.7	3.8	3.7	3.6		3.70	20%	0.63 in.
	26	5.75	5.88	3.4	3.7	3.8	3.6		3.63	23%	0.69 in.
	27	5.75	6.0	3.38	3.5	3.25	3.25		3.48	15%	0.48 in.
	28	4.75	4.82	2.21	2.45	2.86	2.74		2.56	7%	0.15 in.
	29	4.75	4.78	2.46	2.58	2.90	3.16		2.77	16%	0.38 in.
	30	4.75	4.89	2.24	2.37	2.66	2.92		2.55	4%	0.10 in.

*% variation in d = $1 - \frac{\text{d average}}{\text{measured t} \div 2} \times 100$; in. variation in d = $\frac{\text{measured t}}{2}$ - d average

**bar #5 not used for concrete tilt-up panels

SECTION 4

TEST EQUIPMENT AND TEST METHODS

4.1 GENERAL

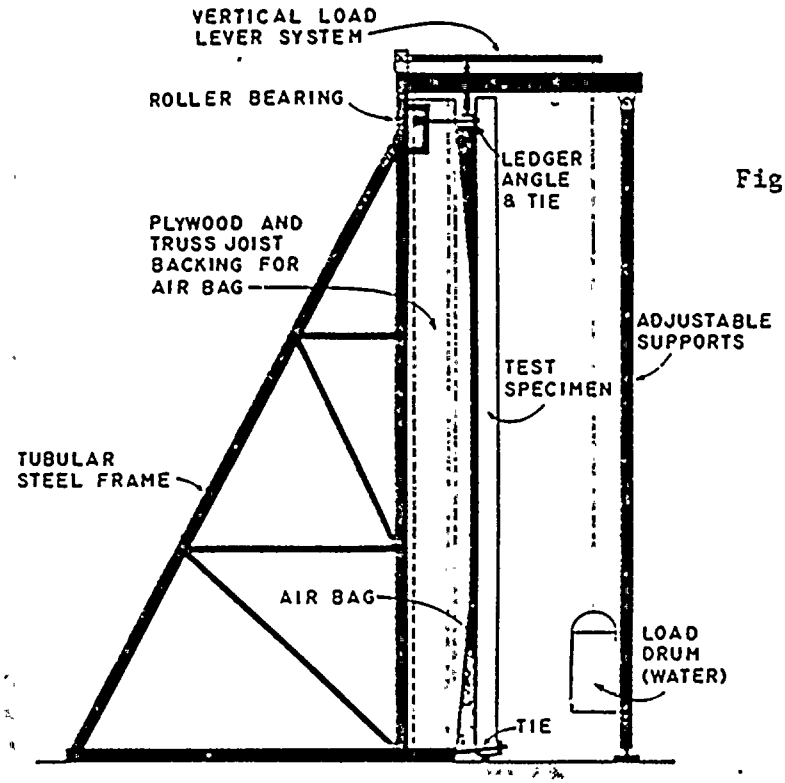
These tests were made to investigate the effect of lateral deflection on the stability of walls subjected to combined vertical and lateral loads. Since the weight of a wall is an important part of the vertical load, it was necessary to test the walls in an upright position.

4.2 LOADING FRAME

A welded steel trussed frame (Fig. 4-1) was constructed for this purpose. A secondary wooden structure of plywood was secured to the face of the frame as a backing for the air bag that supplied the lateral load. The walls were held against the air bag using threaded rods at the four corners of the walls and attached to the steel frame. At the top of the frame, a lever system placed vertical loads on the walls. Loose safety cables attached to the top of the walls and to outriggers with pipe supports were used to prevent total collapse of the walls if rupture should occur. Not shown in Figure 4-1 were a ladder and work platform for making wall attachments at the top, and wheels for moving the test frame from wall to wall.

4.3 PIN CONNECTIONS

The rocker base of the wall (see Sec. 3.2) eliminated moment at the bottom of the walls. The base assembly sat directly on the concrete floor slab. At the base, lateral pressure of the air bag was resisted by a length of 6" x 3" x 1/4" rectangular steel tube across the outer face of the wall, connected to threaded steel rods from each end of the tube to the steel frame. See detail in Figure 4-4.



SIDE ELEVATION OF TEST SETUP

Fig. 4-1 Loading Frame Showing Drums of Water for Vertical Load and Air Bag for Lateral Load.

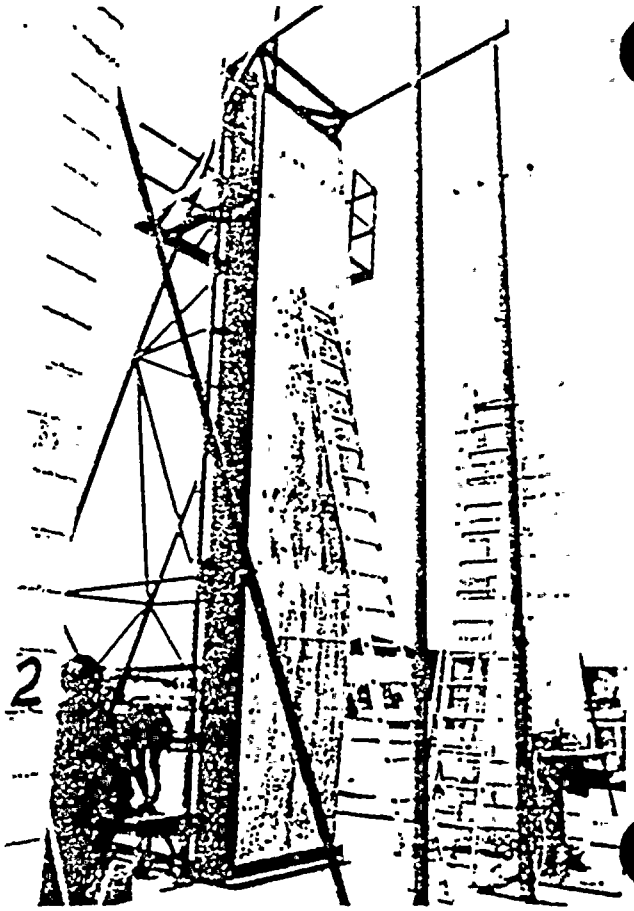


Fig. 4-2 Loading Frame Showing Scab Plywood Forming to Conform to the Loaded Panel.

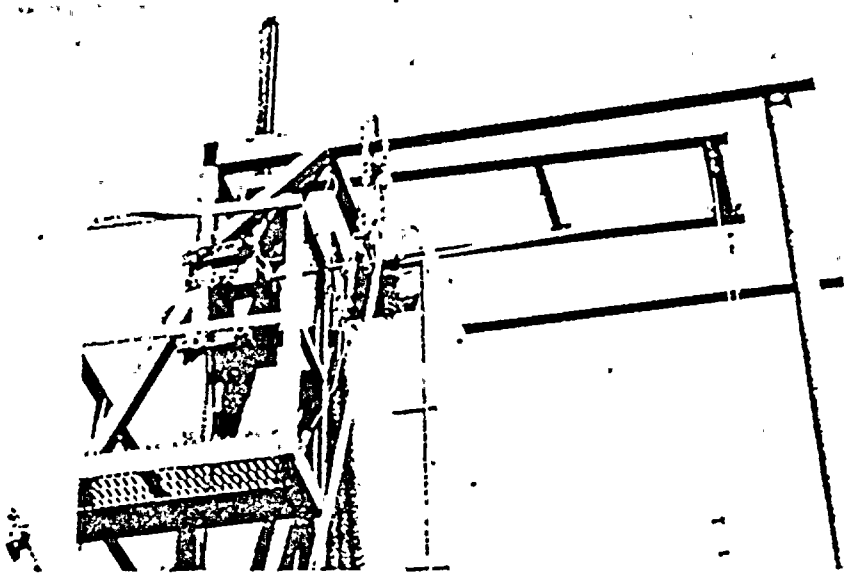


Fig. 4-3 Top Restraint Device that Allows Vertical Movement of Panel and Rotation at the Top. Safety Chain Holds Panel to Prevent Sudden Collapse.

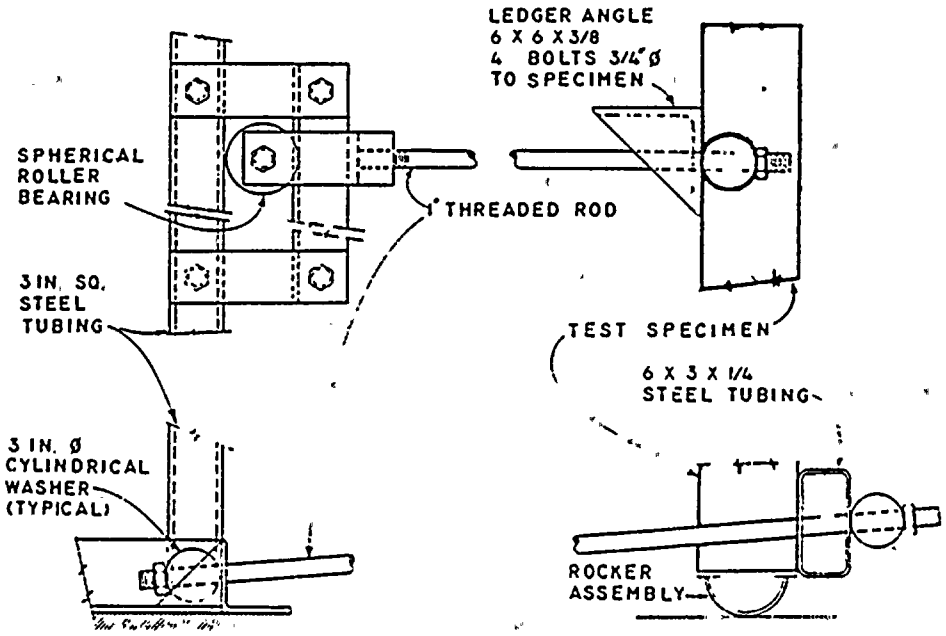


Fig. 4-4 Ties to Test Frame.

At the top of the walls, bolts were built in to secure a 6" x 4" x 3/8" steel ledger angle used to apply eccentric vertical loads to the walls. One-inch threaded rods tied the angle to the frame.

Because an extra wall built especially to test equipment and procedures achieved unusually large deflections, important changes in attachment of the wall to frame were required. The threaded steel rods for the top connections were attached to an assembly having a spherical roller bearing that rode on the steel bar and permitted ample vertical and angular movement. This top assembly is shown in Figures 4-3 and 4-4.

4.4 SAFETY CABLES

The safety cables consisted of a couple of loops of light steel cable passing through a connection at each end of the ledger angle and over the outriggers. Only two of the standard height wall specimens and one shorter one ruptured, but in each case the cables and top anchorage (of the ledger angle) prevented the walls from falling.

4.5 LOADING METHODS

Vertical Loads. The lever system for vertical load (Fig. 4-1) was pivoted at the top of the main frame and projected out past the wall specimen. Two lever arms were used and struts from the levers rested on the ledger angle. Steel drums loaded with water were hung from the lever arms with steel cable after initial displacement readings without vertical or lateral load were made. The quantity of water was adjusted for the desired load.

Lateral Loads. A uniformly distributed lateral load was applied to the walls with an air bag. The air bag consisted of a 20 mil vinyl bladder with welded seams and an outer wear-resistant cover of vinyl-coated nylon 22 oz material with sewed seams. Dimensions of the bag were 18" x 48" and 24' in length. Two ports were provided: one for inflation in one edge of the bag and one in the opposite edge for pressure readings. Grommeted flaps were provided at the top front and rear faces of the bag for hanging it from the outriggers.

Because the magnitudes of deflections of the loaded walls were much greater than anticipated and the unsupported edges of the bag ballooned out, some loss of contact with the wall occurred at large deflections. However, an investigation using the deflection curve obtained from test measurements showed that this loss of contact was negligible below the loading at which yielding of the reinforcement occurred. This is discussed in Section 6.3.

4.6 PANEL EMPLACEMENT

After casting and curing, each concrete wall was lifted by means of its edge inserts in order to avoid high stresses that would occur with end lifting. The walls were stored on edge awaiting testing. Later, when they were transported to the test site, they were again lifted by their edge lifts and rotated in the air by means of another line and inserts in the end of the panels; this careful handling ensured that premature cracking was avoided. Masonry walls were constructed in-place as described in Section 3.4.

All walls were held in place with telescoping pipe bracing anchored to the floor. Once the test frame was in position and the wall secured to it, the bracing was removed. Upon completion of testing, the brace was reinstalled, the test frame removed,

and the wall in its deformed condition allowed to remain in place until a number of walls could be transported at one time.

4.7 AIR PRESSURE MEASUREMENT

In testing, the air bag was inflated by a small (1/2 Hp) compressor, which proved to be ideal. A pressure regulator was provided, but it was found that a small needle valve gave very good control of inflation. Pressure in the bag was measured initially by a double water tube manometer. A difference of one inch of water indicated air pressure in the bag to be 5.2 lb/sq ft. Later, after a trial period when both manometers were used, only the single tube manometer was used (Fig. 4-5). A schematic of the pressure loading system is shown in Figure 4-6.

To verify the validity of calculating lateral load using indicated air pressure, strain gages were cemented to a machined section of the top and bottom threaded reaction rods. Good agreement was found between loads determined by this method and those calculated by using indicated air pressure.

4.8 DEFLECTION MEASUREMENTS

Displacement measurements were made at each support of the walls and at intermediate tenth points to obtain the shape of the elastic curve, using three different methods. In the first method, yardsticks were attached to the walls at such points (Fig. 4-7) and their positions were observed through a transit set (Fig. 4-8) with its line of sight parallel to the wall. This was reliable to the nearest 1/16 in.

A second method of measurement used dial gages calibrated in thousandths of an inch and with 3-in. travel (Fig. 4-9). A portable steel pylon, independent of the test frame, supported the dial gages. The gages were connected to the wall by a nylon coated steel wire tension line so that large deflections of the

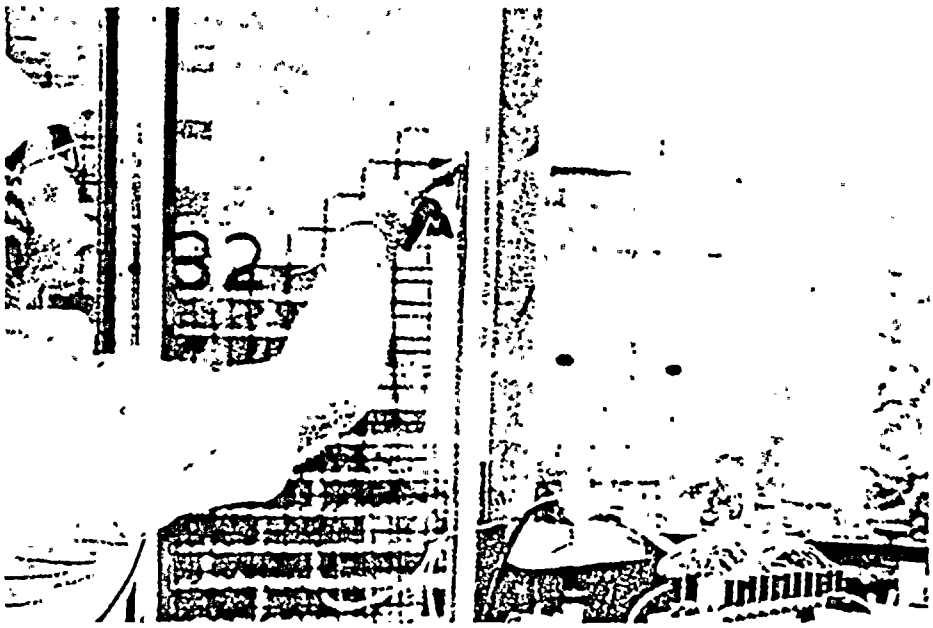


Fig. 4-5 Measurement of Air Pressure. Note U-Type Manometer to Measure Air Pressure on Right. Straight Tube Manometer on Left as a Check.

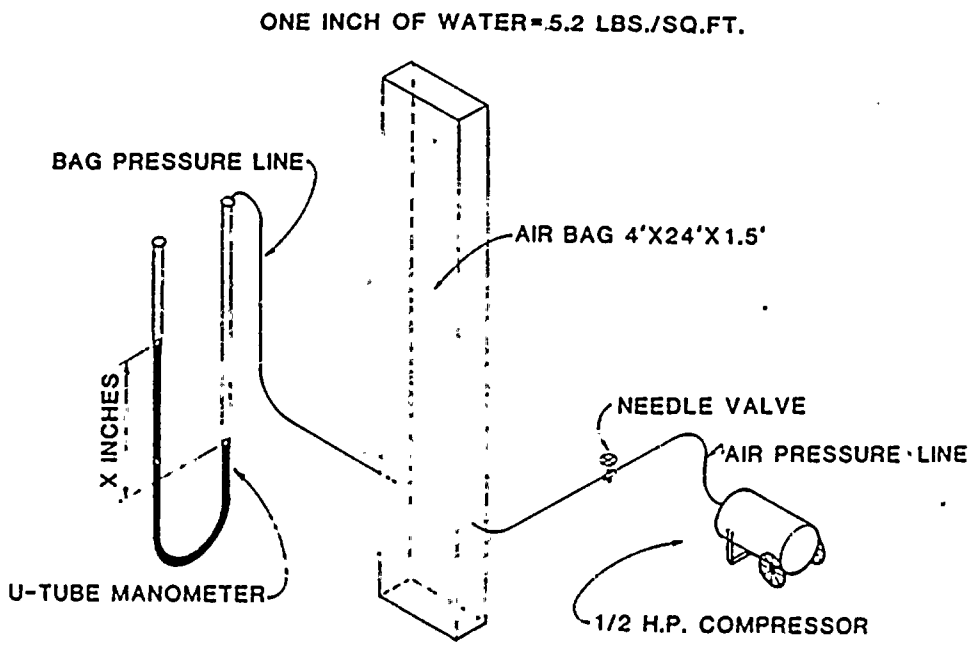


Fig. 4-6 Schematic of Lateral Load System.

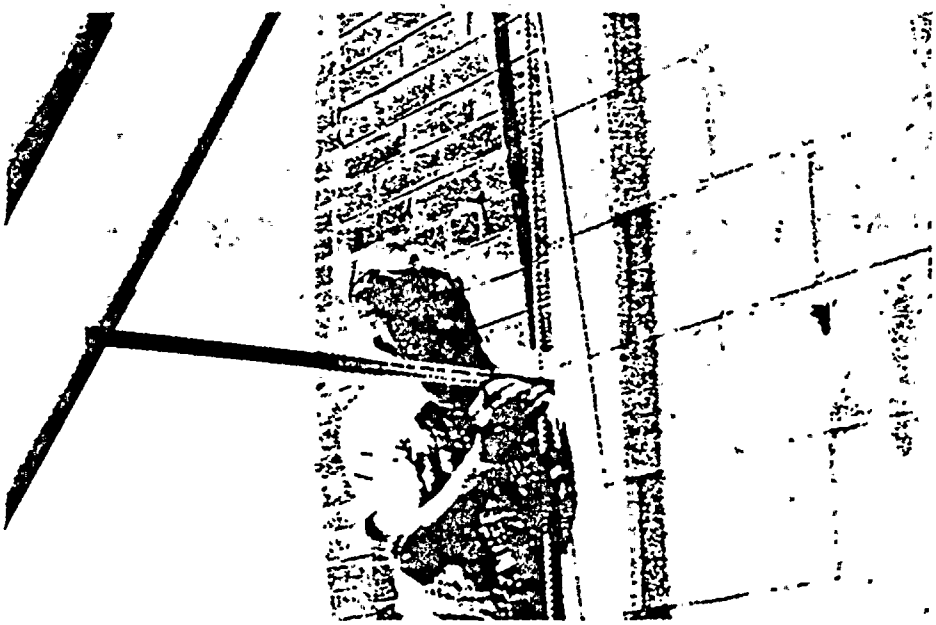


Fig. 4-7 Securing Yardstick to Side of Panel.

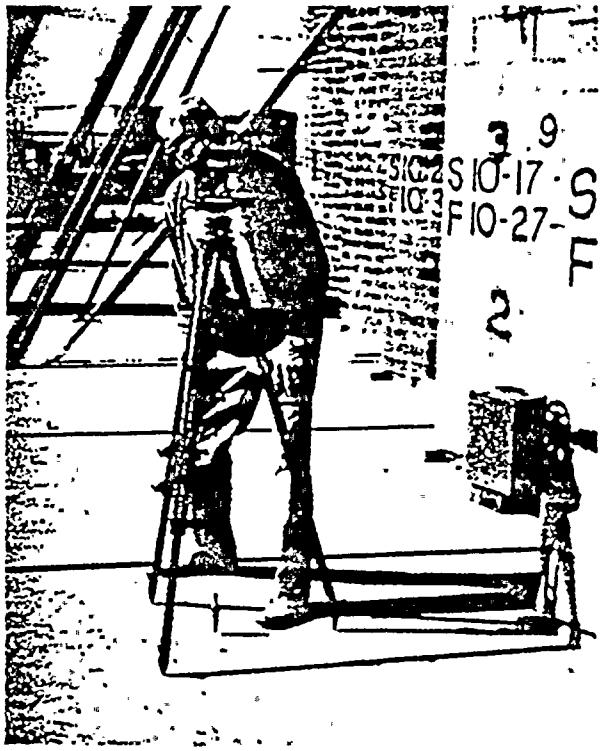


Fig. 4-8 Reading Yardsticks Through Transit as Wall Deflected.

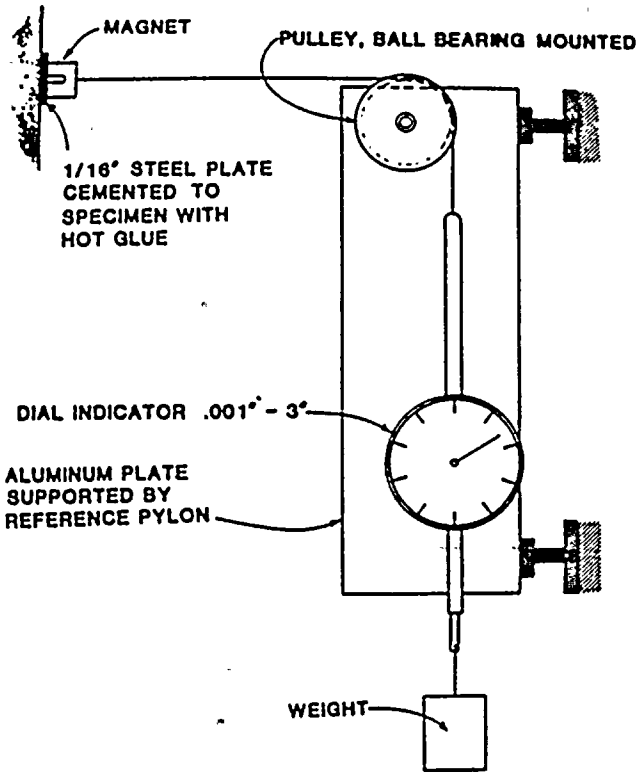


Fig. 4-9 Mounting of Dial Indicators.

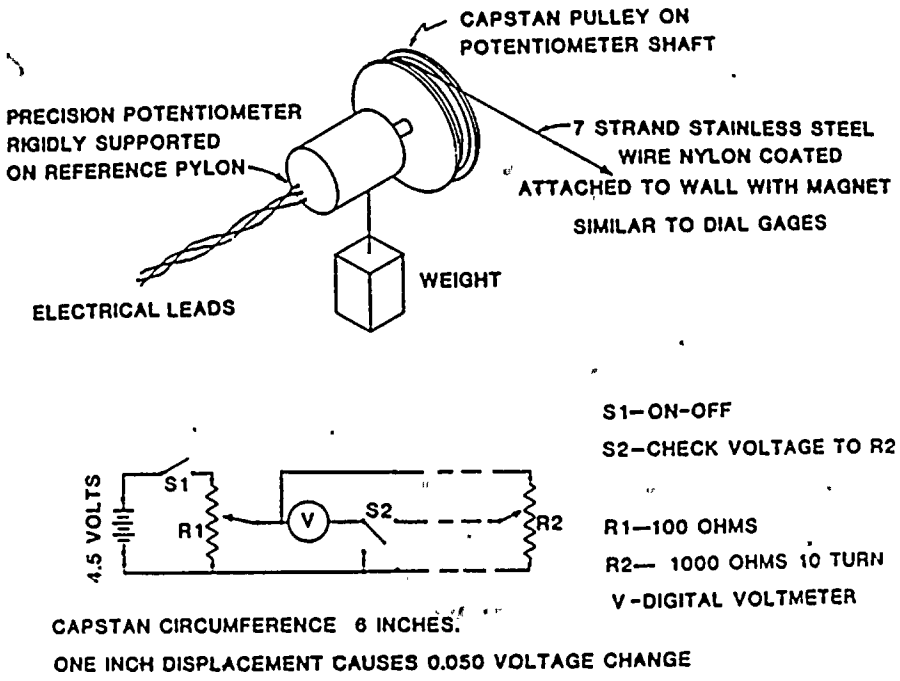


Fig. 4-10 Electrical Displacement Transducer.

walls would not damage them. However, reading the gages (through a telescope) proved to be slow and difficult, and, because the large deflections of the panels exceeded the 3" travel, the use of dial gages was abandoned.

A third method of measurement used a steel wire tension line from the wall, wrapped around a capstan pulley (Fig. 4-10) mounted on the shaft of a ten-turn precision potentiometer (1000 ohm), in order to measure over a large range (~50") with a resolution of 0.02 in. Accuracy of the method was bench-checked using a steel rule graduated in hundredths of an inch. Over a range of 2 ft, the measurements were found to be accurate and repeatable. In use, electrical leads from the 11 electric displacement transducers were taken from the reference pylon where these units replaced the dial gages to a switch box and digital voltmeter measuring to 0.001 volt. Thus, the deflection of all stations could be quickly and conveniently read at the same location where air pressure was controlled.

The electric transducers became the prime source of measurement, but transit readings were made in all cases to provide backup measurements. In two cases the electric units were not used because space limitations prevented use of the reference pylon.

Displacement measurements, along with time and temperature, were recorded at set intervals during both loading and unloading.

4.9 LOAD CONTROL

Although an attempt was made to use displacement control in loading the walls, load control was used in most cases (Fig. 4-11). Loading increments became smaller as maximum load was approached.

Monotonic loads were applied, and the displacements read as rapidly as possible. This process was repeated up to maximum value. Loading was stopped when it was judged that failure might be near. In two cases (both 6" concrete block walls), compressive failures did occur with complete rupture of the walls, but the safety cables prevented the walls from falling. The rupture occurred only after the steel yielded and the walls deflected 17.7 in. and 15.9 in., respectively. The last wall, the 4" clay block wall, 16' high, was intentionally carried to failure with a deflection of 17 in., which again occurred as a compressive failure but after the steel yielded and went into strain hardening condition.

In some of the initial tests, several walls were partially loaded, the bag deflated, and later the load was carried on up to

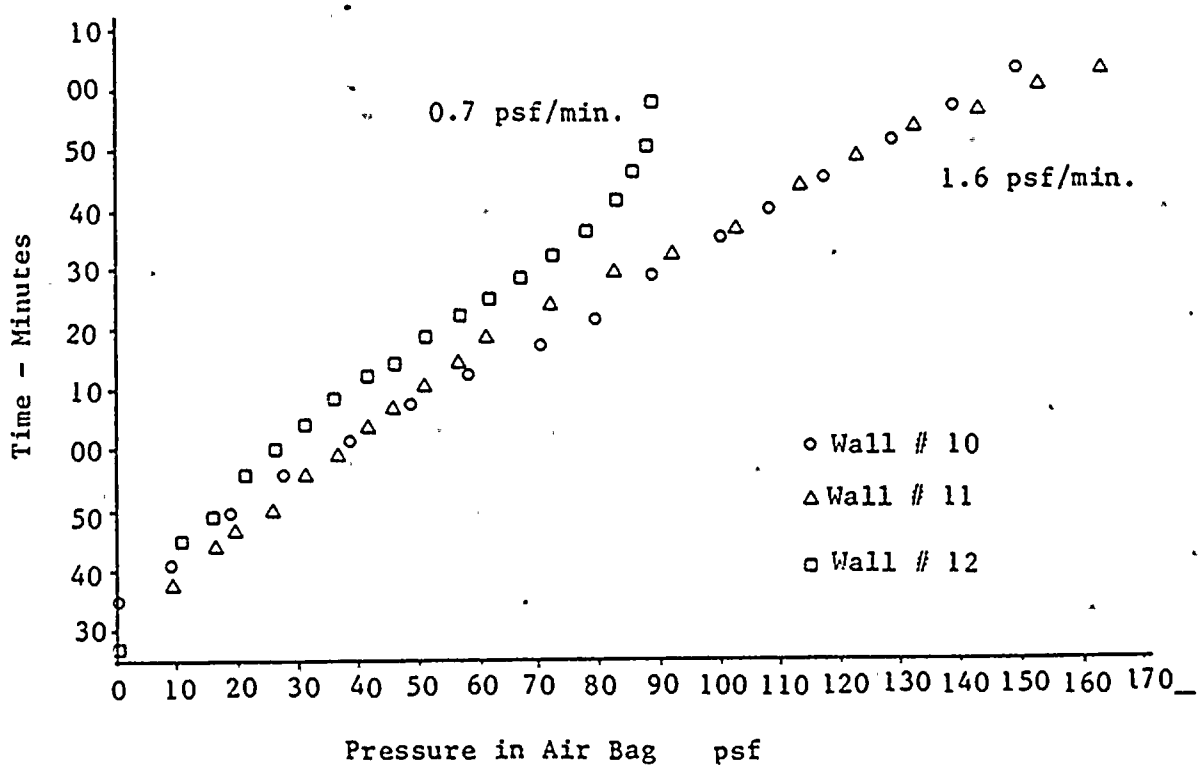


Fig. 4-11 Rate of Lateral Loading; 9.6"-Thick Brick Walls.

maximum. In most cases, displacement measurements were not made during unloading. A vacuum cleaner was used to assist in deflation.

4.10 TESTING ROUTINE

After the testing routine had become well established, the time from beginning of inflation to maximum load was about 2 hr. Initially, many problems were encountered in moving the test frame to a new wall and rigging for testing. At the end, it was possible to move the frame, assemble it to the panel, and test one wall per day. On one occasion, two walls were tested on the same day. A crew of three or four, along with the project director, participated in all tests.

SECTION 5

TEST RESULTS

The test results for the 30 panels are presented here as plots of the load vs. deflection for each panel. The deflection was measured for the mid-height of each wall. A tabulation of the test results is given in Table 5-1: materials, compressive strength, vertical load, lateral load and lateral deflection at yield, maximum lateral deflection, and the date tested.

The maximum lateral loads shown in the load deflection test result curves (Figs. 5-2 through 5-9, 5-11, and 5-12) are those recorded in the test load readings. They do not reflect correction for loss of contact of air bag with the test panel when deflections exceeded 7 to 8 in. (see Sec. 6.3 and the correction load deflection plot for Panel 24, Fig. 5-10).

The concrete tilt-up panels were reinforced with four #4 Grade 60 bars, and the masonry panels used five #4 Grade 60 bars. Crack patterns for the panels can be seen in Figures 6-8 and 6-9 in the next chapter.

5.1 CONCRETE MASONRY PANELS

Of the nine concrete masonry panels, there were three panels for each nominal thickness: 10", 8", and 6". The actual thicknesses were 9-5/8", 7-5/8", and 5-5/8". Three load deflection curves for each thickness are plotted on a single chart (Figs. 5-2, 5-3, and 5-4).

It was fairly evident when cracking developed, as the slope changed significantly. When the yield was reached, the slope shifted again, although this is less discernible on the charts.

TABLE 5-1. 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

Cracks for the masonry walls were essentially in the mortar joints (Fig. 6-9); upon unloading, these cracks tended to close. Depending upon the extent to which the wall was stressed beyond yield, rebound was approximately two-thirds of the total deflection.

5.2 CLAY BRICK

There were two thicknesses of clay brick masonry tested: 9.6" and 7.5". These were of normal two-wythe grouted brick construction. For each thickness, the curves were plotted on the load deflection diagram (Figs. 5-5 and 5-6).

Cracking occurred at the mortar joints, and because the mortar joints were relatively close to each other, a uniformly smooth deflection curve resulted. Again, upon unloading, these cracks tended to close.

5.3 CLAY BLOCK MASONRY

The three 5-1/2" clay block masonry panels exhibited normal deflection characteristics (Fig. 5-7). Two of the panels were loaded to deflections beyond 18 in.

5.4 CONCRETE TILT-UP WALLS

Four thicknesses of concrete walls were tested: 9-1/2", 7-1/4", 5-3/4", and 4-3/4". Three test specimens for each thickness are plotted on the load deflection charts (Figs. 5-8, 5-9, 5-11, and 5-12). Deflection characteristics of the concrete tilt-up walls were similar to those seen in the charts for the other panel constructions, with a sharp break at the cracking moment and then a change in slope at the yield moment where they flatten out.

The crack pattern was quite pronounced on the tilt-up concrete walls (Fig. 6-8), with cracks first developing at the center and then, with increased deflection, the cracks were propagated further. At maximum deflection, cracks were spaced apart approximately two times the thickness of the panel. Upon rebound, the cracks closed up somewhat but still were very much in evidence.

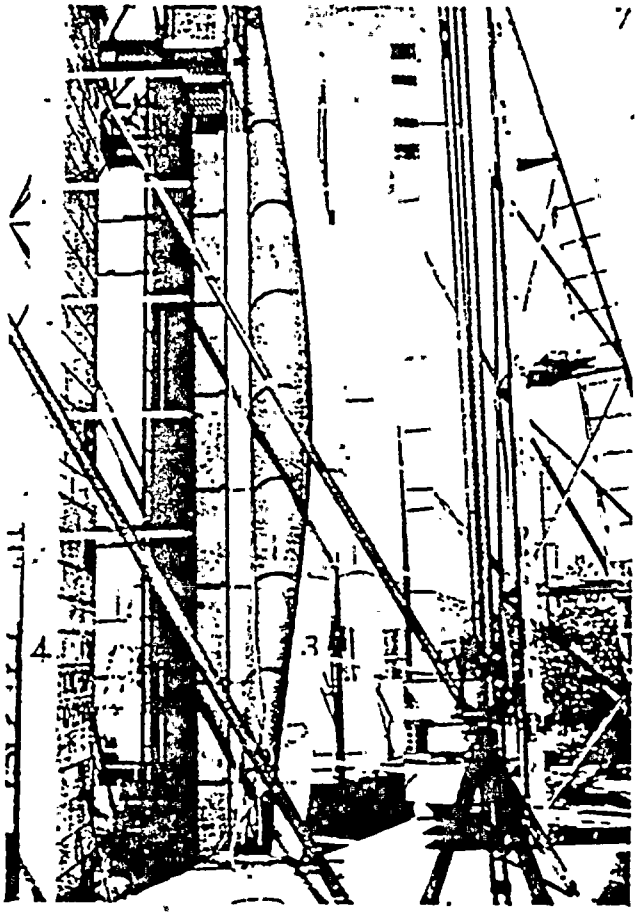


Fig. 5-1 Typical Test Panel.
(This 10"CMU Wall Deflected 18".)

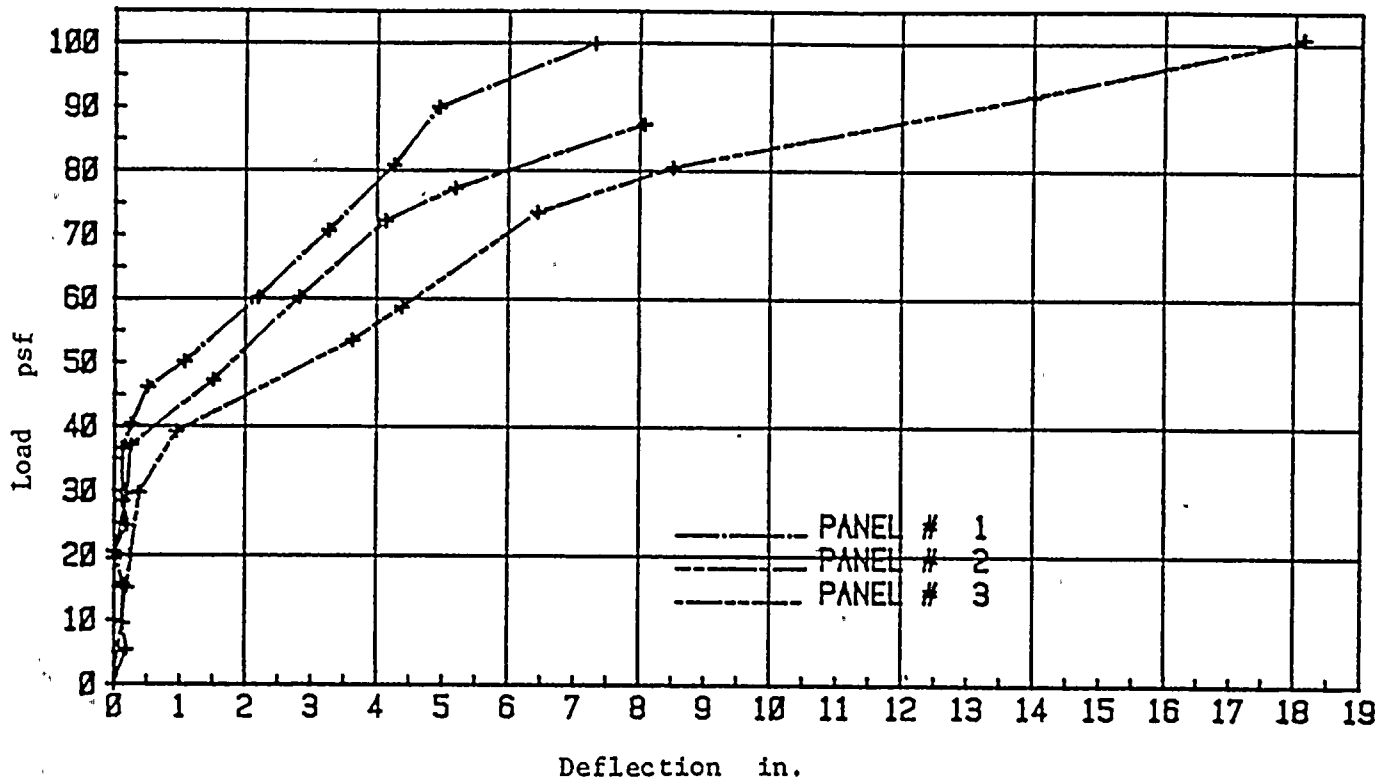


Fig. 5-2 Load - Deflection Curves, 10" Concrete Block Masonry

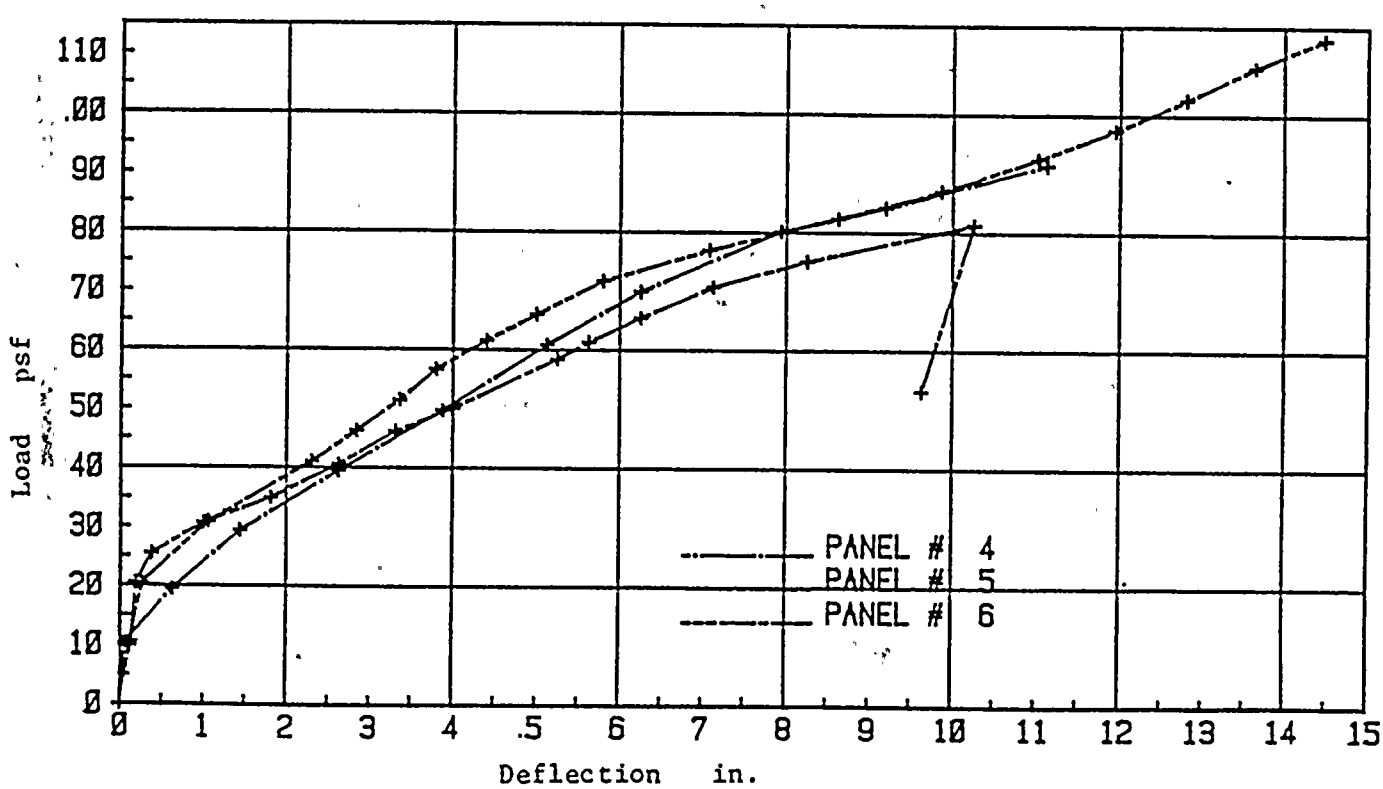


Fig. 5-3 Load - Deflection Curves, 8" Concrete Block Masonry

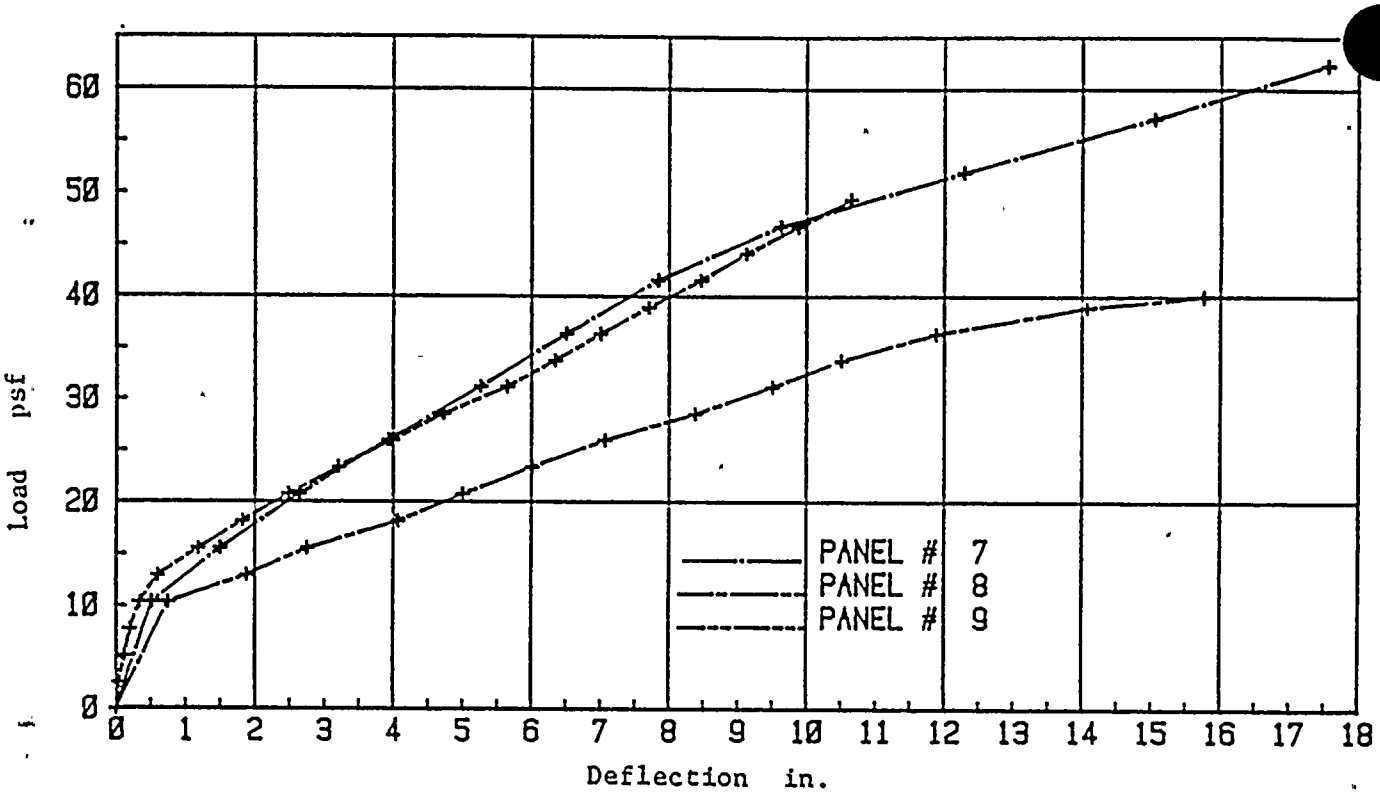


Fig. 5-4 Load - Deflection Curves, 6" Concrete Block Masonry

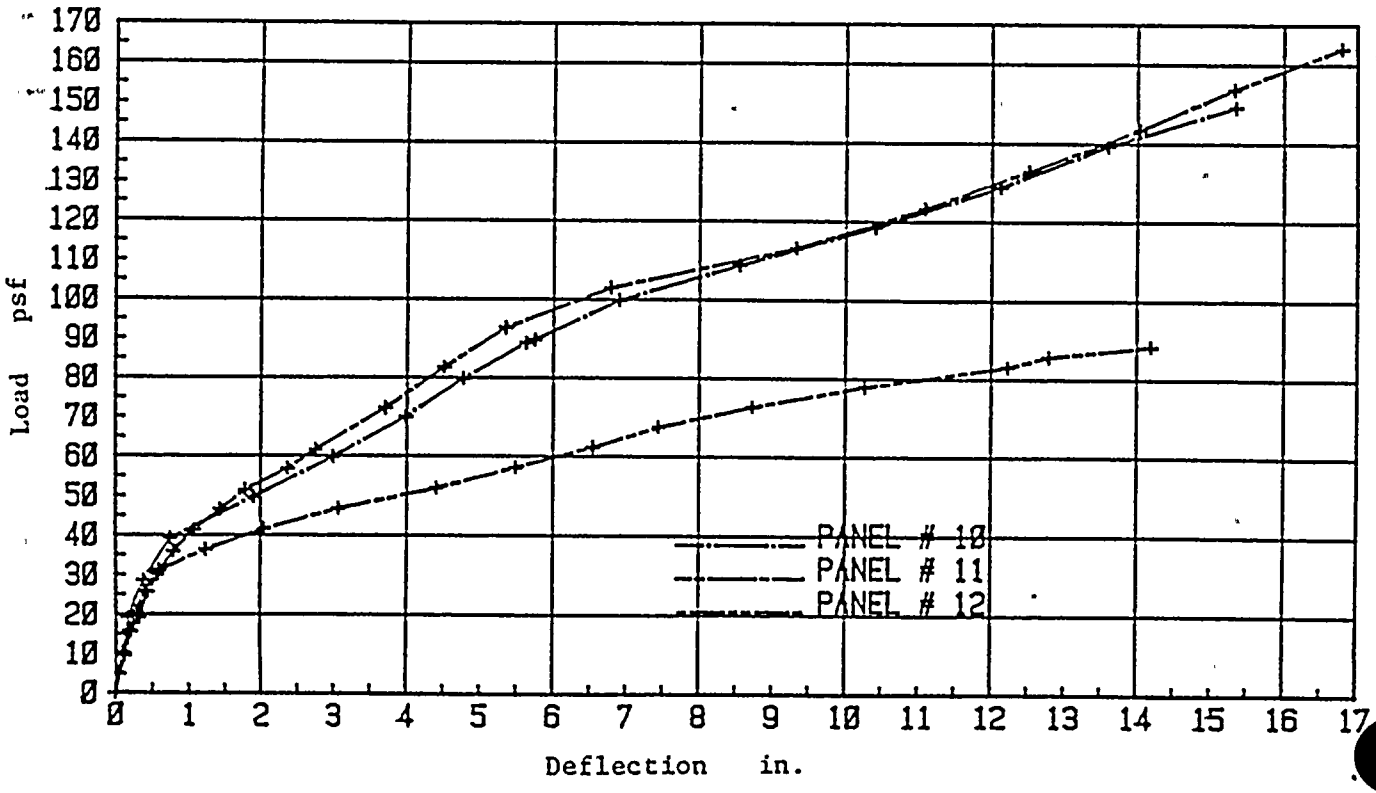


Fig. 5-5 Load - Deflection Curves, 9.6" two-Wythe Brick Masonry

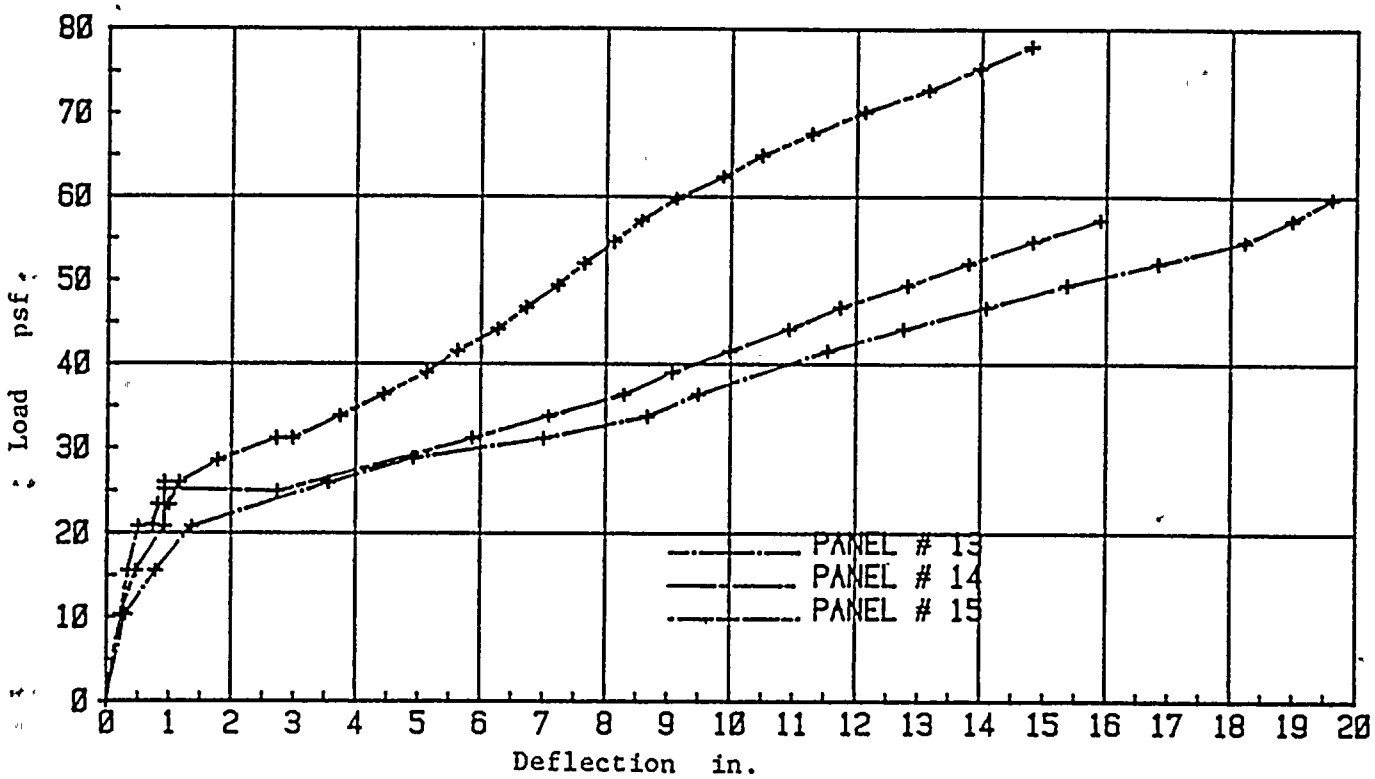


Fig. 5-6 Load - Deflection Curves, 7.5" two-Wythe Brick Masonry

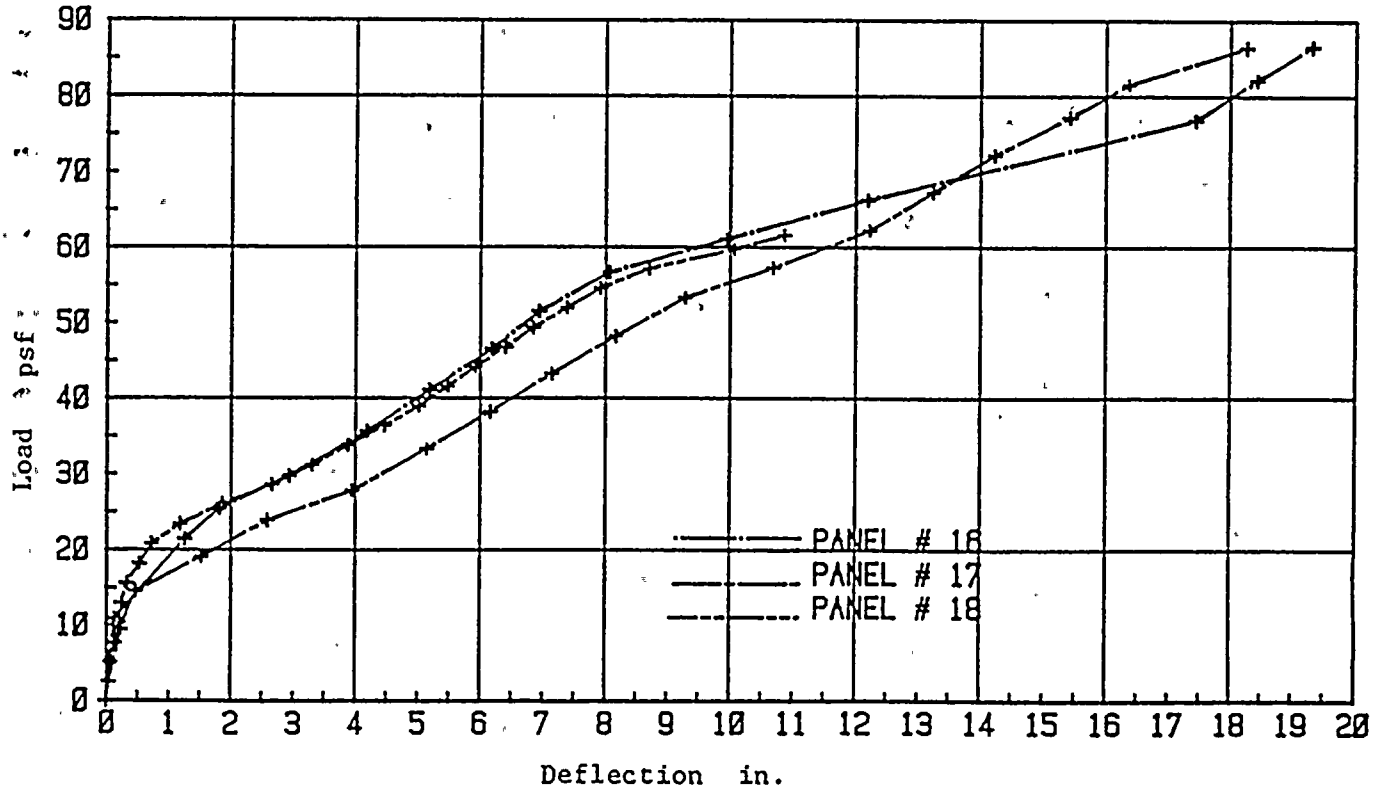


Fig. 5-7 Load - Deflection Curves, 5.5" Hollow Brick

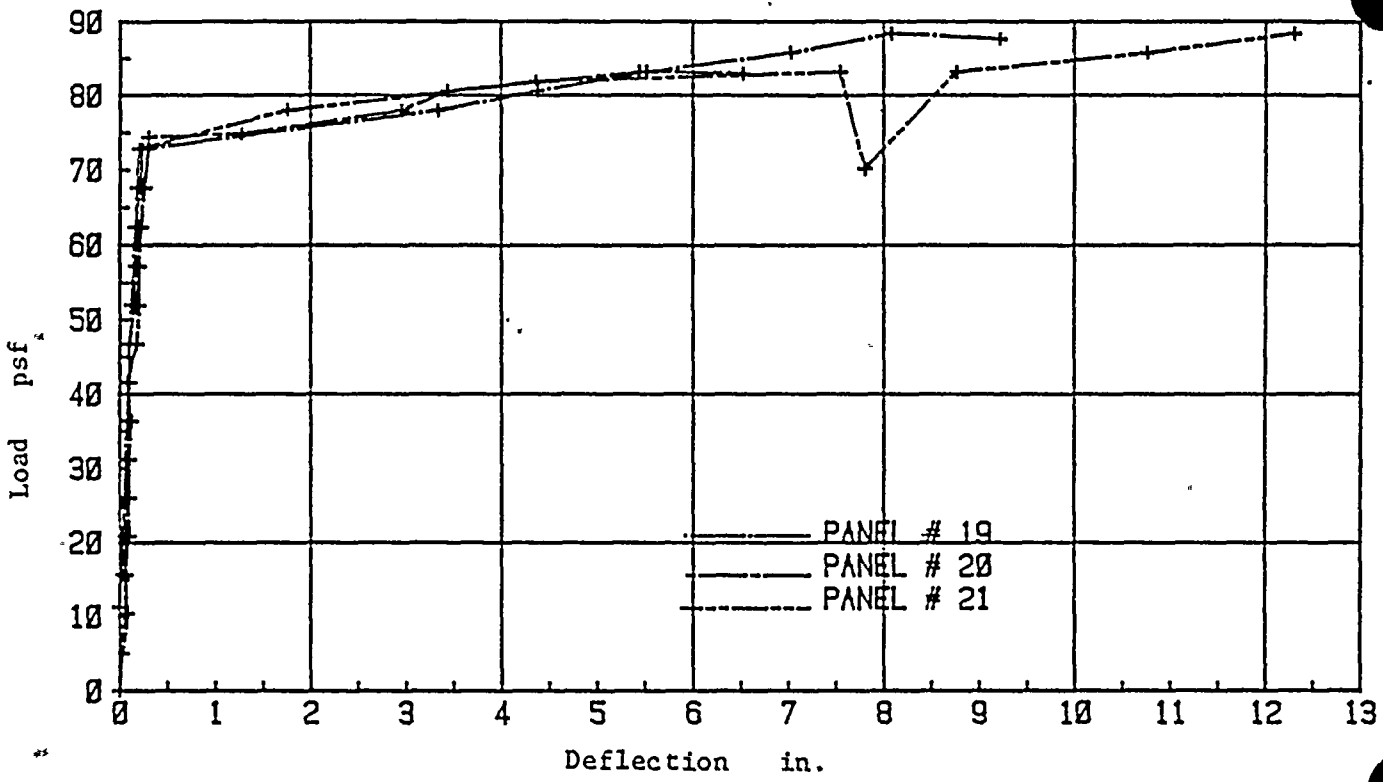


Fig. 5-8 Load - Deflection Curves, 9.50" Concrete Tilt-Up

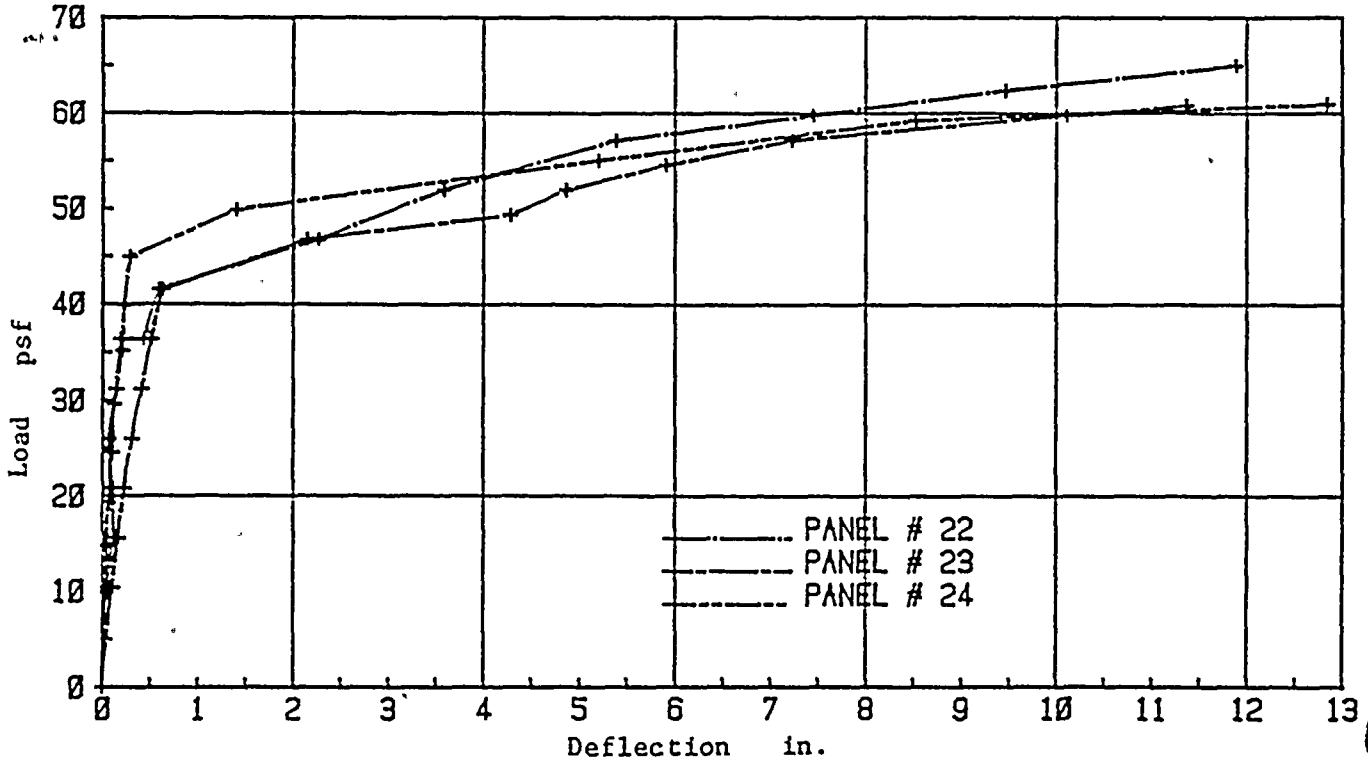


Fig. 5-9 Load - Deflection Curves, 7.25" Concrete Tilt-Up

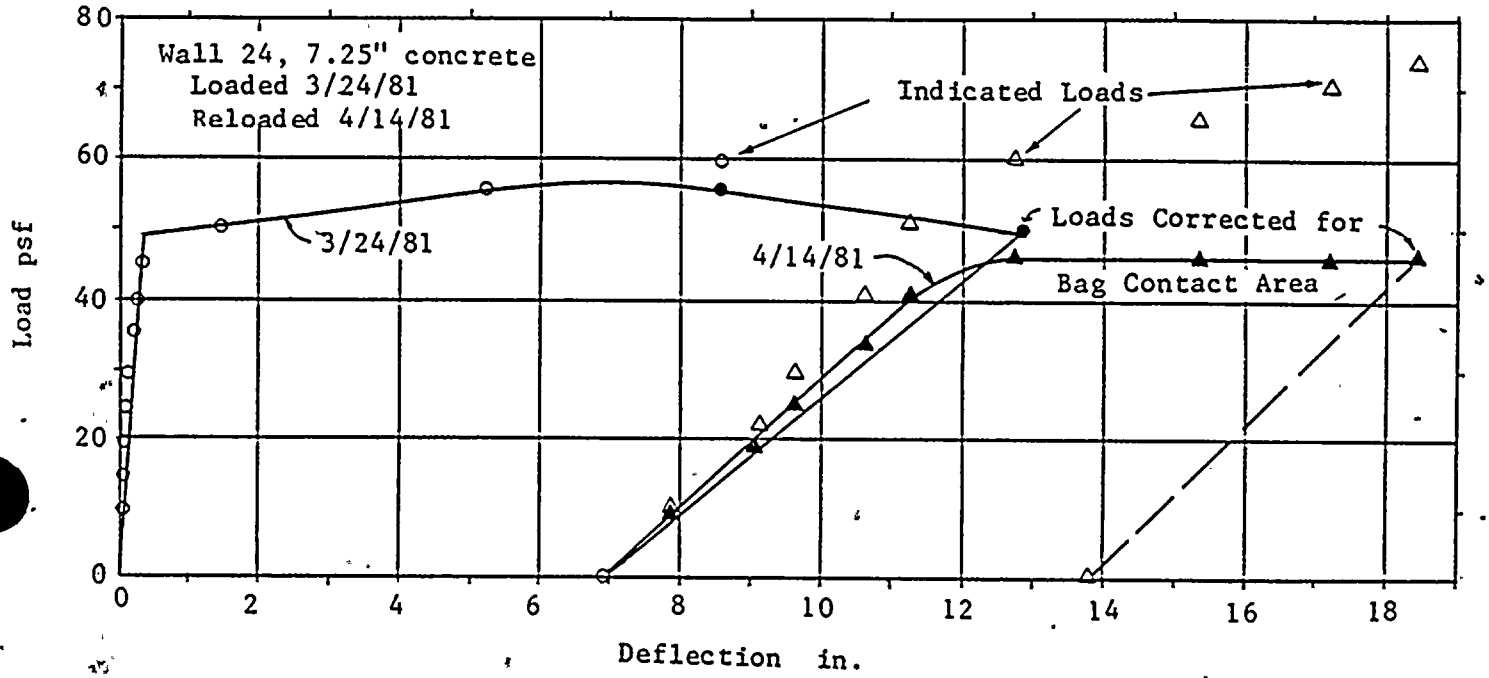


Fig. 5-10 Load-Deflection Plot Corrected for Loss of Air Bag Contact in Mid-Span Area.

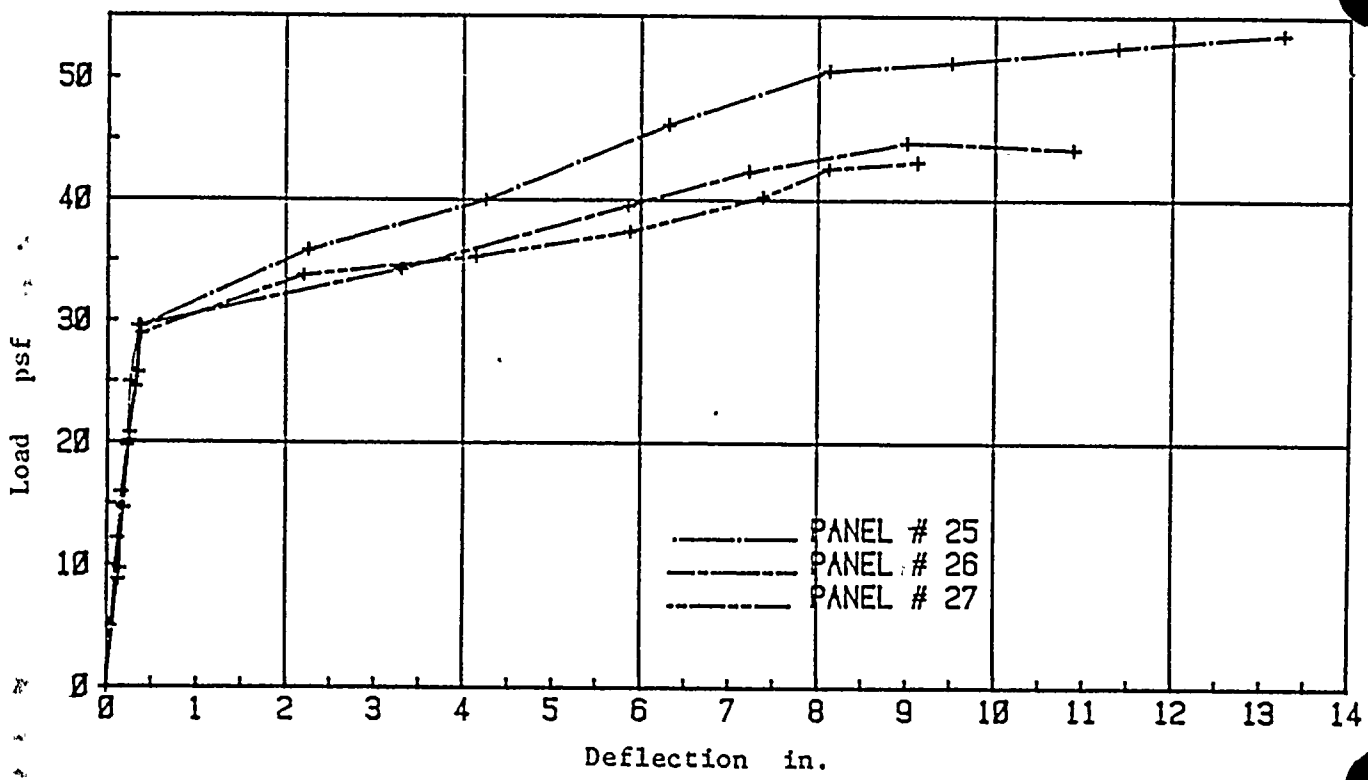


Fig. 5-11 Load - Deflection Curves, 5.75" Concrete Tilt-Up

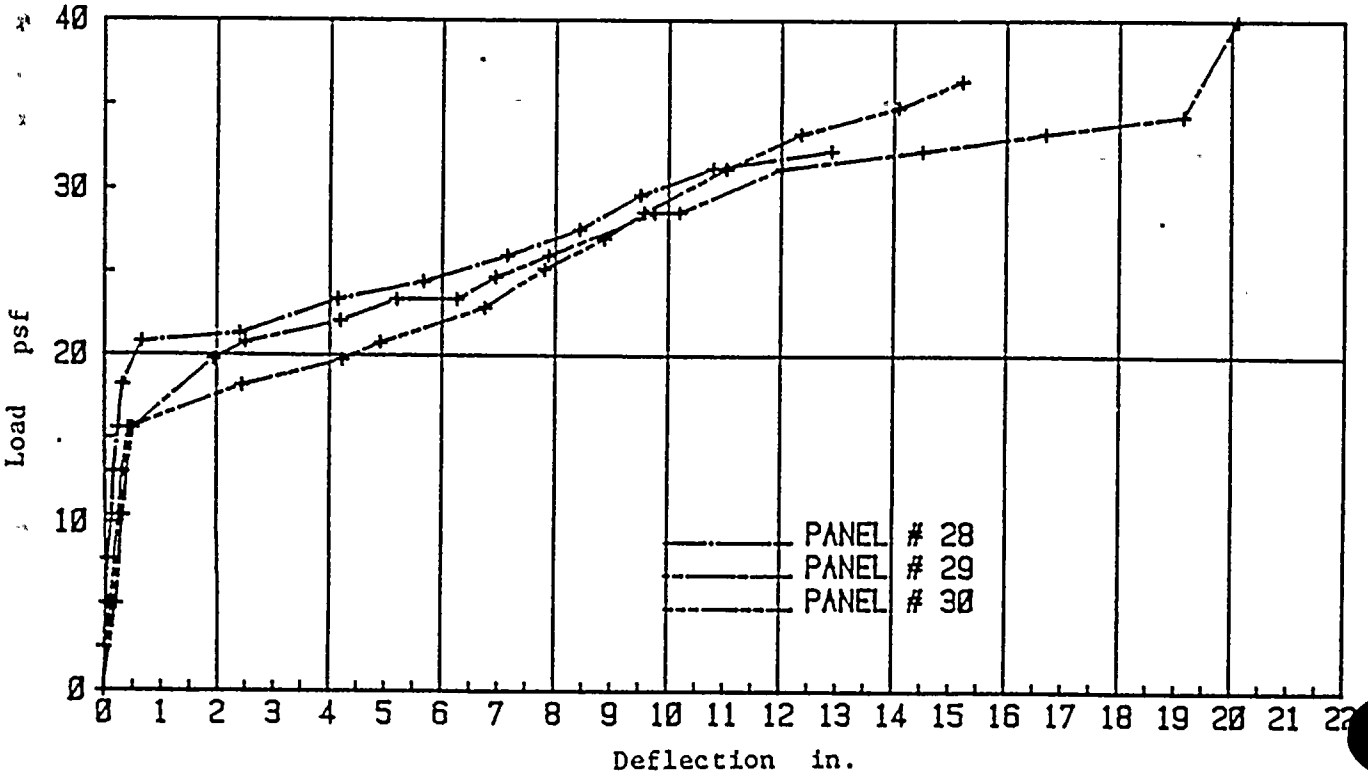


Fig. 5-12 Load - Deflection Curves, 4.75" Concrete Tilt-Up

SECTION 6

INTERPRETATION OF TEST RESULTS

6.1 GENERAL PERFORMANCE

Test results indicated general good performance of the wall panels in the range of the applied loads. Elastic or inelastic lateral instability was not noted during the tests. Wall panels continued to support additional lateral loading well beyond the deflection at which steel reached yield. Interpretation of various aspects of results is given in the following subsections.

6.2 LOAD-DEFLECTION CURVES

Typical idealized composite stress/strain relations for a reinforced panel are shown in Figure 6-1. A similar idealized relation holds for masonry except that the modulus of rupture (point 2) is considerably lower. The load deflection results as seen in Figures 6-2 to 6-5 indicate that walls with h/t ranging between 30 and 60 can resist 50% to 90% of their weight laterally. In addition, the lateral resistance is increasing even when deflections are extremely large. This could be due to strain hardening of the reinforcing steel.

The load-deflection average curves were plotted for each panel thickness, and the results are discussed below for all four types of walls tested.

Concrete. The average curves for concrete panels are given in Figure 6-2. The near vertical incline reflects the panel stiffness when the panel performs as an uncracked section. Panel yielding occurred at a deflection of approximately 3 in. for the 9-1/2" panels, 5 in. for the 7-1/4" panel, 7-1/2 in. for the 5-3/4" panels, and 8-1/2 in. for the 4-3/4" panels.

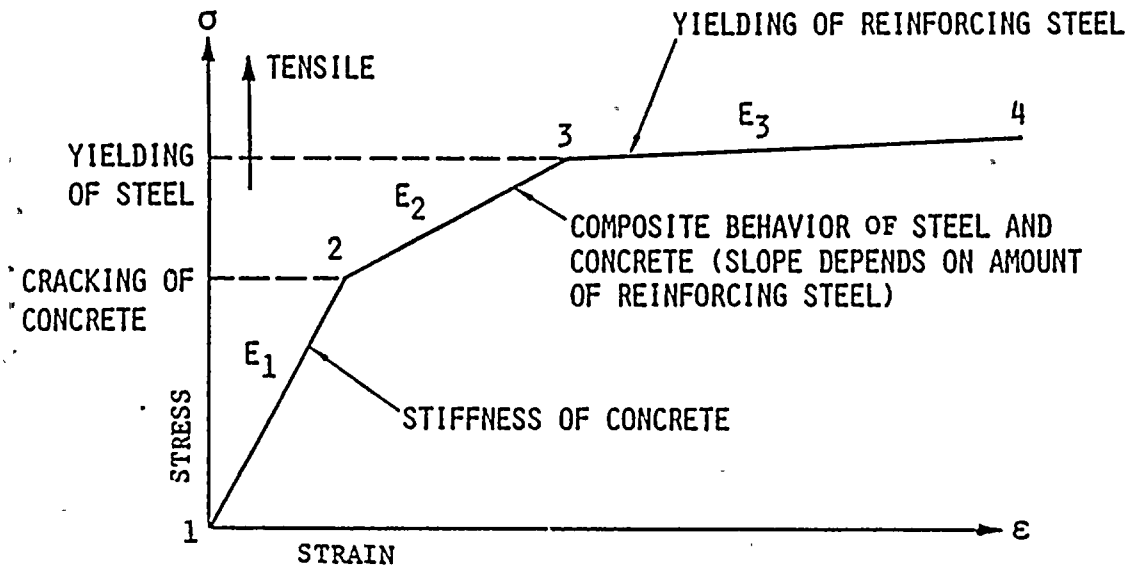


Fig. 6-1 Idealized Composite Stress-Strain Relations for Panel.

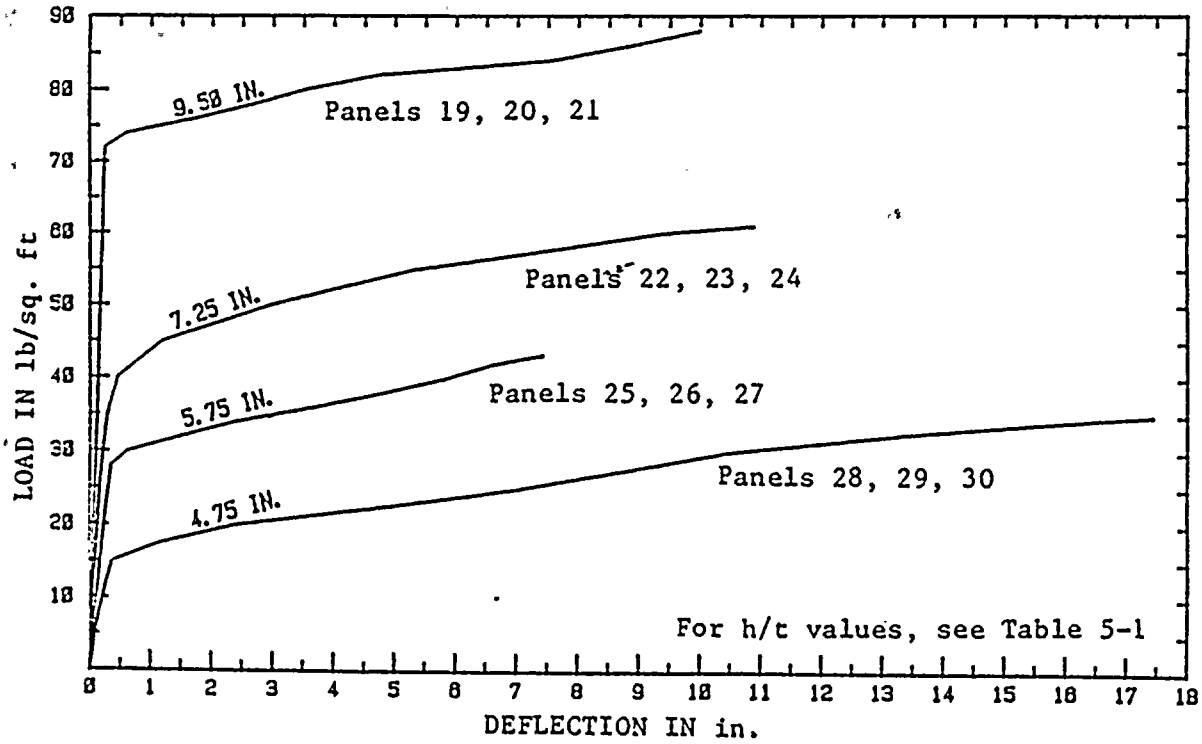


Fig. 6-2 Deflection in Inches at Mid-Height: Concrete Panels.

The onset of cracking (modulus of rupture) for all four thicknesses of concrete panels occurred at a deflection less than 1/2 in. The cracking load increased with increase in panel thickness. However, since all the panels had the same reinforcement, the percentage of reinforcement decreased with the increase in panel thickness. As a result, the thinner panels with higher percentage of reinforcement had a higher percent of increase in load capacity from cracking to yield (compared to the thicker panels).

The cracking load for the 9-1/2" panel was approximately 90% of the yield capacity of the panel, that is, the increase in capacity between cracking and yielding was only 10% of the total yield capacity of the panel.

For the 7-1/4" and 5-3/4" panels the increase from cracking to yield was 30%, while for the 4-3/4" panel the increase was 40%. So it is seen that, in essence, the relative percentage of steel reinforcement is the controlling factor in overall panel performance.

Concrete Block. The average load deflection curves for the three different thicknesses of concrete block walls are shown in Figure 6-3. The performance is similar to that of concrete panels except that the modulus of rupture was considerably lower. The panels, however, continued to sustain load, and yielded at a load considerably higher than the cracking load. For the concrete masonry walls of 10, 8, and 6 in., the steel reached the calculated yield stress of 70 ksi at deflections of 5, 6, and 10 in., respectively. Cracked performance started at approximately 50%, 35%, 20% of the yield, respectively.

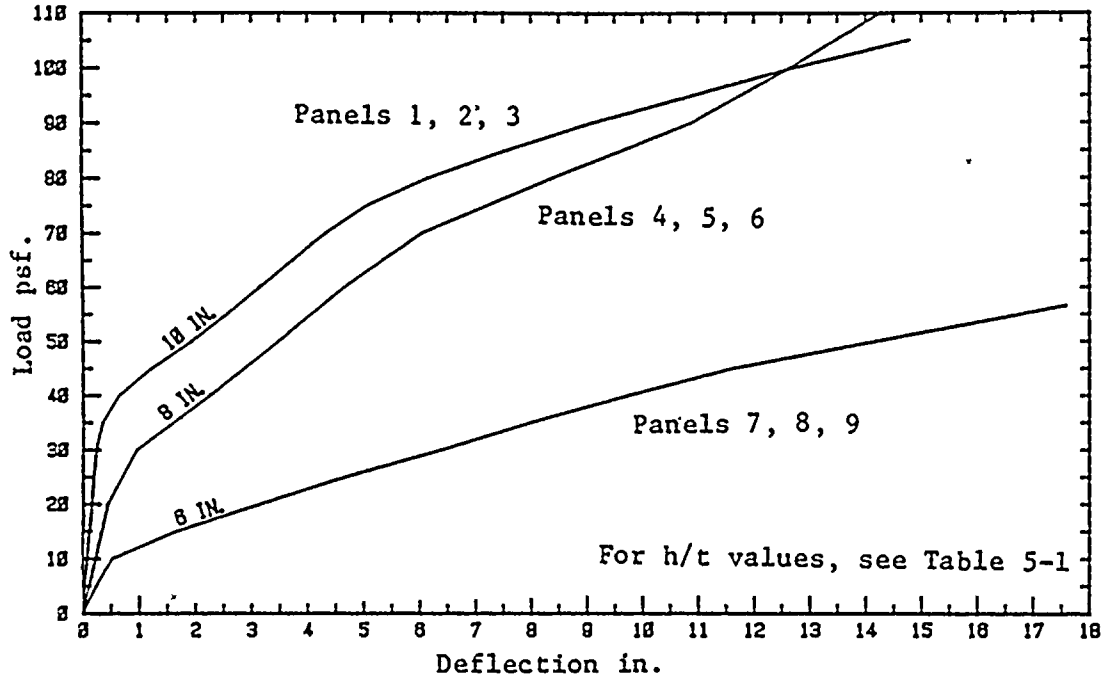


Fig. 6-3 Deflection at Mid-Height: 6", 8", 10" Concrete Block Masonry

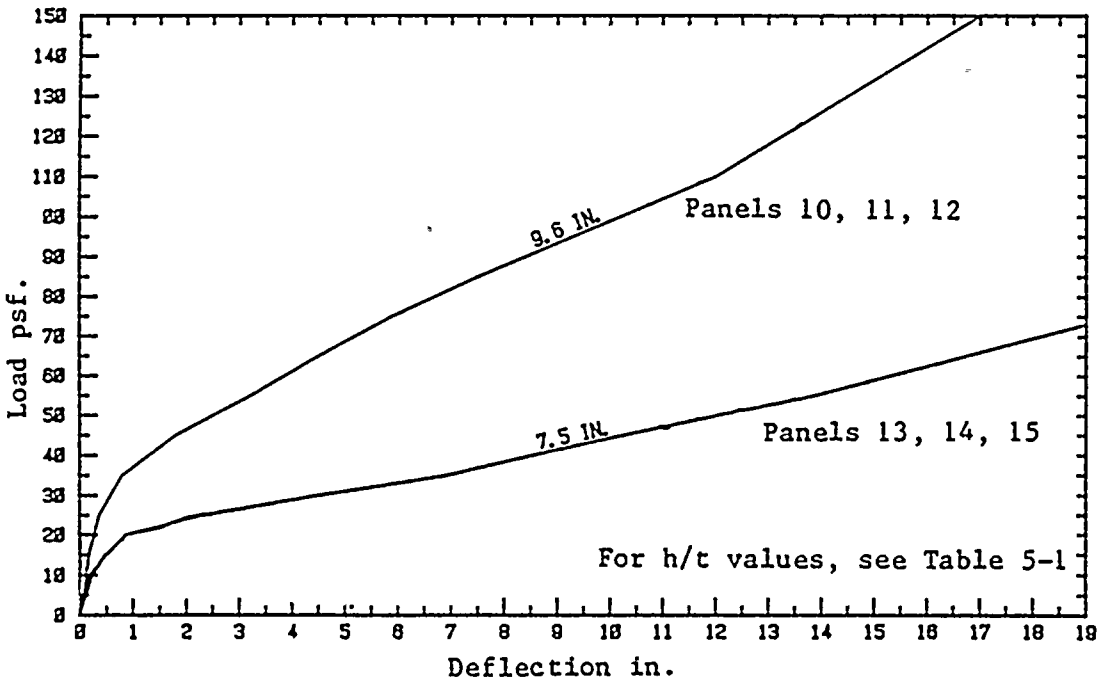


Fig. 6-4 Deflection at Mid-Height: 7.5" and 9.6" Brick Masonry

Clay Brick. The performance of the 9.6" and 7.5" clay brick panels (Fig. 6-4) was somewhat similar to that of the concrete block panels. The rupture of the panels occurred at an early stage, but the panels continued to sustain load. For the 9.6" clay brick, the yield load was considerably higher than the rupture load. The rupture load was approximately 40% of the yield load. Yield is considered to have occurred at deflections of approximately 6 in. for the 9.6" thick wall. The 7.5" thick wall demonstrated a low yield, with rupture approximately 60% of the yield load. Yield for the 7.5" thick wall occurred at approximately a 7 in. deflection.

Clay Block. The performance of the 5.5" clay block panels (Fig. 6-5) most nearly resembled the performance of the concrete block panels. The similar configurations of the units is responsible for this, i.e., the cross-webbings of each type of block. For the clay block panels, the rupture occurred early but the panels continued to sustain load. The rupture load was approximately 40% of the yield load. Yield occurred at approximately 8-1/2 in. for the clay block panels.

6.3 AIR BAG CONTACT AREA

There was a concern that a loss of contact area between the air bag and some side areas of the tested wall panel might have an effect on the test results. However, calculations have shown that loads below yield remained unaffected and loads near yield were only slightly affected.

The question arose when the notes for wall panel 17 showed a 6-3/4 in. separation between the plywood frame and the tested wall, whereas for wall panel 21 the notes show a separation of about 3 in. This was because the air bag was hung from the steel tubes in the former case and from the ledger angle in the latter

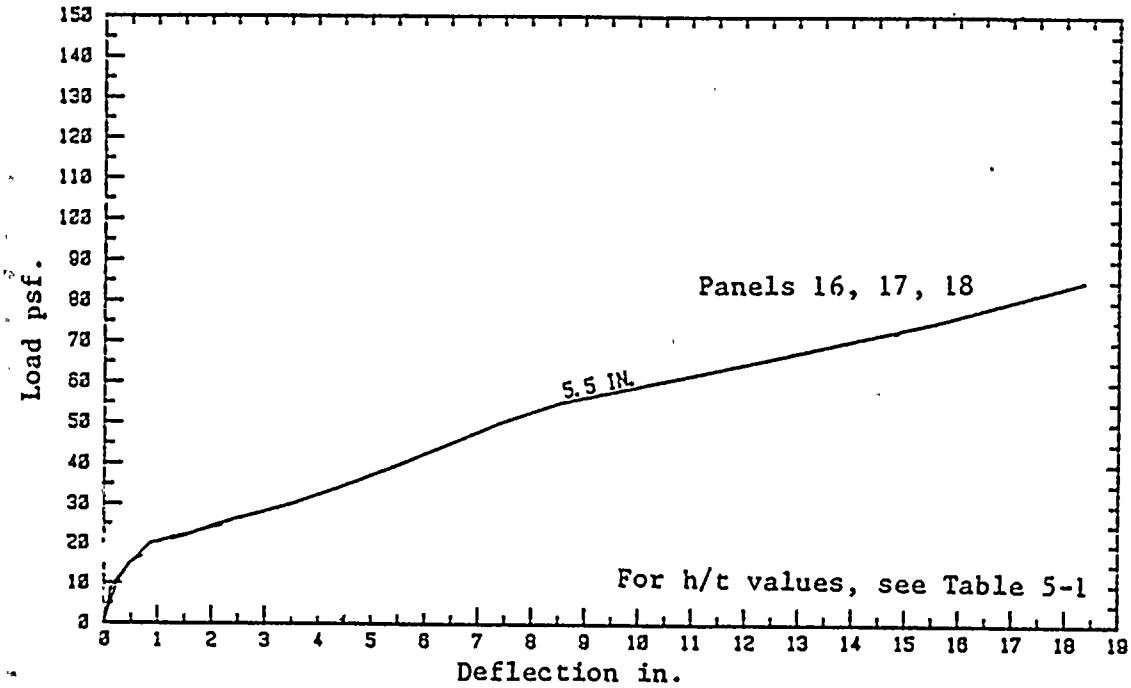


Fig. 6-5 Deflection in Inches at Mid-Height: 5.5" Hollow Brick Masonry.

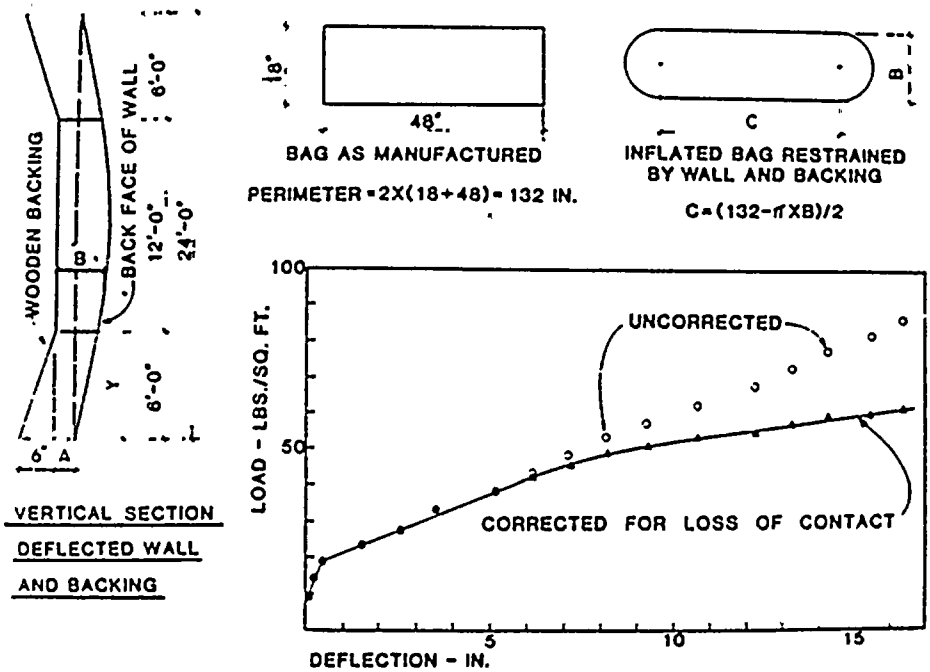


Fig. 6-6 Bag Contact Area.

case. From this time on, the floor positions were marked and the space between wall and plywood was consistently near the 3-in. mark.

Figure 6-6 shows the method used to calculate a correction factor for wall panels 7, 13, and 29. Data from the diagram of contact area was used to calculate the resulting bending moment. From this, the pressure necessary to produce equal moment at midspan for uniform load was calculated to 44.0 psf.' The corrected moment and the moment for this uniform load were then compared for each foot of height of the panel. The average ratio of uniform load moment to corrected moment was 0.96; hence, the effects of this discrepancy proved negligible.

6.4 CRACKING PATTERN

Investigations of crack spacing in reinforced concrete members have indicated that crack spacing decreases with increasing applied load. After stress reaches its critical value, the spacing of visible cracks remains approximately constant. For the average minimum crack spacing, $t < \text{crack spacing} < 2t$, where t is the thickness of the concrete cover (Broms, 1965).

Typical cracking patterns for concrete tilt-up panels and for block walls are shown in Figures 6-8 and 6-9, respectively. For the block walls, the cracking occurred through bedjoint at the mortar block interface on the side opposite from the head joint. These cracks propagated to approximately 3/4 of the thickness toward the compression side.

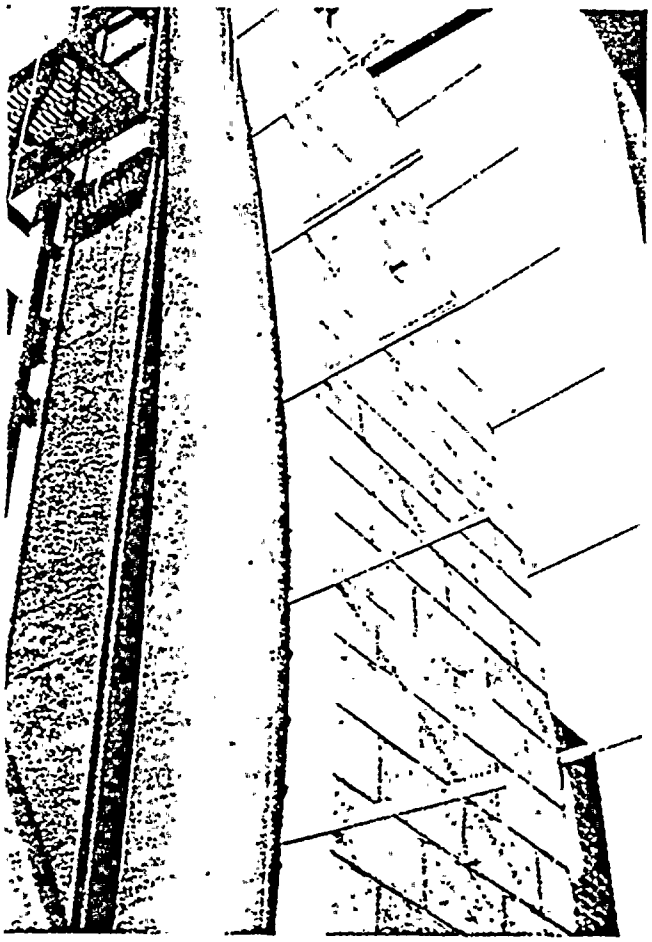


Fig. 6-7 Loss of Edge Contact of Air Bag with Wall Panel at the Middle Third.

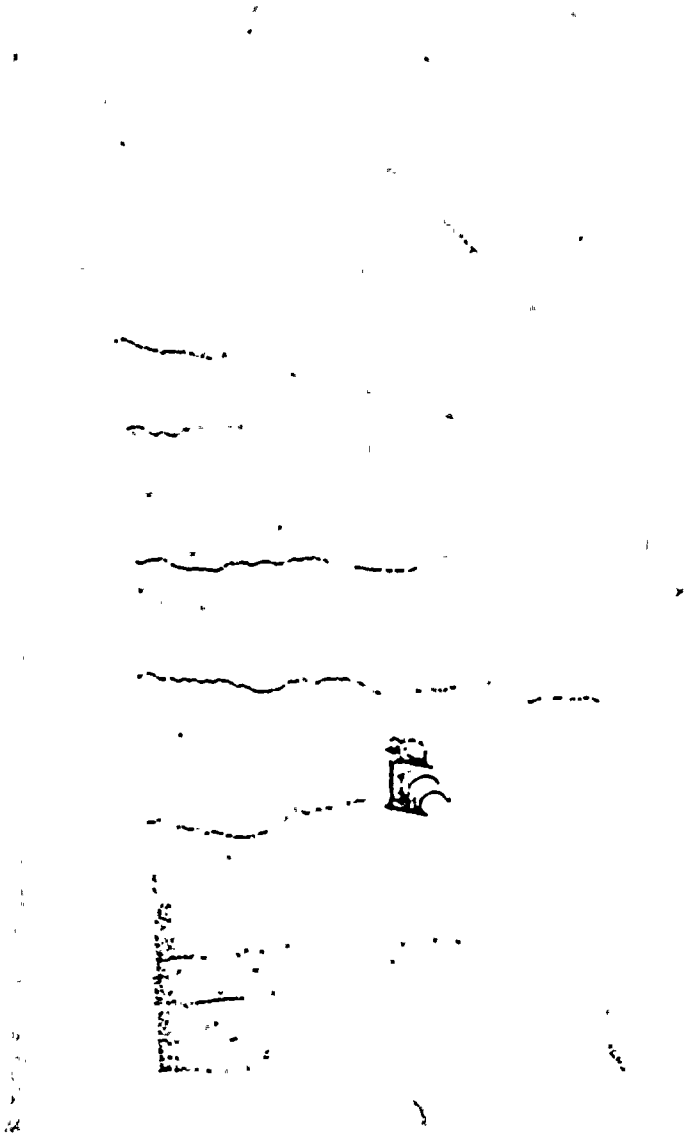


Fig. 6-8 Typical Crack Pattern in Tilt-Up Wall After Major Deflection.

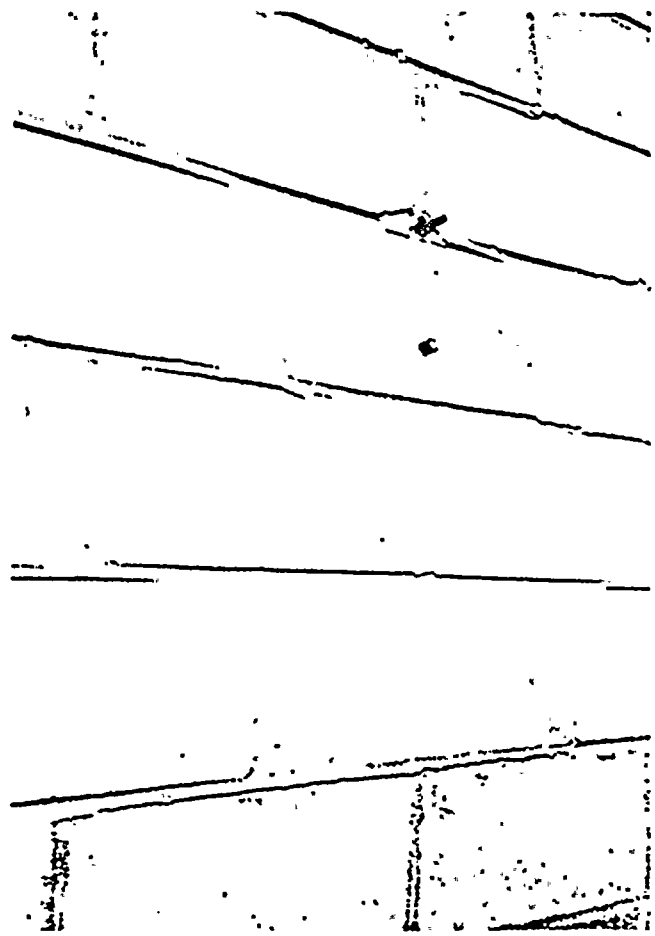


Fig. 6-9 Typical Crack Pattern in Concrete Masonry Walls Under Large Deflection.

6.5 REBOUND

Unloading and reloading were carried out on concrete panel Nos. 24 and 27, with thicknesses of 7-1/4" and 5-3/4" respectively. As shown in Figure 6-10, Panel 24 was loaded to a total midspan deflection of 13 in., which was 6 in. beyond the point where the reinforcing steel reached the yield stress of 70 ksi. Pressure was released and a rebound of approximately 6 in. was recorded. The wall had a permanent set of 6-3/4 in. Twenty days later the wall was again loaded, and its deflection path was just a little steeper than the first rebound curve until yield level was reached, at which time a shallower load deflection curve occurred until a total deflection of 18 in. was reached. The lateral load was then released, and the wall again rebounded 6 in. When compared with Panel 27 in Figure 6-11, it seems that the twenty day wait resulted in the stiffening of the panel.

Panel 27 was loaded to 43 lb and a deflection of 9 in. This was just beyond where the calculated steel stress reached the yield level. The load was removed and rebound readings taken. The wall rebounded 5 in., to a permanent set of 4 in. even though the steel had just reached yield at mid-height of the panel. If this panel experienced a near yield level loading, a 4 in. set would be expected afterwards. Two hours later the panel was reloaded to a lateral load of 40 psf, unloaded to 20 psf, and then brought up again to the yield level. When the wall reached the yield level it increased its rate of deflection until a total deflection of 16 in. was reached at a lateral load of 45 psf. This panel showed a softening effect due to rebound and reloading.

The reloading appears to occur on a slope close to the unloading slope. In addition, the area under the loading-unloading curve provides an indication of the nonlinear hysteretic damping of the system. This data can be interpolated

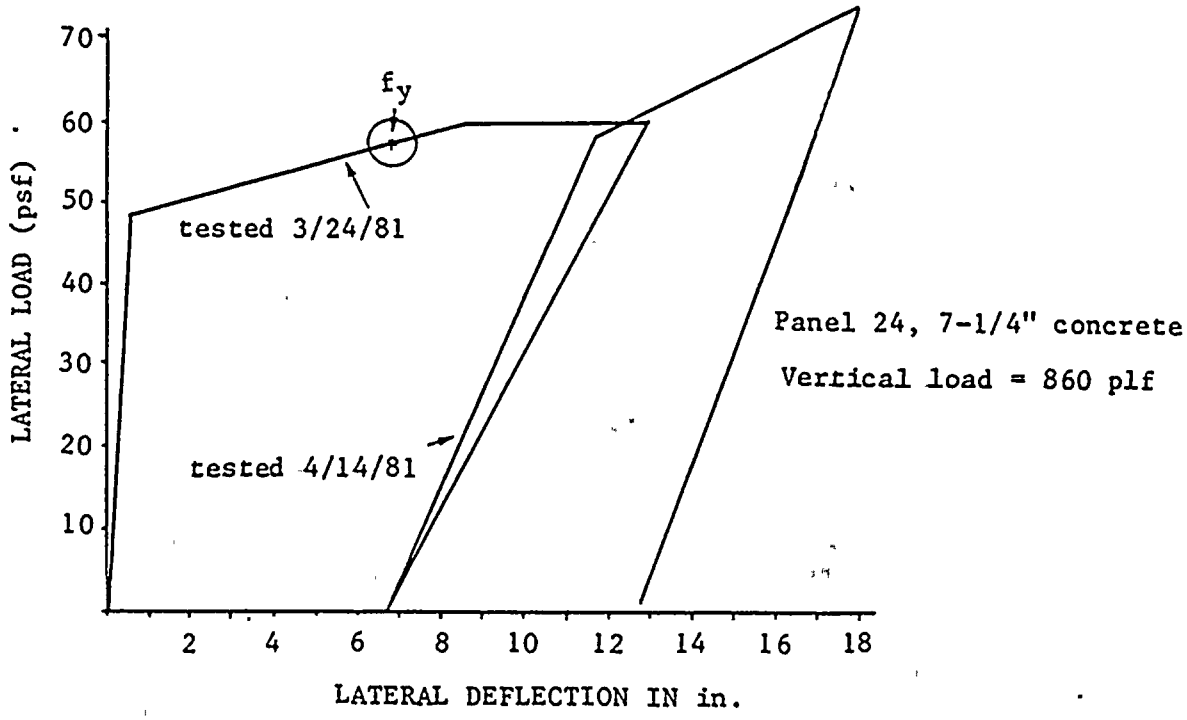


Fig. 6-10 Lateral Deflection at Mid-Height (Inches).

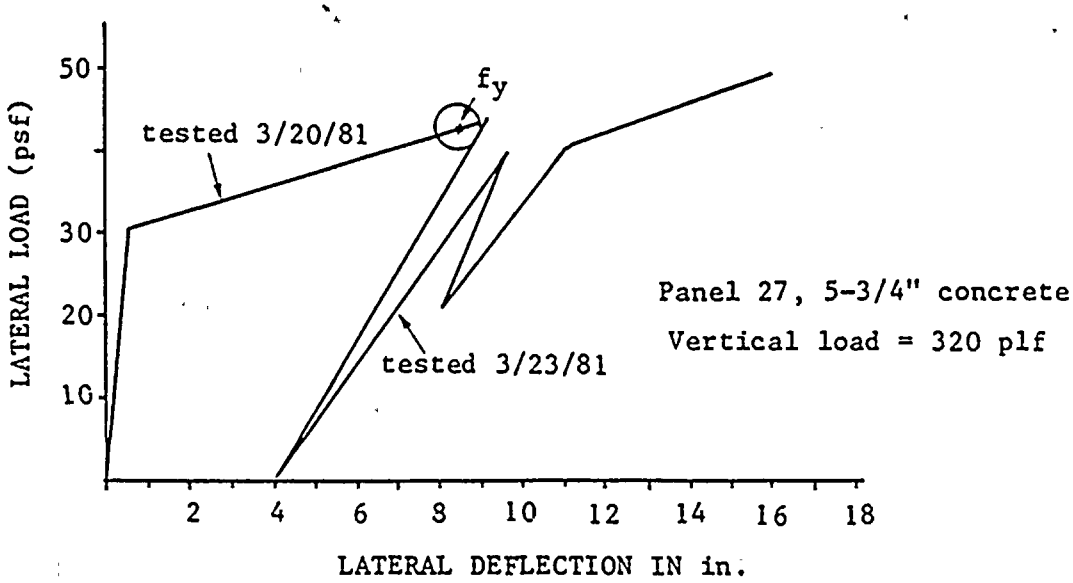


Fig. 6-11 Lateral Deflection at Mid-Height (Inches).

to provide preliminary values to be used in the study of the dynamic performance of such panels.

6.6 SECONDARY MOMENTS DUE TO DEFLECTIONS (PΔ EFFECT)

Evaluation of effects of PΔ moments is of primary interest in this program. The ratio of the PΔ moment when compared to the moment caused by vertical and lateral load is a good indicator of this secondary effect.

The procedure used for finding the PΔ moment at the midheight of the panel is given by

$$P\Delta \text{ Moment} = \text{Roof Load} \times \Delta + \text{Wall Weight Above} \times \Delta \qquad (6\text{-}1)$$

where

PΔ moment	=	Additional moment at the midheight due to deflection, lb-ft
Roof load	=	Vertical load per foot acting on the ledger angle (320 lb and 860 lb/ft were used in the the present tests)
Δ	=	Midheight deflection, ft
Wall weight above	=	Weight of the wall above the midheight, lb/ft

The percentage of PΔ moment can be found from the expression

$$\% \; P\Delta \text{ Moment} = \frac{P\Delta \text{ Moment} \times 100}{\frac{w\,h^2}{8} + \frac{P \cdot e}{2}} \qquad (6\text{-}2)$$

where

% PΔ Moment = % ratio of PΔ moment to the applied moment

w = Applied lateral load, psf;

h = Panel height (24 ft)

P = Roof load/ft (320 lb/ft or 860 lb/ft)

e = Roof load eccentricity, ft (e = 0.25 ft + 1/2 x panel thickness in ft)

The percentage of PΔ moment found from Equation 6-2 is plotted versus normalized deflection in Figures 6-12 through 6-14. The deflection plotted on the abscissca has been divided by the height of the panel. In a normal wall design the horizontal deflection to height ratio is usually less than 0.005. It never exceeds 0.01. The PΔ plots in the above figures show that when deflection to height ratio is less than 0.01, the percentage of PΔ moment is less than 15%, i.e.,

$$\text{for } \Delta/h < 0.01 \text{ , \% P}\Delta \text{ Moment} < 15\% \qquad (6\text{-}3)$$

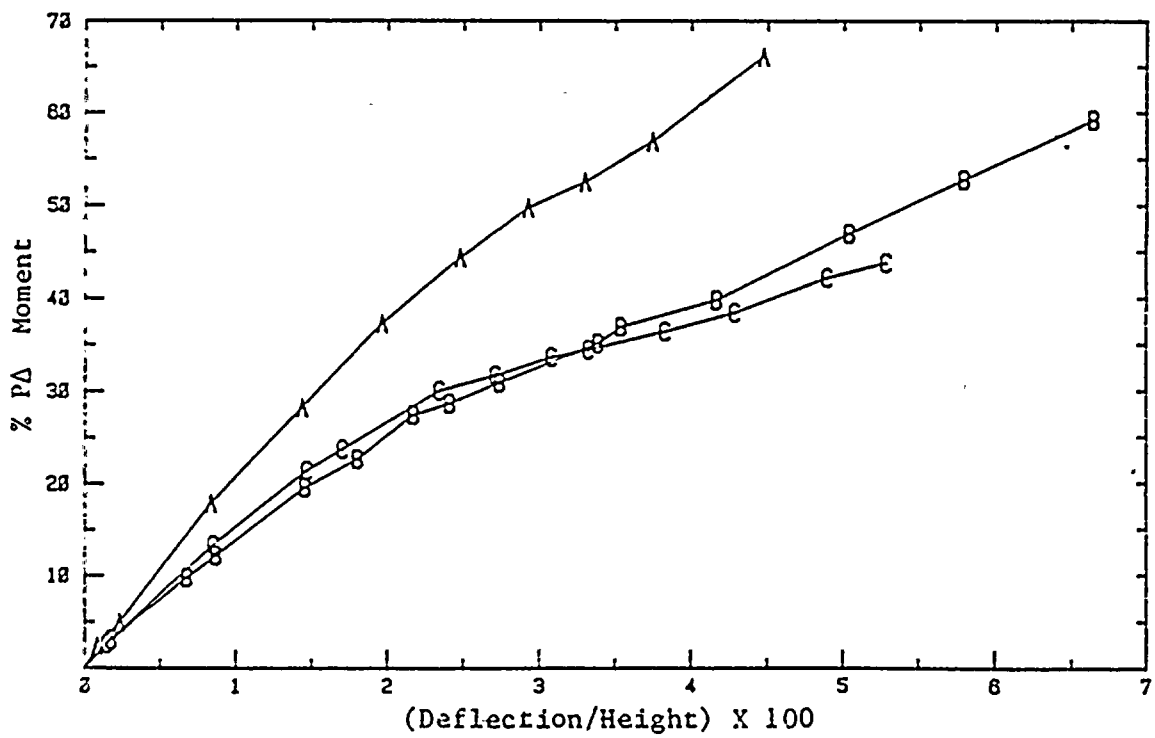


Fig. 6-12 % PΔ Moment, 4.75" Concrete Tilt-up

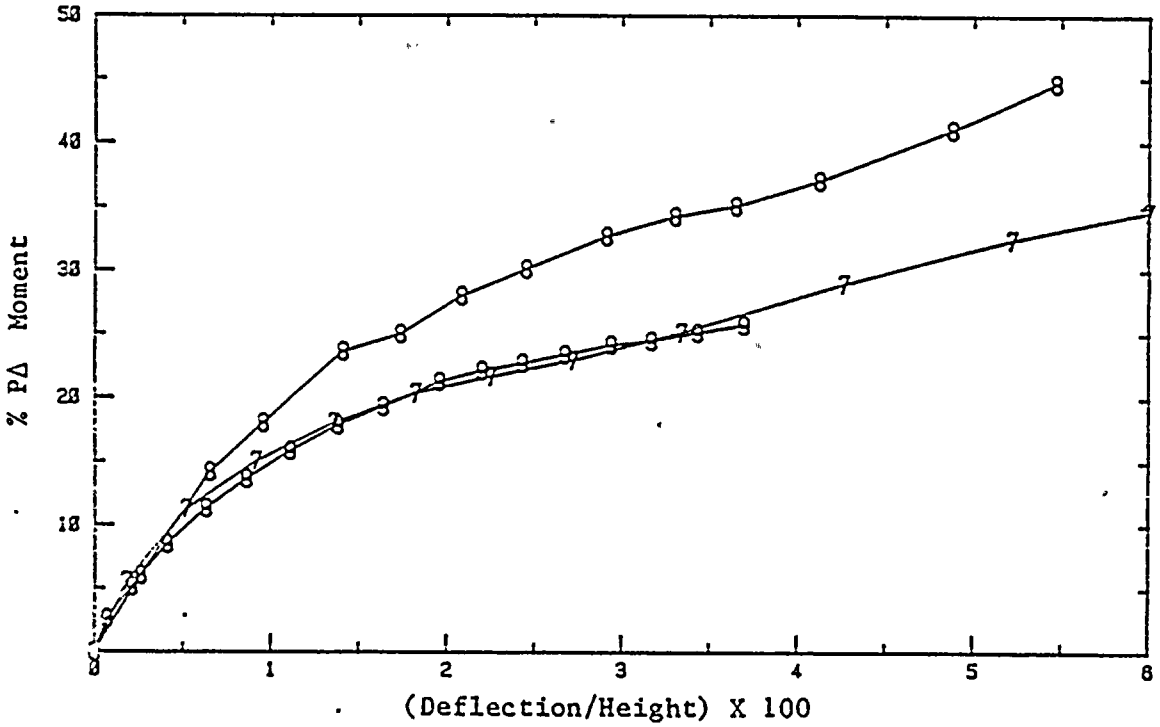


Fig. 6-13 % PΔ Moment, 6" Concrete Block

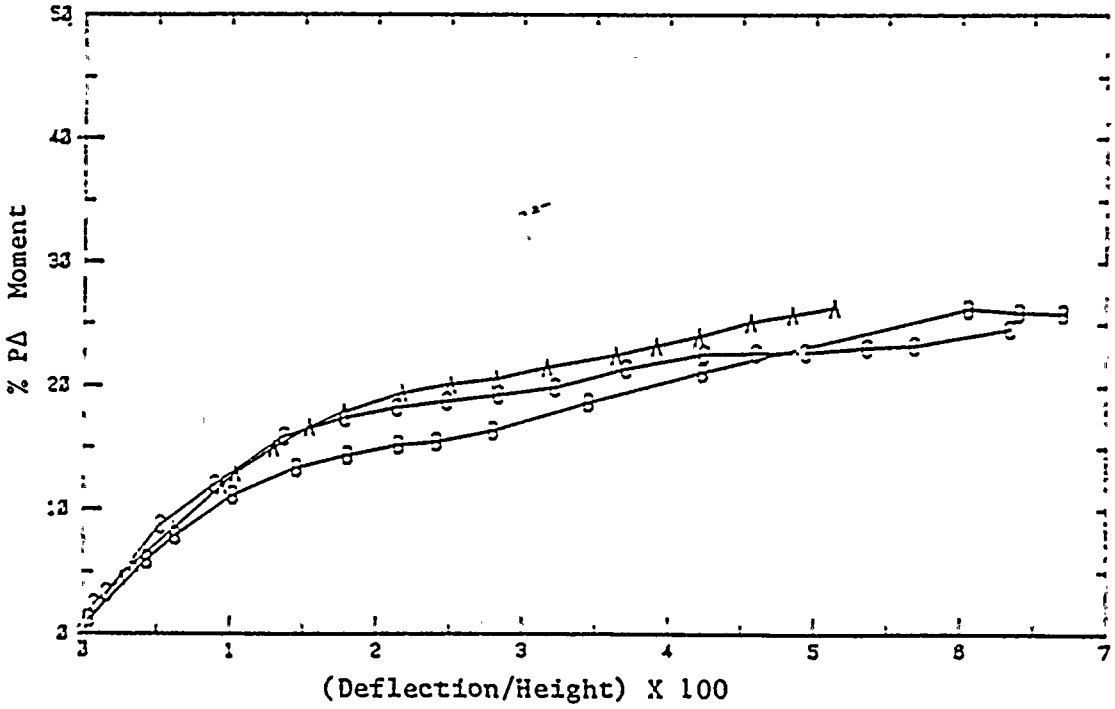


Fig. 6-14 % PΔ Moment, 5.5" Hollow Brick

This significant result shows that, in the normal working ranges of design, the PΔ moment is a minority contributor to the total moment.

6.7 AXIAL FORCE-MOMENT INTERACTION DIAGRAMS

The axial force-moment interaction diagram serves as an indicator of the strength of the cross sections. When the actual material properties are used in the Whitney stress block procedure, an accurate representation of the strength is obtained. Interaction diagrams of several configurations are shown in Figure 6-15. The axial load, P_o, strength of the 5.5" clay brick panel is larger than thicker concrete and concrete block panels because of the high f'_m value for the brick.

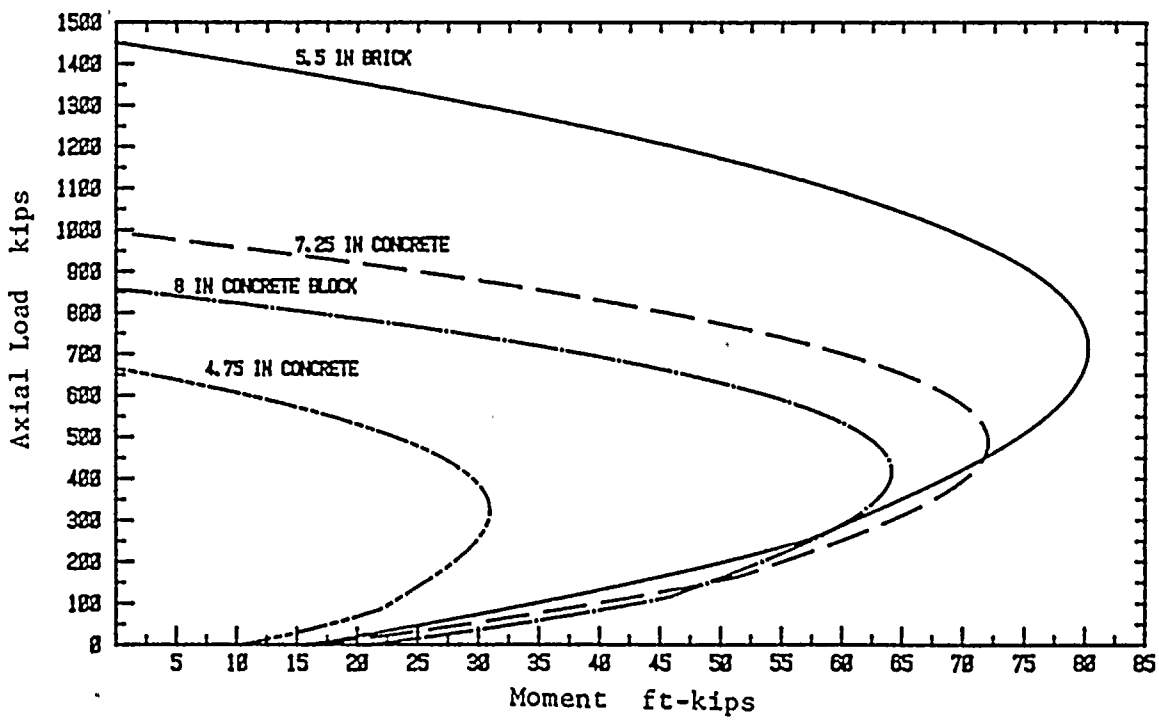


Fig. 6-15 Interaction Diagrams of Four Configurations.

In the present tests, our interest in the interaction diagrams lies in the low axial load range. A representation of the interaction diagrams for the low axial loads is given in Figure 6-16. In this range the moment strength increases slightly with axial load. The moment strength is primarily dependent on the amount and depth of steel in the cross section.

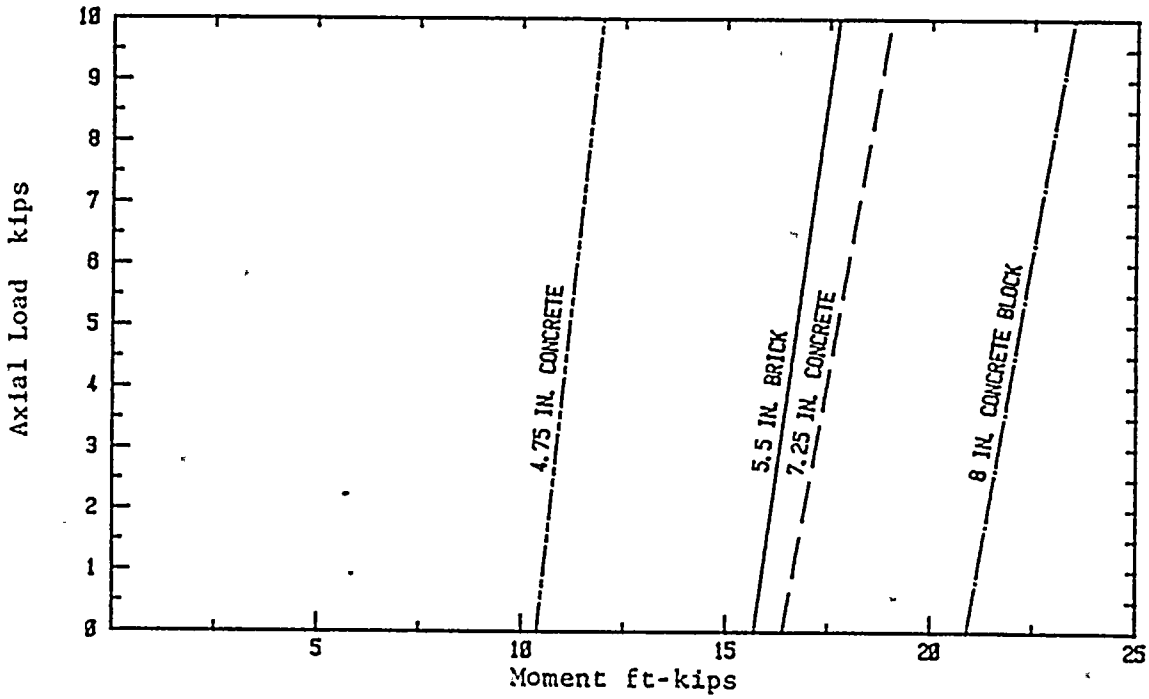


Fig. 6-16 Interaction Diagrams at Low Levels of Axial Load.

The resisting moments and axial forces in two concrete and two block panels for various deflection to height ratios are plotted in Figures 6-17 and 6-18.

Figure 6-17 is a plot of axial load or force versus moment. The two almost vertical lines representing predicted strength of 4.75" and 9.5" thick concrete panels were found from the interaction diagram calculation procedure. Actual f'_c = 4 ksi and

f_y = 70 ksi values were used in the calculation. The points clustered around each curve are the measured moments and axial forces in the panel for various deflection to height ratios (Δ/h). For the thin 4.75" (h/t = 60) panel the measured moment corresponding to Δ/h = 0.005 is approximately 60% of the predicted yield moment. In the case of a thicker 9.5" (h/t = 30) panel the measured moment corresponding to Δ/h = 0.005 is 95% of the predicted yield moment. This result shows that for thin panels, deflection constraints will control the design while for thick panels, strength will be the limiting constraint.

The measured moment exceeds the predicted moment for $\Delta/h > 0.02$ in both panels (Fig. 6-17). This occurs because of strain hardening of the reinforcement.

A similar plot of measured moments for various Δ/h values is plotted on an interaction diagram representing block panels in Figure 6-18. Here the measured moments do not exceed the predicted yield moment in the 6" block panel. A similar result is found for the 10" panel except when Δ/h = 0.03. The block panels are more flexible than the concrete panels and therefore reach larger deflections before yield occurs. Deflection would be an important consideration in design of both 6" and 10" walls.

These figures show an expanded scale of the portion of the interaction curve located near the origin. The points plotted on the large scale curve are the yield point moments determined by test. The variation between the test moments and the predictive interaction curves, for the most part, have been attributed to the mislocation of the reinforcement. Good agreement was obtaind for concrete, concrete masonry, and the brick panels.

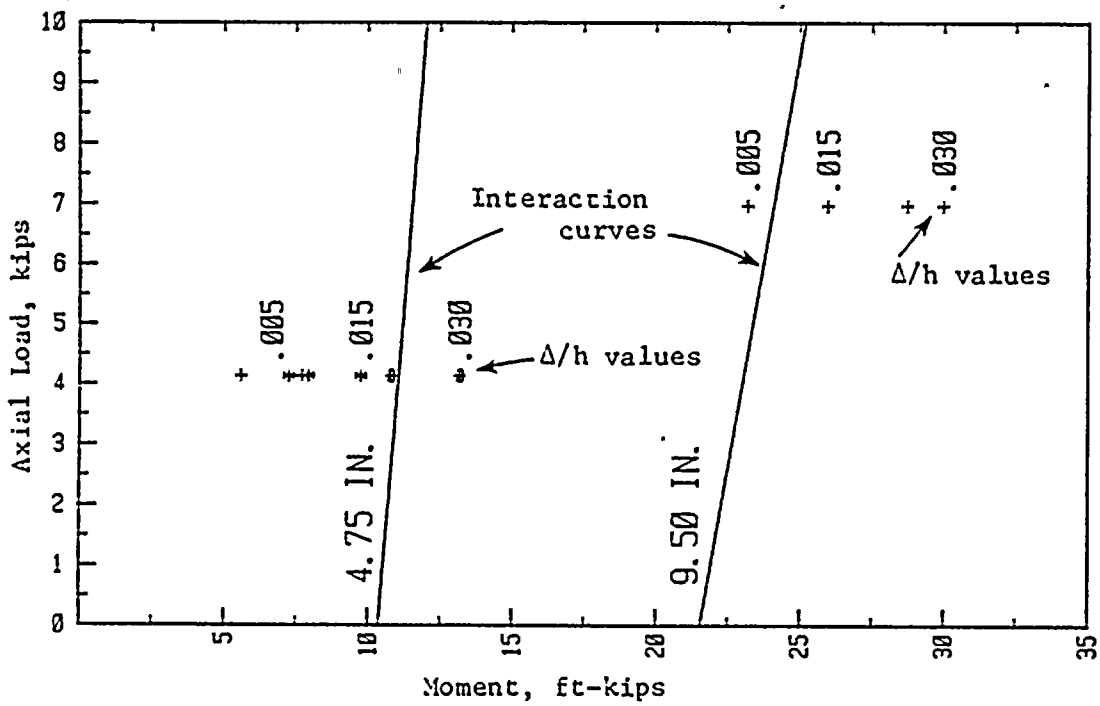


Fig. 6-17 Measured Moments for Various Δ/h Levels Compared with Interaction Diagrams for Concrete Panels.

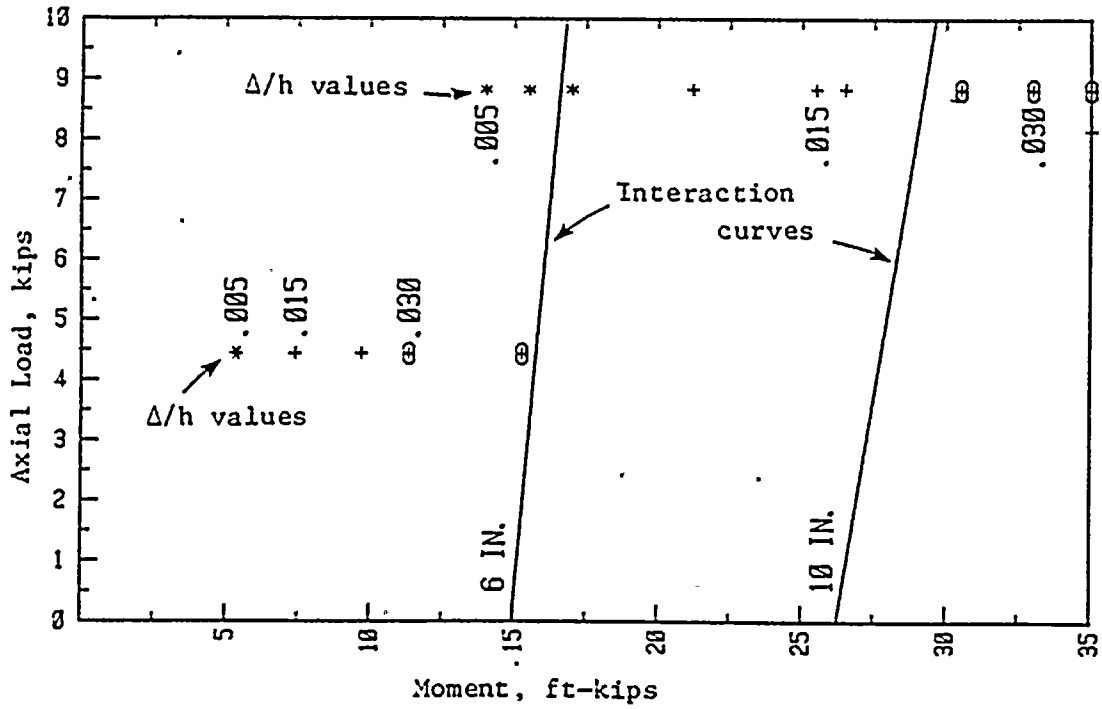


Fig. 6-18 Measured Moments for Various Δ/h Levels Compared with Interaction Diagrams for Concrete Block.

6.8 PREDICTIONS OF DEFLECTIONS USING MOMENT/CURVATURE RELATIONSHIPS

The fundamental theory for flexural deflection is based on a relationship of moment and curvature. Accordingly, a mathematical model was constructed for computer analysis to compare results with the experimental data.

The method must first develop the interaction curve based on the section properties. The interaction diagram uses the Whitney stress block concept, and the depth to the neutral axis is the basis for computing the curvatures corresponding to the load and moment points. From the interaction diagram, a family of Moment/Curvature relationships for each point on the wall is generated since the vertical load varies with height of the wall.

After obtaining the Moment/Curvature relationships, a moment based on $wl^2/8 + Pe$ is applied and a set of curvatures is obtained, which are then integrated to obtain an initial set of deflections that are used to calculate the $P\Delta$ moments, which are added to the original moments, and a new set of curvatures are generated and integrated. This continues until the solution converges.

Application of this procedure to the test walls produced load/deflection curves that had excellent correlation with the test results. The results for the 9.5" concrete panel are shown in Figure 6-19.

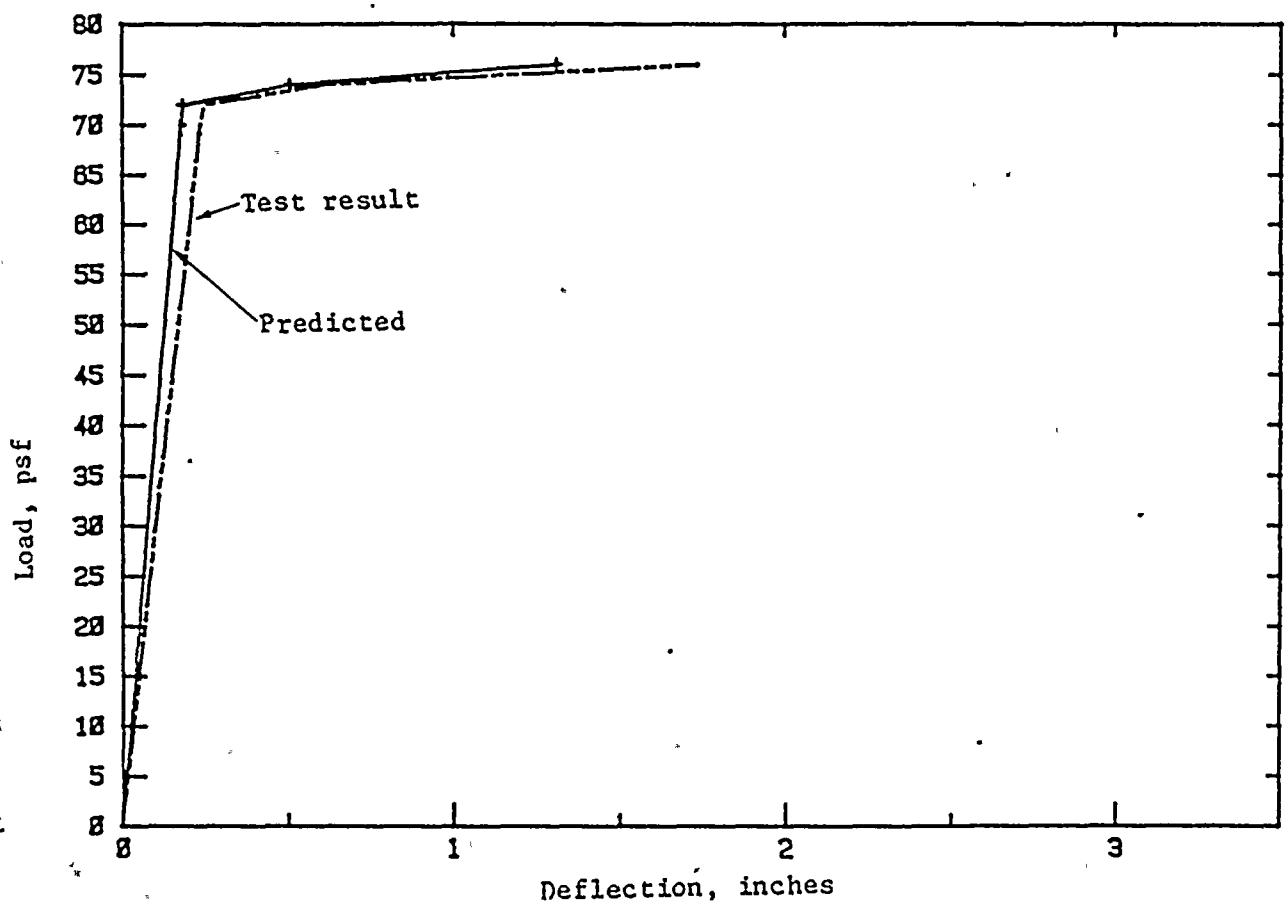


Fig. 6-19 Predicted and Test Curves for 9.5" Concrete Panel.

SECTION 7

DEVELOPMENT OF DESIGN METHODS

7.1 INTRODUCTION

The experimental results demonstrated that wall response to horizontal loads resembled the behavior of shallow reinforced concrete beams subjected to uniform load (Ref. 13). After cracking, the walls were flexible, and after yield their load-deflection curves were flat. Shear failure or bond slip did not occur, so that even when deflections were large, a reduction in panel resistance did not occur.

Vertical loads used in the experiments were chosen to represent typical tributary design loads used for buildings in California. Panelized wood roof systems used in single-story construction generate loads that seldom exceed 300 lb/ft. The gravitational weight per unit length of tilt-up and masonry panels is much greater than the tributary roof load per unit length. The vertical load at the foundation level is usually four to eight times greater than the roof load. Therefore, any secondary moment (Ref. 25) in the panel due to vertical load will be primarily caused by the weight of the panel. The roof load will contribute only 12 to 25% to the secondary moment.

During the experiment, the lateral pressure load imparted to the panel by the air bag was increased until deflections reached two to three times the panel thickness. Up to the yield level, the relationship between lateral load and midheight lateral deflection resembled a bilinear form, which means the response can be represented by two straight lines (Ref. 28). The characteristics of the bilinear relationship are shown in Figure 7-1. Up to a load that induces cracking, the response is described by a steep, straight line. During a further load increase, a cracking pattern is developed and the load deflection relation is

curved. After the cracking pattern has stabilized, further increases in the load induce a low-slope, straight-line response.

Mathematical representation of load-deflection relations, such as those depicted in Figure 7-1, go back to the 1940's. Accurate predictions can be obtained using methods similar to those given in Reference 41. Even the curved portion of the load-deflection curve can be predicted easily (see Sec. 7-6).

The testing and analysis work of the Slender Walls Test Program focused heavily on design considerations. The analysis methods developed herein are intended for use in design. Load-deflection relations are needed because both strength and serviceability are considered in slender wall design.

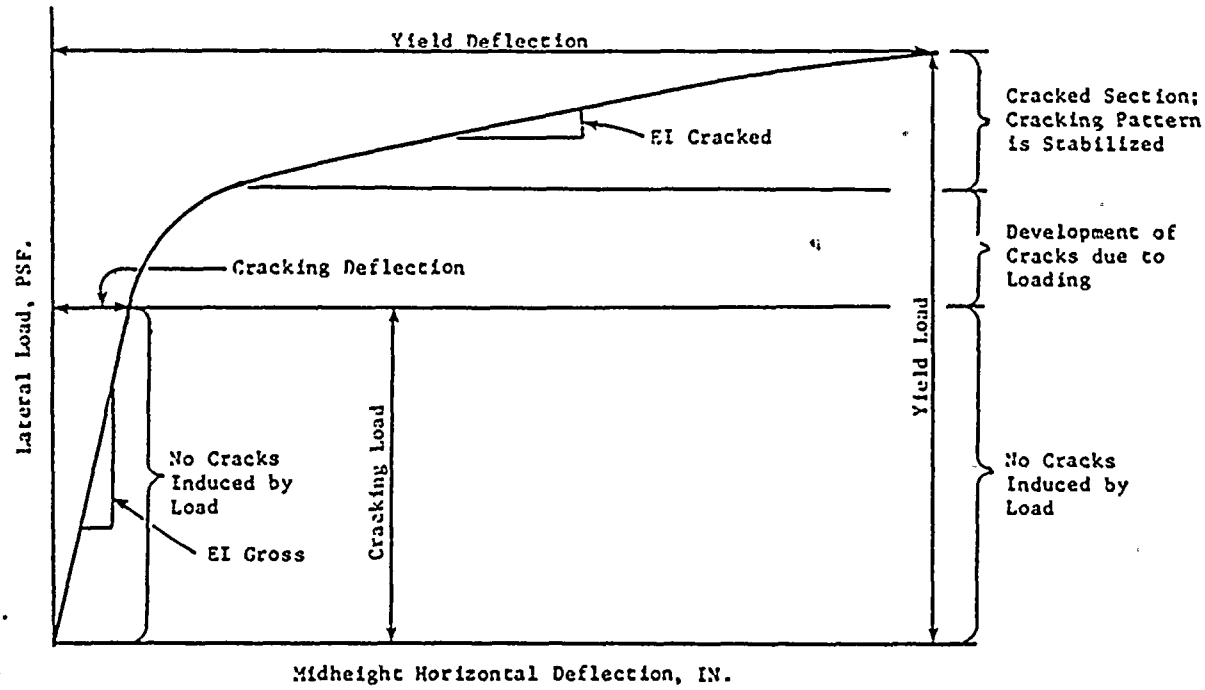


Fig. 7-1 Characteristics of Bilinear Load-Deflection Relation

7.2 DESIGN VARIABLES

The principal design variables are wall height, thickness, and amount of vertical reinforcement. The height dimension is set to satisfy functional and architectural needs. Under the current code procedures, wall thickness is then chosen to satisfy the height-to-thickness ratio (h/t) requirements (Refs. 1, 14, 38). Strength requirements are considered when selecting amounts of reinforcement. Frequently the minimum percentages are used in order to satisfy temperature and shrinkage requirements.

The measured flexibility of the wall specimens showed that some of the thin walls with high h/t values resisted the factored design loads without yielding, although the deflections were large (Fig. 7-2).

7.3 STRENGTH AND DEFLECTION CHARACTERISTICS

The 6" concrete block panels that were tested had an h/t = 51.2. The reinforcement ratio based on the gross section, ρ_g, was 0.0037. These panels exhibited significant horizontal deflections under service loads and yet carried loads in excess of the factored design loads without yielding. A load-deflection curve representing the average of the three panel responses, shown in Figure 7-3, will be used to present strength and stiffness characteristics.

The midheight deflection of the panel was 2.5 in. when it was subjected to a lateral service load equal to 17.4 psf, or 30% of the wall weight. When this deflection is converted to a height/horizontal deflection ratio, $h/\Delta = 24 \times 12/2.5 = 115.2$. The height/horizontal deflection ratio is similar to the span/vertical deflection ratio used as a design criterion for limiting vertical deflections in beams (Refs. 1, 14, 38). For horizontal beams a ratio of 115.2 indicates a large deflection; however, a ratio of 115.2 is an acceptable value for the walls of a one-story building.

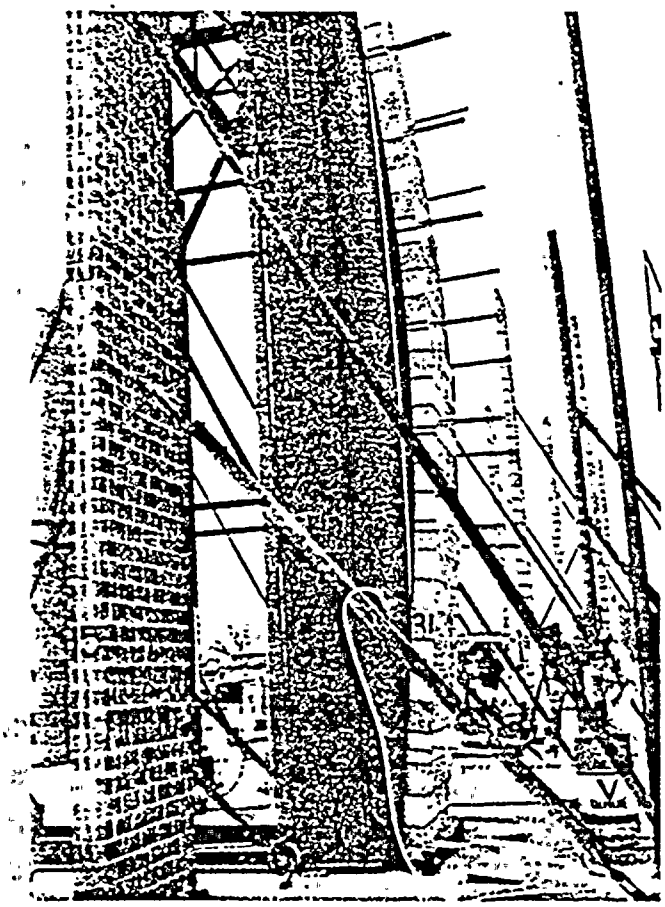


Fig. 7-2 Flexible 6 in. Concrete Masonry Panel

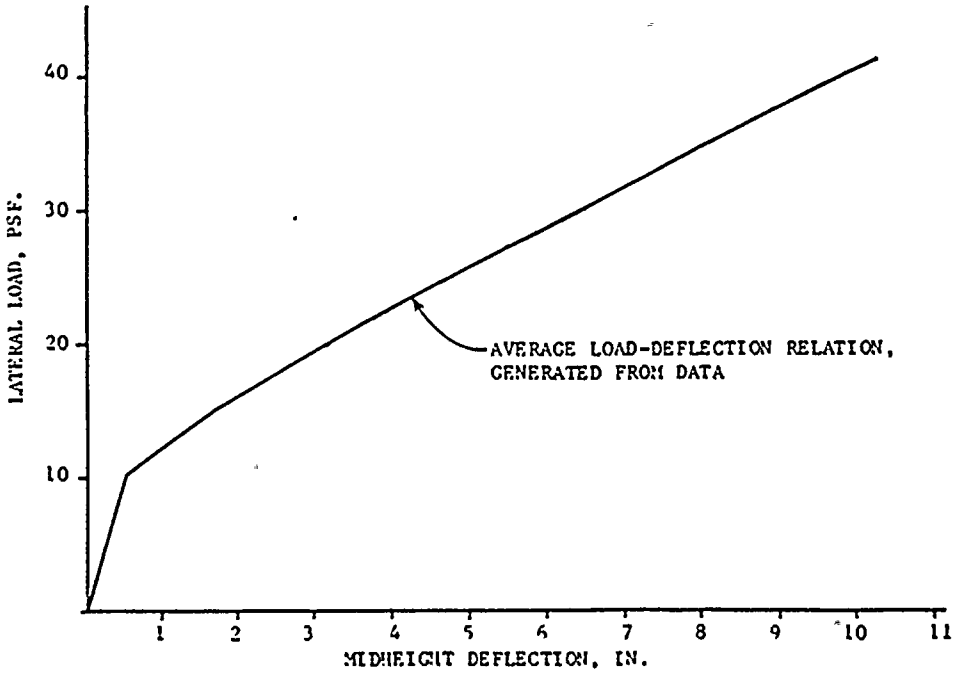


Fig. 7-3 Load-Deflection, 6" Concrete Block Masonry.

The measured load-deflection curve (Fig. 7-3) reached a yield level when the lateral load was 41.2 psf. This lateral load is equivalent to 71% of the wall weight. The ratio of measured yield load/service load, R_m, is 2.36, which is a large value. In ultimate strength design (Refs. 1, 14, 38) the ideal design yield load/service load ratio, R_n, is given by

$$R_n = \frac{U}{\phi E} = \frac{1.4}{0.8} = 1.75 \qquad (7\text{-}1)$$

where U = ultimate load, $\phi = 0.8$ (see Sec. 7.5), and E = earthquake service load. Since the measured ratio, R_m, exceeds the ideal design ratio, R_n (i.e., $R_m > R_n$), the panel can carry the factored loads without yielding.

For a panel design to be considered adequate, the panel must meet both deflection standards and strength standards. The 6" concrete block panel with $\rho_g = 0.0037$ carried the factored design loads with margins to spare; its height/deflection (h/Δ) ratio was $h/\Delta = 115.2$ when subjected to a service load. Therefore, the finding of the research project is that this panel barely meets deflection design standards when a reasonable h/Δ limit ($h/\Delta = 100$; see section 7.4.2) is used. The panel easily satisfies the requirement for strength under factored loads.

7.4 STRENGTH AND DEFLECTION DESIGN CRITERIA

The design criteria selected by the Committee are deflection under service load and yield strength under factored load. A successful design must satisfy both criteria.

7.4.1 STRENGTH CRITERIA

Loads and resistance are considered when strength is evaluated.

Loads

1. Factored loads are based on:

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

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

$$U = 0.9D + 1.43E \quad \text{or}$$

$$U = 0.9D + 1.3W \qquad (7\text{-}2)$$

whichever is the most severe. These load factors are used for design of elements, systems, and connections of a building subjected to either seismic or wind forces (Sec. 2609, UBC, 1979, 1982 eds.). Essentially they are used for out-of-plane forces in the design of tall, slender walls. Other load factors, where $U = 1.4\ (D + L + E)$, are for shear walls and frames that, at the discretion of the designer, are used to resist forces parallel to them (Sec. 2627(d), UBC, 1979, 1982 eds.).

2. Lateral and vertical loads are used in computing the maximum design moment, M_u, which, for practical purposes, occurs at the midheight of the panel.

3. Secondary moments induced by deflections at the midheight of the panel are included in the maximum design moment, M_u.

4. Axial forces at midheight include effects from roof load and panel weight.

Resistance or Capacity of Panel Cross Sections

1. Ultimate strength design concepts are used when evaluating the moment capacity of cross sections. A Whitney stress block idealization is assumed for the compressive stress distribution in the concrete and the masonry (Ref. 18). This rectangular stress block is a simple generalized form of the ultimate stress strain curve when maximum strain of 0.003 is achieved.

The design compressive strength, f'_c, is used for concrete calculations. Similarly, f'_m for masonry can be an assumed value or can be established from prism tests. Yield stresses used in reinforcement calculations correspond with the grade of the reinforcement.

The limiting compressive strain in the concrete masonry units and brick masonry units is an important consideration in ultimate strength design. In the present study, a limiting compressive strain of 0.003 was used for both concrete and masonry materials.

The balanced reinforcement percentages, ρ_b and ρ_{gb}, found using the tested values for f'_c, f'_m, and f_y are listed in Table 7-1. Also included is the actual reinforcement percentage, ρ_g, that was used in the panels. The actual reinforcement percentage/balanced reinforcement percentage ratio ρ_g/ρ_{gb} is also given.

TABLE 7-1. BALANCED REINFORCEMENT PERCENTAGES

Tested Values, f_y = 70,000 psi

Type	f'_m or f'_c, psi	% Balanced ρ_b*	% Gross Balanced ρ_{gb}†	% Actual ρ_g†	Ratio ρ_g/ρ_{gb}
6" CMU	3185	1.82	0.91	0.37	0.41
8" CMU	2595	1.48	0.74	0.27	0.36
10" CMU	2460	1.41	0.71	0.22	0.31
5-1/2" Hollow Brick	6243	3.57	1.79	0.38	0.21
9-5/8" 2-Wythe Brick	3060	1.75	0.88	0.22	0.25
4-3/4" Concrete	4000	2.29	1.15	0.35	0.31
5-3/4" Concrete	4000	2.29	1.15	0.29	0.25
7-1/4" Concrete	4000	2.29	1.15	0.23	0.20
9-1/2" Concrete	4000	2.29	1.15	0.18	0.15

Design Values, f_y = 60,000 psi

Type	f'_m or f'_c, psi	Balanced ρ_b*	Balanced ρ_{gb}†	Maximum Design ρ_g†	Maximum ρ_g/ρ_{gb}
6" CMU	1500	1.07	0.54	0.32††	0.59
8" CMU	1500	1.07	0.54	0.32	0.59
10" CMU	1500	1.07	0.54	0.32	0.59
5-1/2" Hollow Brick	2500	1.78	0.89	0.40	0.45
5-1/2" Hollow Brick	5000	3.56	1.78	0.40	0.22
9-5/8" 2-Wythe Brick	1800	1.28	0.64	0.40	0.63
4-3/4" Concrete	3000	2.14	1.07	0.53§	0.50§
5-3/4" Concrte	3000	2.14	1.07	0.53	0.50
7-1/4" Concrete	3000	2.14	1.07	0.53	0.50
9-1/2" Concrete	3000	2.14	1.07	0.53	0.50

*Percentage based on d distance, i.e., d = t/2, based on Equation 7-3.

†Percentage based on t (Eq. 7-4).

††ρ_g < 0.32% (Eqs. 7-3, 7-4, and 7-5).

§0.50 ρ_{gb} (Ref. 1, Appendix A).

The present code limitations on balanced steel for concrete flexural members is considered to be satisfactory for the prevention of brittle failure.

For the 6" concrete masonry panel, the block prisms had f'_m (tested) = 3185 psi, which is a high value for block with a specified masonry assembly design strength f'_m (design) = 1500 psi. A more realistic tested compressive strength of the block is 2200 psi for Grade N units with a specified masonry assembly design strength f'_m (design) = 1500 psi. The overstrength of concrete masonry units is explained by reviewing the ASTM requirements for block properties. The compressive strength of Grade N units must be at least 1000 psi on the gross area for the average of three units. To meet the ASTM C90 requirements of 1000 psi compressive strength based on the gross area, it is necessary to manufacture block with an average compressive strength of 2200 psi based on the net area. The grout used in masonry assemblies with f'_m (design) = 1500 psi must have a 28-day minimum strength of 2000 psi. The average compressive strength of grout will exceed 2200 psi. Therefore, the block will have at least the value f'_m (tested) = 2200 psi.

References 14 and 38 state that an f'_m = 1500 psi may be assumed without tests when the requirements of ASTM C90 for concrete block Grade N and ASTM C62 or C216 for clay brick Grade MW with 2000 psi grout and type M or S mortar are specified.

A balanced reinforcement percentage, ρ_b, for f'_m (design) = 1500 psi for concrete masonry units based on the more realistic strength of the block and f'_m (tested) = 2200 psi and f_y (tested) = 70,000 psi is

$$\rho_b = \frac{85\ \beta\ f'_m}{f_y}\left(\frac{87,000}{87,000 + f_y}\right) = 1.26\% \qquad (7\text{-}3)$$

where $\beta = 0.85$. With $d = t/2$, the balanced percentage of tension steel based on the gross cross-sectional area, ρ_{gb}, is

$$\rho_{gb} = \frac{1}{2} \rho_b = 0.63\% \tag{7-4}$$

It was decided to limit the design ρ_g to 1/2 of the ρ_{gb} given in Equation 7-4; therefore, for concrete masonry

$$\rho_g \leq \frac{1}{2} \rho_{gb} = \begin{cases} 0.32\% \text{ for } f_y = 60{,}000 \text{ psi} \\ 0.50\% \text{ for } f_y = 40{,}000 \text{ psi} \end{cases} \tag{7-5}$$

A ρ_g (design) $\leq$ 0.32% is a conservative limit value for concrete masonry units with f'_m (design) = 1500 psi and f_y (design) = 60,000 psi. In the tested 6" concrete masonry block panels, the ρ_g (tested) = 0.37% but it was necessary to use ρ_g (design) $\leq$ 0.32% because of the high f'_m (tested) = 3185 psi found from the block prism tests.

A listing of design ρ_g's and ratios for f_y = 60,000 psi is given in Table 7-1. The ρ_g values for concrete block, brick, and concrete walls are listed. The ρ_g (design) values for hollow brick units and two-wythe brick units is limited by ρ_g (design) $\leq$ 0.40% for f'_m (design) = 1500 psi and f_y (design) = 60,000 psi. This is conservative since the strength of brick units falls in the 5000 to 6000 psi range.

Flexural forces acting on over-reinforced cross sections may cause brittle failure. The maximum design ρ_g and maximum ρ_g/ρ_{gb} values were set at low levels so that brittle failures can be avoided. More testing is needed to establish the upper limit of the ρ_g/ρ_{gb} values.

2. Axial forces are considered when evaluating the moment strength, M_n, at the midheight cross section. A capacity reduction factor, ϕ, is used in the relation

$$M_u \leq \phi M_n \tag{7-6}$$

which required that the maximum design moment must be less than or equal to the moment capacity reduced by the ϕ factor.

7.4.2 DEFLECTION CRITERIA

Loads and stiffness are considered when deflection is evaluated. An h/Δ limit is selected, which keeps the midheight deflections within acceptable limits. The purpose of the deflection computation is to prevent designs of overly flexible panels and to assure reasonable straightness after a service level loading.

Lateral Loading

1. Working or service loads are used when calculating the midheight deflection.

2. Lateral and vertical loads are used in computing the maximum horizontal deflection, which for practical purposes occurs at the midheight of the panel.

3. Secondary moments induced by deflections at the midheight of the panel are represented in the deflection calculation.

4. ϕ factors are not used in the deflection calculation.

Stiffness of the Panel

1. The load-deflection relation for the panel is assumed to obey a bilinear law (Fig. 7-1).

2. The slopes of the straight line parts of the load-deflection curve are as follows: (a) up to cracking load, a gross section I is used to compute deflection from the load; (b) additional deflection beyond the cracking load is computed with the cracked I. The ratio of gross to cracked moment of inertia varies between 10 and 25. I values for panels used in typical designs are given in Table 7-2.

3. The load at which cracking occurs is observed from the experimental data. The modulus of rupture and cracking moment can be computed from the cracking load.

4. The midheight deflection, Δ, is computed for a panel with simple support at the top and bottom. In this procedure, called the PΔ Method, the Δ is found in the following:

$$\Delta = \begin{cases} \dfrac{5Mh^2}{48EI_{gross}} & \text{for } M < M_{cr} \\[2ex] \dfrac{5\,M_{cr}h^2}{48EI_{gross}} + \dfrac{5(M - M_{cr})h^2}{48EI_{cracked}} & \text{for } M_{cr} < M < M_{yield} \end{cases} \qquad (7\text{-}7)$$

where

h = Height of the wall

M = Service moment at the midheight of the panel, including P-Δ effects (see following section for midheight moments)

E = Code-prescribed modulus of elasticity of the concrete (Ref. 1) or masonry (Ref. 39)

I_{gross}, $I_{cracked}$ = Gross, cracked moment of inertia of the wall cross section

M_{cr}, M_n = Cracking, strength moment of the concrete or masonry

5. The deflection at the top of the panel is assumed to be zero in Equation 7-6. The effect of diaphragm deflections on midheight motions of the panel is not considered in the deflection calculation. This is not in conflict with the purpose of the deflection calculation, which is to eliminate overly flexible panels.

6. The deflection Equation 7-7 is based on simple support conditions. Deflection calculations for other top and bottom support conditions must be performed using more basic procedures such as moment-area or virtual work methods. A "dock height" building is a case in which moment area or virtual work methods are needed.

Selection of h/Δ Limit

The midheight deflection found from Equation 7-6 is limited so that a serviceable panel is designed. The deflection limit to be used in calculations is given by the relation

$$h/\Delta = 100 \qquad (7\text{-}8)$$

The maximum deflections allowed are directly proportional to the height of the wall. A tabulation of maximum deflections is given in Table 7-3. These are large deflections, but they are manageable in industrial buildings. Higher and lower values for the h/Δ limit were considered. A higher value would eliminate tall, slender walls with high ρ_g values. A lower value would permit unserviceable panels with midheight deflections that are very large.

TABLE 7-2. GROSS AND CRACKED MOMENTS OF INERTIA (I in.^{4}) OF TYPICAL PANELS PER FOOT OF WIDTH

Type	Gross I	Cracked I	$\frac{\text{Gross I}}{\text{Cracked I}}$	Cracked I	$\frac{\text{Gross I}}{\text{Cracked I}}$
		$\rho_g = 0.20\%$		$\rho_g = 0.40\%$	
4-3/4" Concrete	107.2	4.2	25.6	7.3	14.7
5-3/4" Concrete	190.2	7.4	25.6	12.9	14.7
7-1/4" Concrete	381.1	14.9	25.6	25.9	14.7
9-1/2" Concrete	857.4	33.5	25.6	58.2	14.7

Note: f'_c = 3000 psi for concrete

		$\rho_g = 0.20\%$		$\rho_g = 0.32\%$	
6" CMU	177.1	12.5	14.3	17.3	10.3
8" CMU	443.3	31.0	14.3	43.0	10.3
10" CMU	891.7	62.4	14.3	102.6	10.3

Note: f'_m = 1500 psi for CMU

		$\rho_g = 0.20\%$		$\rho_g = 0.40\%$	
5-1/2" Hollow brick with f'_m = 2500 psi	166.4	7.8	21.3	13.4	12.5
5-1/2" Hollow brick with f'_m = 5000 psi	166.4	4.4	38.1	7.8	21.3
9-5/8" Two-Wythe Brick with f'_m = 1800 psi	884.7	53.8	16.4	89.9	9.8

TABLE 7-3: MAXIMUM ALLOWABLE DEFLECTIONS FOR WALLS OF VARIOUS HEIGHTS BASED ON $h/\Delta = 100$

Wall Height, ft	Midheight Deflection, in.
15	1.8
20	2.4
25	3.0
30	3.6
35	4.2

7.4.3 DETERMINATION OF MOMENTS AT THE MIDHEIGHT OF THE PANEL

The moment at the midheight of the wall can be found from statics. Consider the panel support and free body diagrams shown in Figure 7-4. The horizontal force at the roof line, H_T, is found by summing moments about B. The result is

$$H_T = \frac{wh}{2} + \frac{2P_p\Delta}{3h} - \frac{P_o e}{h} \qquad (7\text{-}9)$$

where

H_T = Horizontal force at the roof line

w = Lateral load acting on the panel

P_p = Weight of the panel

P_o = Load at the roof line

e = Eccentricity of the roof load

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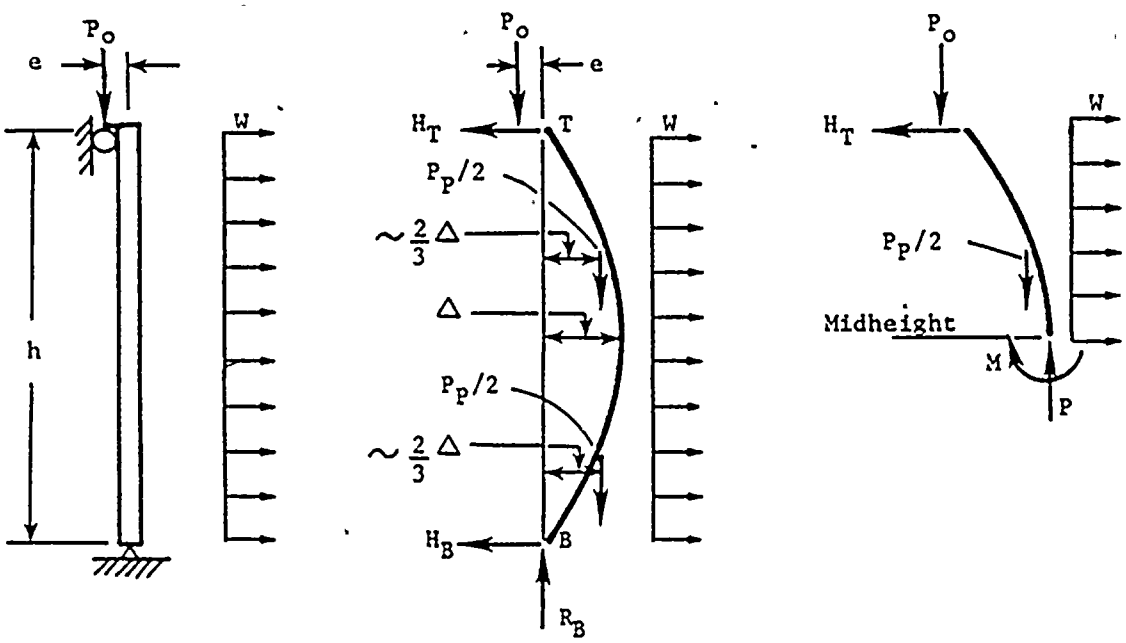


Fig. 7-4 Panel Support and Free Body Diagrams.

By summing moments about the panel midheight, the relation for midheight moment, M, is obtained:

$$M = \frac{wh^2}{8} + \frac{P_p\Delta}{2} + P_o\Delta + \frac{P_o e}{2} \tag{7-10}$$

The moment found from Equation 7-10 depends on the midheight deflection, Δ. The deflection Δ is found from Equation 7-7, where, it is noted, M is required as input there; this is an iterative process. It has been shown that $Pe\Delta/2$ and $P\Delta$ were not the dominant terms of Equation 7-10 (Ref. 32). Therefore, this iterative procedure using approximate values of M and Δ will converge rapidly. The approximation for the midheight moment in the i + 1 iteration cycle is found after Equation 7-10 is modified to

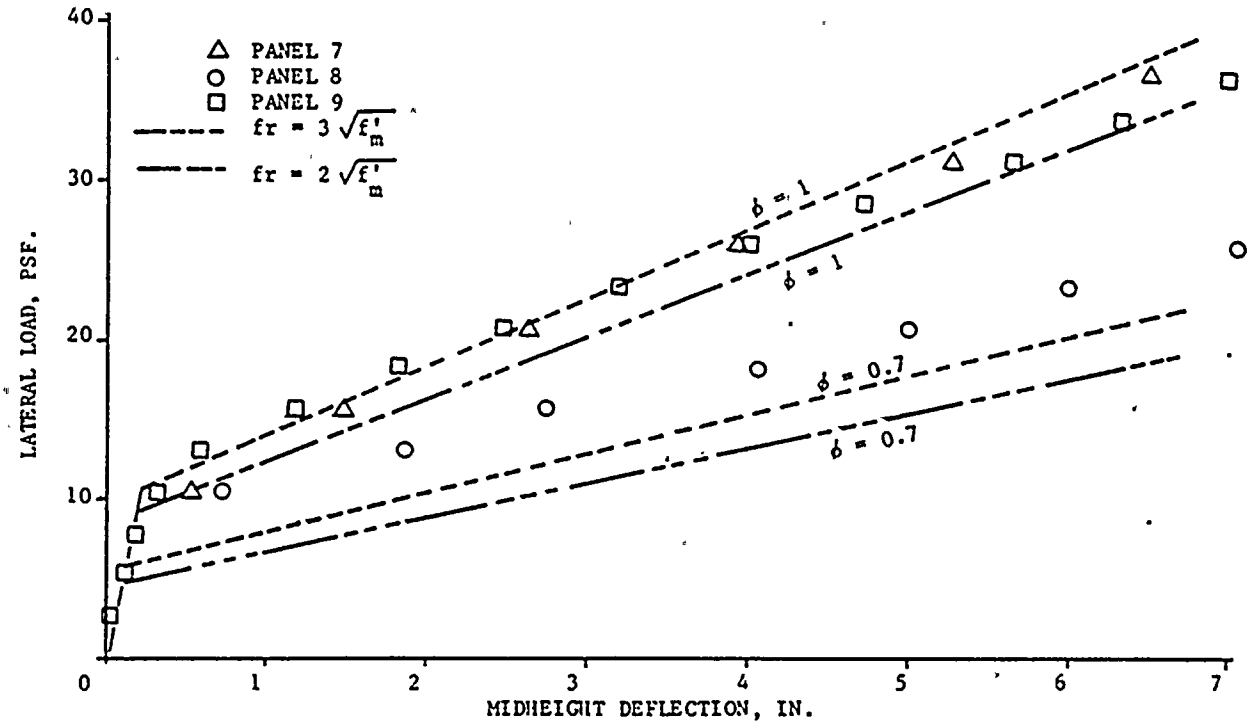


Fig. 7-5 Selection of ϕ Factor and Cracking Moment: 6" Concrete Block Masonry.

mine M_{cr} and ϕ values that put the ϕw vs. Δ relations on the safe side of the experimental data. The use of the ϕ factor for putting prediction relations on the safe side of experimental data is a widely accepted approach (Ref. 15).

For simplification of design, the number of different M_{cr} and ϕ values was kept to a minimum. Different M_{cr} and ϕ values were used for each type of material, i.e., concrete, concrete block masonry, hollow brick masonry, and wythe brick masonry. Each type had distinct force-deflection properties, and thus different M_{cr} and ϕ values were needed for accurate prediction of behavior. The M_{cr} and ϕ factors that were selected for the different materials are as listed in Table 7-4. For the ϕw vs. Δ relations that incorporate these M_{cr} and ϕ values, plots for three panel types are given in Figures 7-6

through 7-8. The M_{cr} and ϕ values were chosen by inspecting the ϕw vs. Δ plots for each type of material (Figs. 7-6 through 7-8). The values were selected so the predicted yield point fell below or near the lower boundary of the data for all thicknesses of panels of the same type.

Values of M_{cr} and ϕ for masonry panels subjected to non-continuous inspection listed in Table 7-4 were selected so that the wall heights for various thicknesses and reinforcement percentages were essentially the same as allowed under present codes when the arbitrary h/t limit is not imposed.

TABLE 7-4. ϕ AND M_{cr}^* FACTORS FOR DIFFERENT MATERIAL TYPES USED FOR WALLS

Type	Special Inspection ϕ Factor	Noncontinuous Inspection 0.8ϕ Factor	f_r^*
Concrete	0.9	0.72	$5\sqrt{f'_c}$
Concrete Masonry Units	0.8	0.64	$2.5\sqrt{f'_m}$
Hollow Brick Masonry	0.85	0.68	$2.5\sqrt{f'_m}$
Two-Wythe Brick Construction	0.75	0.60	$2.0\sqrt{f'_m}$

*$M_{cr} = Sf_r$

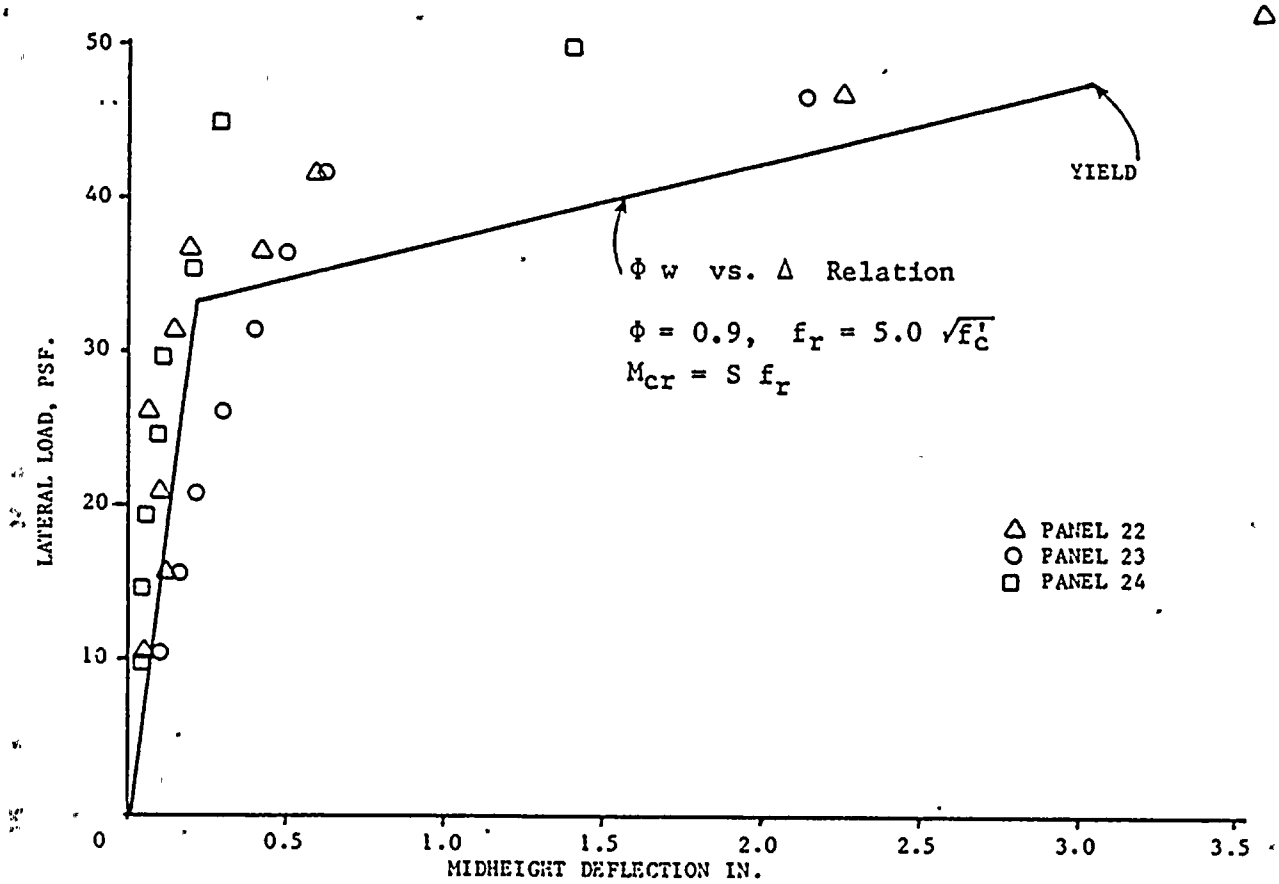


Fig. 7-6 M_{cr} and ϕ 7.25" Concrete Panel

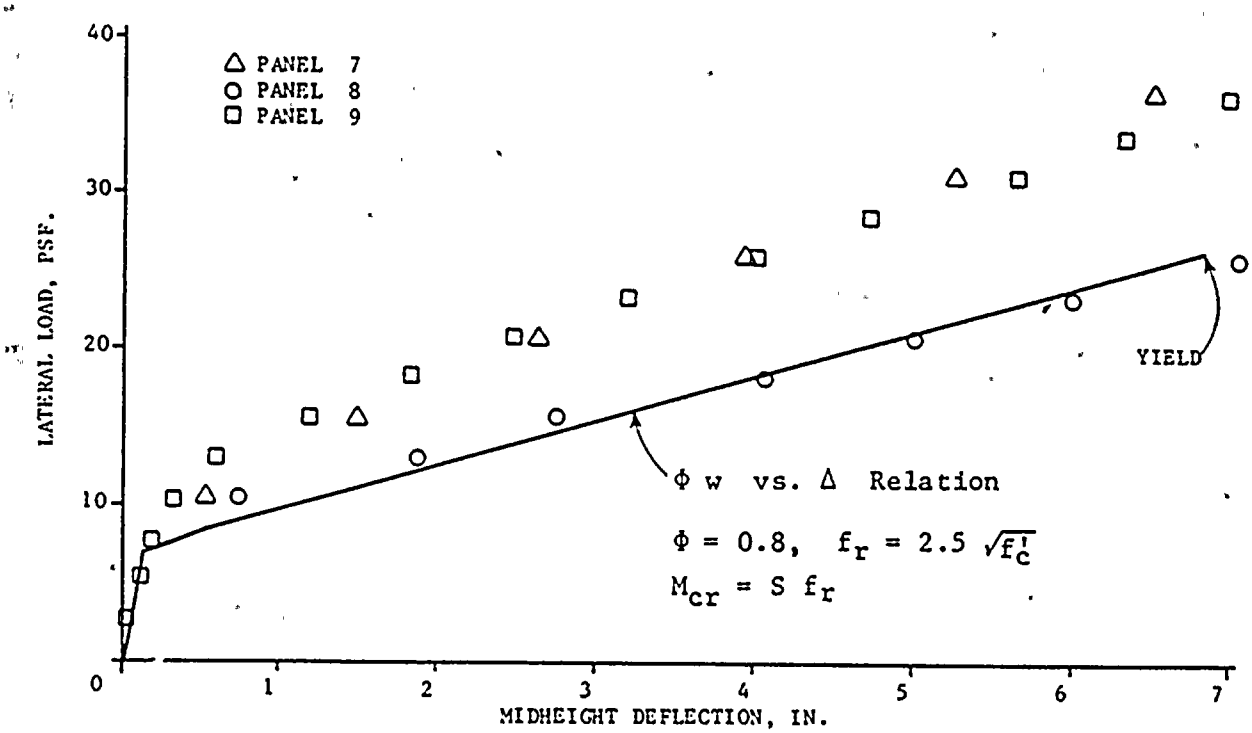


Fig. 7-7 M_{cr} and ϕ 6" Concrete Block Masonry

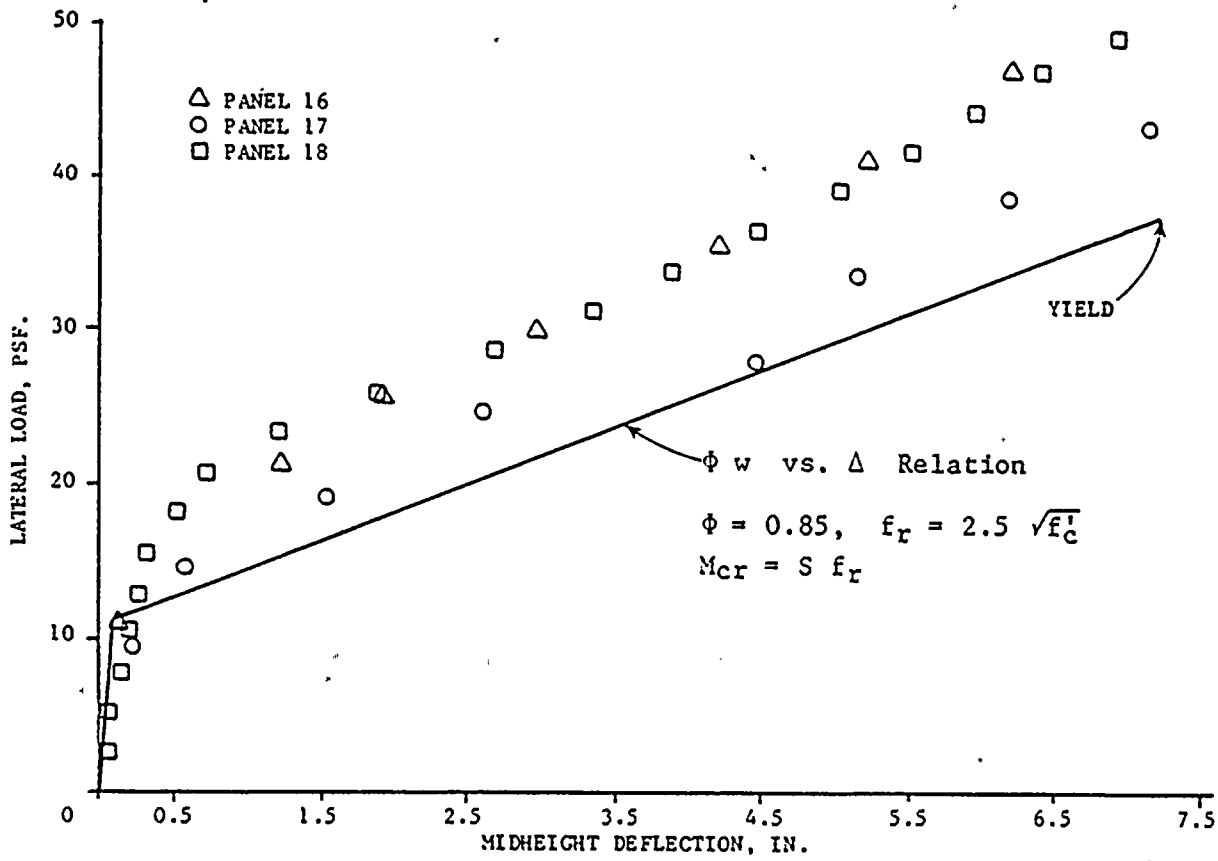


Fig. 7-8 ϕ Selection 5.5" Hollow Brick Masonry

SECTION 8

DESIGN EXAMPLES

8.1 INTRODUCTION

Two design methods for walls of buildings with small axial loads have been formulated, and are presented here in the form of seven example problems. Both methods incorporate modulus of rupture, ϕ factors, determination of Δ at service loads and at factored ultimate loads; these elements ensure that designs fall on the safe side of the experimental data. It is of primary importance that both serviceability and strength requirements are satisfied in each method.

The serviceability requirement concerns deflection under service load; it is a necessary condition that the midheight deflection of the wall does not exceed 1/100 of wall height.

The strength requirement is calculated using factored loads. The factored moment, M_u, includes both Pe and PΔ moments that are added to the primary moments due to lateral and vertical loads. ϕ factors are used to account for the variability of material performance, effects of tolerances, and reliability of the computation. The nominal moment, M_n, is found from ultimate strength theory. The strength requirement is $M_u \leqq \phi M_n$.

The proposed design methods are intended to replace the current code methods with their arbitrary h/t limitations (Refs. 14 and 38). The proposed methods agree closely with measured load-deflection behavior that occurred during the experimental testing program.

Examples 1 and 2 use the PΔ Design Method, an iterative approach to determine deflections for calculating magnitude of secondary moments. Examples 3 through 7 use the Yellow Book

Method to determine deflection at service load and at the yield of steel in order to determine the secondary moments. Note that the examples from the Yellow Book Method are given in the work-sheet format common to engineering practice.

8.2 DESIGN EXAMPLES USING PΔ DESIGN METHOD

Two design examples using the PΔ Design Method developed in this report are presented here in problem format.

EXAMPLE 1:

Given: A 20'-tall hollow brick masonry constructed with 5.50" units. The fully grouted wall is simply supported at the top and bottom. The unit weight w = 56 psf and f'_m = 5000 psi. The f_y = 60 ksi. The wall is reinforced with #4 reinforcement. The spacing, s, is 18 inches. The roof load is 320 lb/ft applied through the ledger with a 3" eccentricity.

Required: Determine if the wall is adequate.

Solution:

1. Material Quantities
   a. Modulus of Elasticity
      $E_m = 1000\ f'_m = 5 \times 10^6$ psi
   b. Modular Ratio
      $n = E_s/E_m = 29 \times 10^6/E_m = 5.8$

2. Geometric Quantities
   a. Steel Ratio based on Gross Section
      $\rho_g = A_s/\text{spacing} \times t = 0.2/18 \times 5.5 = 0.002$
   b. Distance to Steel
      $d = t/2 = 5.5/2 = 2.75$ in.

3. Weights: Based on b = 12 in.

a. Superimposed Vertical Dead Load

$P_o = 320$ lb.

b. Weight of Wall

$w = 56$ psf; lateral load $= 0.3 \quad w = 0.3 \times 56 = 16.8$ psf

c. Total Weight of Wall

$P_p = 56 \times 1 \times 20 = 1120$ lb/ft

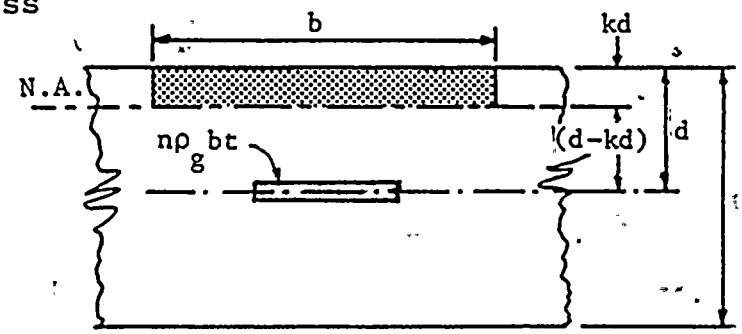
4. Deflection Calculation

a. Modulus of Rupture

$f_r = 2.5 \sqrt{f'_m} = 177$ psi

Section Modulus and Moment of Inertia Based on Gross Section

b. $S = bt^2/6 = 60.5$ in.^{3} $\quad I_{gross} = bt^3/12 = 166.38$ in.^{4}



c. Moment at Cracking

$M_{cr} = Sf_r = 10{,}700$ lb·in.

d. Cracked Moment of Inertia

$\alpha = 2n\,\rho_g d = 0.0638$

$\beta = 2n\,\rho_g d^2 = 0.1755$

$$kd = 0.5\left(-\alpha + \sqrt{\alpha^2 + 4\beta}\right) = 0.3882 \text{ in.}$$

Moment of Inertia of Cracked Section

$$I_{cr} = \frac{b\,kd^3}{3} + np_g bt\,(d - kd)^2 = 4.5047 \text{ in.}^4$$

e. Calculate Mid-Height Moment Under Service Load by Iteration

Set $\Delta^1 = 0$

$$M^2 = \frac{wh^2}{8} + \frac{P_p \Delta^1}{2} + P_o \Delta^1 + \frac{P_o e}{2} \qquad \text{(Eq. 7-11)}$$

$$= \frac{0.3 \times 56 \times 240^2}{8 \times 12} + \frac{1120 \times 0}{2} + 320 \times 0 + \frac{320 \times 5.75}{2}$$

$M^2 = 11{,}000$ lb. in.

Since $M^2 > M_{cr}$

$$\Delta^2 = \frac{5\,M_{cr}\,h^2}{48EI_{gross}} + \frac{5\,(M^2 - M_{cr})\,h^2}{48EI_{cracked}} \qquad \text{(Eq. 7-12)}$$

$$\Delta^2 = \frac{5 \times 10{,}700 \times 240^2}{48 \times 5 \times 10^6 \times 166.38} + \frac{5(11{,}000 - 10{,}700)240^2}{48 \times 5 \times 10^6 \times 4.5047}$$

$$= 0.0772 + 0.0799 = 0.1571 \text{ in.}$$

$$M^3 = M^2 + \frac{1120 \times 0.1571}{2} + 320 \times 0.1571$$

$$M^3 = 11{,}138 \text{ lb. in.}$$

By interpolation

$$\Delta^3 = 0.0772 + \frac{0.0799 \times (11{,}138 - 10{,}700)}{(11{,}000 - 10{,}700)}$$

$$\Delta^3 = 0.1939 \text{ in.}$$

$$M^4 = M^2 + \frac{1120 \times 0.1939}{2} + 320 \times 0.1939$$

$$= 11{,}166 \text{ lb. in.}$$

$$\Delta^4 = 0.0772 + \frac{0.0799\ (11{,}166 - 10{,}700)}{(11{,}000 - 10{,}700)}$$

$$\Delta^4 = 0.2013 \text{ in.}$$

f. Convergence (Eq. 7-13)

$$\frac{M^4 - M^3}{M^3} = \frac{11{,}166 - 11{,}138}{11{,}138} = 0.0025$$

Close enough - no need for check of deflection convergence.

g. Check h/Δ (Eq. 7-8)

$h/\Delta = 240/0.1963 = 1233 \gg 100$

Deflection requirement is satisfied

5. Strength Calculation

a. Calculate Mid-Height Moment Under factored Load

$w_u = 1.4 \times 0.3 \times 56 = 23.52$ psf

$P_{pu} = 0.75 \times 1.4 \times P_p = 1176$ lbs.

$P_{ou} = 0.75 \times 1.4 \times P_o = 336$ lbs.

$$M^2 = \frac{23.52 \times 20^2 \times 12}{8} + \frac{1176 \times 0}{2} + 336 \times 0 + \frac{336 \times 5.75}{2}$$

$$= 15{,}078 \text{ lb. in.}$$

Since $M^2 > M_{cr}$

$$\Delta^2 = 0.0772 + \frac{0.0799\ (15{,}078 - 10{,}700)}{(11{,}000 - 10{,}700)}$$

$$\Delta^2 = 1.243 \text{ in.}$$

$$M^3 = M^2 + \frac{1176 \times 1.243}{2} + 336 \times 1.243$$

$$= 16{,}227 \text{ lb. in.}$$

$$\Delta^3 = 0.0772 + \frac{0.0799\ (16{,}227 - 10{,}700)}{(11{,}000 - 10{,}700)}$$

$$= 1.549 \text{ in.}$$

$$M^4 = 15{,}078 + 1176 \times \frac{1.549}{2} + 336 \times 1.549$$

$$M^4 = 16{,}509 \text{ lb. in.}$$

b. Convergence

$$\frac{M^4 - M^3}{M^4} = \frac{16{,}509 - 16{,}227}{16{,}509} = 0.017$$

Close enough - no need for check of deflection convergence.

c. Find Nominal Strength, M_n

1) $c = (P_o + 0.5\ P_p + \rho_g bt\ f_y)/0.85\beta\ f'_m\ b$

$$= \frac{(336 + 0.5 \times 1176 + 0.002 \times 12 \times 5.5 \times 60{,}000)}{0.85 \times 0.80 \times 5000 \times 12}$$

$$c = 0.2168 \text{ in.}$$

2) $M_n = 0.85\beta\ f'_m\ bc\ (d/2 - \beta c/2) + \rho_g bt\ f_y\ d/2$

$$= 0.85 \times 0.80 \times 5000 \times 12 \times 0.2168\ (0.5 \times 275 - 0.80 \times 0.5 \times 0.2168) + 0.002 \times 12 \times 5.5 \times 60{,}000 \times 0.5 \times 2.75$$

$$M_n = 22{,}285 \text{ lb. in.}$$

d. $\phi \ M_n = 0.85 \times 22{,}285 = 18{,}942$ lb. in.

e. Since $M_u \approx M^4 = 16{,}509 < \phi \ M_n = 18{,}942$ lb. in.

Strength requirement is satisfied

EXAMPLE 2:

The example is repeated with different properties to illustrate a design which is controlled by deflection.

Given: A 23.8'-tall hollow brick masonry wall constructed with 5.50" units. The fully grouted wall is simply supported at the top and bottom. The unit weight w = 56 psf and $f'_m = 2500$ psi. The $f_y = 60$ ksi. The wall is reinforced with two #4 bundled bars spaced at 18 inches. The roof load is 320 lb/ft applied through the ledger with a 3" eccentricity.

Required: Determine if the wall is adequate.

Solution:

1. Material Quantities

   a. Modulus of Elasticity

   $E_m = 1{,}000 \ f'_m = 2.5 \times 10^6$ psi

   b. Modular Ratio

   $n = E_s/E_m = 29 \times 10^6/2.5 \times 10^6 = 11.6$

2. Geometric Quantities

   a. Steel Ratio Based on Gross Section

   $\rho_g = A_s/st = 0.4/18 \times 5.5 = 0.004$

   b. Distance to Steel

   $d = t/2 = 5.5/2 = 2.75$ in.

3. Weights - Base on b = 12 in.

   a. Superimposed Vertical Dead Load

   $P_o = 320$ lbs.

   b. Weight of Wall

   w = 56 psf; lateral load = 0.3 x 56 = 16.8 psf

   c. Total Weight of Wall

   $P_p = 56 \times 1 \times 23.8 = 1333$ lbs/ft

4. Deflection Calculation

a. Modulus of Rupture

$f_r = 2.5\sqrt{f'_m} = 125$ psi

Section Modulus and Moment of Inertia Based on Gross Section

b. $S = bt^2/6 = 60.5$ in.^{3}; $I_{gross} = bt^3/12 = 166.38$ in.^{4}

c. Moment at Cracking

$M_{cr} = Sf_r = 7563$ lb. in.

d. Cracked Moment of Inertia

$\alpha = 2\, n\rho_g d = 0.2552$

$\beta = w\, n\rho_g d^2 = 0.7018$

$kd = 0.5\left(-\alpha + \sqrt{\alpha^2 + 4\beta}\right) = 0.7198$ in.

$I_{cr} = \frac{bkd^3}{3} + n\rho_g bt\,(d - kd)^2 = 12.62$ in.^{4}

e. Calculate Mid-Height Moment Under Service Load by Iteration

Set $\Delta^1 = 0$.

$$M^2 = \frac{wh^2}{8} + \frac{P_p\,\Delta^1}{2} + P_o\,\Delta^1 + \frac{P_o e}{2} \qquad \text{(Eq. 7-11)}$$

$$= \frac{0.3 \times 56 \times 23.8^2 \times 12}{8} + \frac{1333 \times 0}{2} + 320 \times 0 + \frac{320 \times 5.75}{2}$$

$M^2 = 15{,}194$ lb. in.

Since $M^2 > M_{cr}$ (Eq. 7-12)

$$\Delta^2 = \frac{5M_{cr}\,h^2}{48EI_{gross}} + \frac{5\,(M^2 - M_{cr})\,h^2}{48EI_{cracked}}\ ;\ h = 23.8 \times 12 = 285.6 \text{ in.}$$

$$\Delta^2 = \left(\frac{5 \times 7563 \times 285.6^2}{2.5 \times 10^6 \times 166.38} + \frac{5 \times (15{,}194 - 7563) \times 285.6^2}{2.5 \times 10^6 \times 12.6223}\right) \times \frac{1}{48}$$

$= 0.1545 + 2.0547 = 2.2092$ in.

$$M^3 = M^2 + \frac{1333 \times 2.2092}{2} + 320 \times 2.2092$$

$$M^3 = 17{,}373 \text{ lb.-in.}$$

By interpolation

$$\Delta^3 = 0.1545 + \frac{2.0547 \times (17{,}373 - 7{,}563)}{(15{,}194 - 7{,}563)}$$

$$\Delta^3 = 2.7959 \text{ in.}$$

$$M^4 = M^2 + \frac{1{,}333 \times 2.7959}{2} + 2.7959 \times 320$$

$$= 17{,}952 \text{ lb.-in.}$$

$$\Delta^4 = 0.1545 + \frac{2.0547 \times (17{,}952 - 7{,}563)}{(15{,}194 - 7{,}563)}$$

$$\Delta^4 = 2.9518 \text{ in.}$$

f. Convergence (Eq. 7-13)

$$\frac{M^4 - M^3}{M^4} - \frac{17{,}952 - 17{,}373}{17{,}952} = 0.0323$$

Close enough - no need to check deflection convergence.

g. Check h/Δ (Eq. 7-8)

$h/\Delta = 23.8 \times 12/2.95 = 96.8$

5. Strength Calculation

a. Calculate Mid-Height Moment Under Factored Load

$w_u = 1.4 \times 0.3 \times 56 = 23.52$ psf

$P_{pu} = 0.75 \times 1.4 \times P_p = 1{,}400$ lbs.

$P_{ou} = 0.75 \times 1.4 \times P_o = 336$ lbs.

$$M^2 = \frac{23.52 \times 23.8^2 \times 12}{8} + \frac{1400 \times 0}{2} + 336 \times 0 + \frac{336 \times 5.75}{2}$$

$$= 20{,}950 \text{ lb.-in.}$$

Since $M^2 > M_{cr}$

$$\Delta^2 = 0.1545 + \frac{2.0547\ (20{,}950 - 7{,}573)}{(15{,}194 - 7{,}563)}$$

$$\Delta^2 = 3.759 \text{ in.}$$

$$M^3 = M^2 + \frac{1400 \times 3.759}{2} + 336 \times 3.759$$

$$= 24{,}844 \text{ lb.-in.}$$

$$\Delta^3 = 0.1545 + \frac{2.0547\ (24{,}844 - 7{,}563)}{(15{,}194 - 7{,}563)}$$

$$= 4.8075 \text{ in.}$$

$$M^4 = M^2 + \frac{1400 \times 4.8075}{2} + 336 \times 4.8075$$

$$= 25{,}931 \text{ lb.-in.}$$

$$M^4 = 25{,}931 \text{ lb.-in.}$$

b. Convergence (Eq. 7-13)

$$\frac{M^4 - M^3}{M^4} = \frac{25{,}931 - 24{,}844}{25{,}931} = 0.0419$$

Close enough - no need to check deflection convergence.

c. Final Nominal Strength, M_n

1. $c = (P_o + 0.5\ P_p + \rho\, bt\ f_y)/.85\beta_1\ f'_m\ b$

$$= \frac{(336 + 0.5 \times 1400 + .004 \times 12 \times 5.5 \times 60{,}000)}{.85 \times .85 \times 2500 \times 12}$$

$c = 0.7786$ in.

2. $M_n = .85\beta_1\ f'_m\ bc\ (d/2 - \beta_1 c/2) + \rho_g bt\ f_y\ d/2$

$$= .85 \times .85 \times 2500 \times 12 \times .7786\ \left(\frac{2.75 - 0.85 \times .7786}{2}\right)$$

$+0.004 \times 12 \times 5.5 \times 60{,}000 \times 0.5 \times 2.75$

$M_n = 39{,}400$ lb.-in.

d. $\phi M_n = 0.85 \times 39{,}400$ lb.-in. $= 33{,}490$ lb.-in.

e. Since $M_u \simeq M^4 = 25{,}931 < \phi M_n = 33{,}490$ lb.-in.

Strength requirement is satisfied

In the first example, both strength and service requirements were satisfied although strength is the controlling factor. While in the second example, deflection is the major factor. This latter design should be accepted since the deflection ratio is within 3% of the required value.

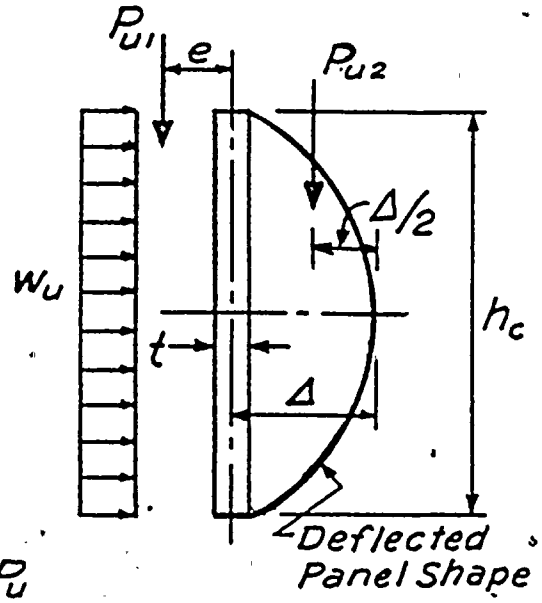
8.3 DESIGN EXAMPLES USING SEAOSC YELLOW BOOK METHOD

The research project was undertaken to test the design procedures outlined in the SEAOSC Yellow Book (Ref. 30) and to develop new techniques. The performances of the test specimens and the analysis of results verify the design procedures stated in Reference 30. In some cases, the examples presented here modify the Yellow Book Method so that it is adaptable for masonry design. The modifications apply to modulus of elasticity, modulus of rupture, ϕ factors, etc., and to deflection control.

SEAOSC YELLOW BOOK METHOD

with additional provisions for masonry design

SLENDERNESS ANALYSIS



When tension controls, the effect of axial load can be accounted for by adding the axial load to the force in the reinforcing steel.

For the panel shown, the maximum deflection to determine the $P\Delta$ effect will occur when the nominal moment strength M_n at the midheight of the wall is reached.

$M_n = A_s(\text{eff.}) f_y (d - a/2) =$ Nominal Mom. Str.

Where: $A_s(\text{eff.}) = \dfrac{P_u + A_s f_y}{f_y} \simeq A_s$ for small P_u

$$a = \frac{P_u + A_s f_y}{0.85 f_c' b} = 0.85c$$

$\phi = 0.90 - 2.0 P_u / f_c' A_g$; for small P_u neglect second term:

For values of ϕ for concrete and brick masonry see Table 7-4.

$M_u = w_u h_c^2/8 + P_{u1} e/2 + (P_{u1} + P_{u2})\Delta_n$; where $\Delta_n = \dfrac{5 M_n h_c^2}{48 E_c I_{cr}}$

$E_c = 57{,}000\sqrt{f_c'}$ for normal weight concrete.

$E_m = 1000 f_m'$ for concrete and brick masonry

$I_{cr} = n A_s(\text{eff.})(d-c)^2 + bc^3/3$, and $I_g = bt^3/12$

When $M_u \leq \phi M_n$ wall section is adequate for strength.

Service Deflection Limit $= \Delta_s = h_c/100$

$M_{cr} = f_r I_g / y_t$; where $y_t = t/2$; For values of f_r see Table 7-4.

$\Delta_{cr} = 5 M_{cr} h_c^2 / 48 E_c I_g$

At service loads: $M_s = w h_c^2/8 + P_1 e/2 + (P_1 + P_2)\Delta_s$

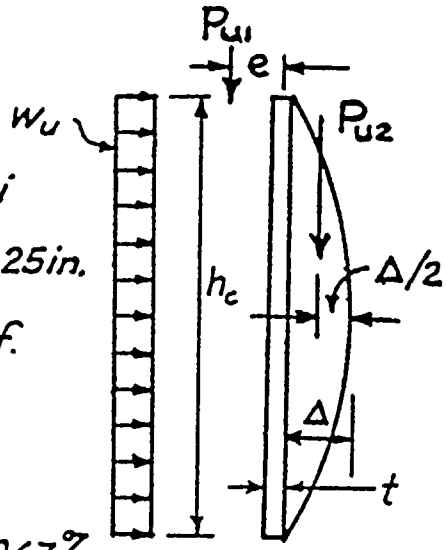
$$\Delta_s = \Delta_{cr} + \frac{M_s - M_{cr}}{M_n - M_{cr}} (\Delta_n - \Delta_{cr})$$

When $\Delta_s \leq h_c/100$ wall section satisfies deflection limit.

DESIGN EXAMPLE NO.3

Work sheet 1 of 2

TILT-UP CONCRETE PANEL



Wall on continuous footing

$f_c' = 3000$ psi @ 28 days, $f_y = 60{,}000$ psi

Panel Height $= h_c = 24.0$ ft., $e = t/2 + 3.5 = 6.25$ in.

Tributary Roof Dead Load $= P_1 = 320$ plf.

Trial section and reinforcing:

Select $t = 5\frac{1}{2}$ in., Reinf. #4 @ 9" o.c.

$A_s = 0.20 \times 12/9 = 0.267$ in.²/ft.; $0.50\rho_b = 1.067\%$

$A_g = b \times t = 12 \times 5.5 = 66$ in.²; $\rho = A_s/bd = 0.267/12 \times 2.75 = 0.008 < \rho_b$

Panel Wt. $= 150 \times 5.5/12 = 68.8$ psf; Half Panel Wt. $= P_2 = 68.8 \times 12 = 825$ plf.

Seismic force $\doteq w = 0.3 \times 68.8 = 20.6$ psf.

ULTIMATE LOADS: (Ref. U.B.C. Sec. 2609(d)), $U = .75(1.4D + 1.7L + 1.87E)$

$P_{u1} = 0.75 \times 1.4 \times 320 = 336$ plf

$P_{u2} = 0.75 \times 1.4 \times 825 = 866$ plf; $P_{u1} + P_{u2} = 1202$ plf. $= P_u$

$w_u = 0.75 \times 1.87 \times 20.6 = 28.9$ psf.

NOMINAL MOMENT STRENGTH

$$A_s(\text{effective}) = \frac{P_u + A_s f_y}{f_y} = \frac{1202 + .267 \times 60000}{60000} = 0.29 \text{ in}^2 = A_{se}$$

$$a = \frac{P_u + A_s f_y}{0.85 f_c' b} = \frac{1202 + 16020}{.85 \times 3000 \times 12} = 0.563 \text{ in.}$$

$c = a/0.85 = 0.563/0.85 = 0.662$ in.

$\phi = 0.90$ (See Table 7-4)

$M_n = A_{se} f_y (d - a/2) = 0.29 \times 60{,}000(2.75 - .563/2) = 42{,}980$ in-lbs.

$\phi M_n = 0.90 \times 42{,}980 = 38{,}680$ in-lbs. $= 38.7$ in-kips

FACTORED ULTIMATE MOMENT

$E_c = 57000\sqrt{f_c'} = 57000\sqrt{3000} = 3{,}120{,}000\ psi$

$n = E_s/E_c = 29{,}000{,}000/3{,}120{,}000 = 9.3$

$I_{cr} = n\,A_{se}(d-c)^2 + bc^3/3$

$= 9.3 \times .287(2.75-.662)^2 + 12 \times .662^3/3 = 12.80\ in.^4$

$\Delta_n = \dfrac{5\,M_n\,h_c^2}{48\,E_c\,I_{cr}} = \dfrac{5 \times 43.0 \times \overline{24 \times 12}^2}{48 \times 3{,}120 \times 1280} = 9.3\ in.$

$M_u = \dfrac{w_u \cdot h_c^2}{8} + P_{u1} \cdot e/2 + (P_{u1} + P_{u2})\Delta_n$

$= 28.9 \times 24^2 \times 12/8 + 336 \cdot 6.25/2 + 1202 \cdot 9.3$

$= 24{,}970 + 1{,}050 + 11{,}180 = 37{,}200\ in\text{-}lb. = 37.2\ in\text{-}kips$

The capacity of 38.7 exceeds the applied moment of 37.2, section O.K.

CHECK ON DEFLECTION LIMITATION

Max. $\Delta_s = h_c/100$ at unfactored service loads

$f_r = 5\sqrt{f_c'} = 5\sqrt{3000} = 274\ psi;\ I_g = bt^3/12 = 166\ in.^4$

$M_{cr} = f_r \cdot I_g/y_t = .274 \times 166/2.75 = 16.5\ in.\text{-}kips$

$\Delta_{cr} = \dfrac{5\,M_{cr}\,h_c^2}{48\,E_c\,I_g} = \dfrac{5 \cdot 13.2 \times \overline{24 \times 12}^2}{48 \cdot 3120 \times 166} = 0.275\ in.$

At unfactored service loads assume $\Delta = h_c/100 = 2.88\ in.$

$M_s = w\,h_c^2/8 + P_1 \cdot e/2 + (P_1 + P_2)\Delta$

$= 20.6 \times 24^2 \cdot 12/8 + 320 \cdot 6.25/2 + (320 + 825)2.88$

$= 17{,}800 + 1{,}000 + 3{,}300 = 22{,}100\ in\cdot lb. = 22.1\ in\text{-}kips$

$\Delta_s = \Delta_{cr} + \dfrac{M_s - M_{cr}}{M_n - M_{cr}}(\Delta_n - \Delta_{cr})$

$= 0.275 + \dfrac{22.1 - 16.5}{43.0 - 16.5}(9.3 - 0.275) = 2.18\ in.$

The service deflection is within limits, section is adequate.

FACTORED ULTIMATE MOMENT

$E_c = 57000\sqrt{f'_c} = 57000\sqrt{3000} = 3{,}120{,}000\ psi$

$n = E_s/E_c = 29{,}000{,}000/3{,}120{,}000 = 9.3$

$I_{cr} = n\,A_{se}(d-c)^2 + bc^3/3$

$= 9.3 \times .287(2.75-.662)^2 + 12 \times .662^3/3 = 12.80\ in.^4$

$\Delta_n = \dfrac{5\,M_n\,h_c^2}{48\,E_c\,I_{cr}} = \dfrac{5 \times 43.0 \times \overline{24 \times 12}^2}{48 \times 3{,}120 \times 1280} = 9.3\ in.$

$M_u = \dfrac{w_u \cdot h_c^2}{8} + P_{u1} \cdot e/2 + (P_{u1} + P_{u2})\Delta_n$

$= 28.9 \times 24^2 \times 12/8 + 336 \cdot 6.25/2 + 1202 \cdot 9.3$

$= 24{,}970 + 1{,}050 + 11{,}180 = 37{,}200\ in\text{-}lb. = 37.2\ in\text{-}kips$

The capacity of 38.7 exceeds the applied moment of 37.2, section O.K.

CHECK ON DEFLECTION LIMITATION

Max. $\Delta_s = h_c/100$ at unfactored service loads

$f_r = 5\sqrt{f'_c} = 5\sqrt{3000} = 274\ psi;\ I_g = bt^3/12 = 166\ in.^4$

$M_{cr} = f_r \cdot I_g/y_t = .274 \times 166/2.75 = 16.5\ in.\text{-}kips$

$\Delta_{cr} = \dfrac{5\,M_{cr}\,h_c^2}{48\,E_c\,I_g} = \dfrac{5 \times 13.2 \times \overline{24 \times 12}^2}{48 \times 3120 \times 166} = 0.275\ in.$

At unfactored service loads assume $\Delta = h_c/100 = 2.88\ in.$

$M_s = w\,h_c^2/8 + P_1 \cdot e/2 + (P_1 + P_2)\Delta$

$= 20.6 \times 24^2 \times 12/8 + 320 \cdot 6.25/2 + (320 + 825)\,2.88$

$= 17{,}800 + 1{,}000 + 3{,}300 = 22{,}100\ in\cdot lb. = 22.1\ in\text{-}kips$

$\Delta_s = \Delta_{cr} + \dfrac{M_s - M_{cr}}{M_n - M_{cr}}(\Delta_n - \Delta_{cr})$

$= 0.275 + \dfrac{22.1 - 16.5}{43.0 - 16.5}(9.3 - 0.275) = 2.18\ in.$

The service deflection is within limits, section is adequate.

DESIGN EXAMPLE NO. 4

CONCRETE MASONRY PANEL

Design based on modified SEAOSC Yellow Book method.

$f'_m = 1500\,psi$; $f_y = 60{,}000\,psi$. See sketch for Example 1

Panel Ht. $= h_c = 20.0\,ft.$; $e = 6.3\,in.$

Tributary Roof Dead Load $= P_1 = 320\,plf$

Trial selection: 6" CMU ($t = 5.63"$); Reinf. #5 @ 16" o.c.

$A_s = 0.31 \times 12/16 = 0.233\ in.^2/ft.$; $A_g = bt = 12 \times 5.63 = 67.6\ in.^2$

$d = t/2 = 2.81\ in.$; $\rho = A_s/bd = 0.233/12 \times 2.81 = 0.0069 < \rho_b$

Panel Wt. $= 6 \times 10 = 60\,psf$; Half Panel Wt. $= 60 \times 20/2 = 600\,plf = P_2$

Seismic force $= w = 0.3 \times 60 = 18\,psf$

ULTIMATE LOADS: (Ref. U.B.C. Sec. 2609d); $U = .75(1.4D + 1.4L + 1.8$

$P_{u1} = 0.75 \times 1.4 \times 320 = 336\,plf$

$P_{u2} = 0.75 \times 1.4 \times 600 = 630\,plf$; $P_{u1} + P_{u2} = 966\,plf = P_u$

$w_u = 0.75 \times 1.87 \times 18 = 25.2\,psf$

NOMINAL MOMENT STRENGTH

$$A_s(\text{effective}) = \frac{P_u + A_s f_y}{f_y} = \frac{966 + .233 \cdot 60{,}000}{60{,}000} = 0.249\ in^2 = A_{se}$$

$$a = \frac{P_u + A_s f_y}{.85 f'_m b} = \frac{966 + .233 \times 60{,}000}{.85 \times 1500 \times 12} = 0.977\ in.$$

$c = a/.85 = 0.977/.85 = 1.149\ in.$

$\phi = 0.80$ (See Table 7-4, CMU Special Inspection)

$M_n = A_{se} f_y (d - a/2)$

$= 0.249 \cdot 60{,}000(2.81 - .977/2) = 34{,}690\ in\text{-}lbs. = 34.7\ in\text{-}kips$

$\phi M_n = 0.80 \times 34.7 = 27.8\ in\text{-}kips$

DESIGN EXAMPLE NO. 5

BRICK MASONRY PANEL ON CONTINUOUS FOOTING

Design based on modified SEAOSC Yellow Book method

$f'_m = 1800$ psi, $f_y = 60{,}000$ psi. See sketch for Example 1.

Panel Ht. $= h_c = 27.0$ ft.; $e = 8.0$ in.

Tributary Roof Dead Load $= 320$ plf. $= P_1$

Trial section: 9" Brick Wall ($t = 9.0$"); Reinf. #6 @ 16" o.c.

$A_s = 0.44 \cdot 12/16 = 0.33$ in²/ft.; $A_g = bt = 12 \cdot 9 = 108$ in²

$d = t/2 = 9/2 = 4.5$ in.; $\rho = A_s/bd = 0.33/12 \times 4.5 = 0.006 < \rho_b$

Panel Wt. $= 9 \times 10 = 90$ psf; Half Panel Wt. $= 90 \times 27/2 = 1215$ plf $= P_2$

Seismic force $= w = 0.3 \cdot 90 = 27$ psf

ULTIMATE LOADS: (Ref. U.B.C. Sec. 2609d); $U = .75(1.4D + 1.4L + 1.87E)$

$P_{u1} = 0.75 \times 1.4 \times 320 = 336$ plf

$P_{u2} = 0.75 \cdot 1.4 \times 1215 = 1276$ plf; $P_{u1} + P_{u2} = 1612$ plf $= P_u$

$w_u = 0.75 \cdot 1.87 \times 27 = 37.9$ psf

NOMINAL MOMENT STRENGTH

$$A_s(\text{effective}) = \frac{P_u + A_s f_y}{f_y} = \frac{1612 + .33 \cdot 60{,}000}{60{,}000} = 0.357 \text{ in}^2 = A_{se}$$

$$a = \frac{P_u + A_s f_y}{.85 f'_m b} = \frac{1612 + 0.33 \times 60{,}000}{.85 \times 1800 \times 12} = 1.166 \text{ in.}$$

$c = a/.85 = 1.166/.85 = 1.372$ in.

$\phi = 0.70$ (see Table 7-4, Brick, Special Inspection).

$M_n = A_{se} f_y (d - a/2)$

$= .357 \times 60{,}000(4.5 - 1.166/2) = 83{,}900$ in·lb. $= 83.9$ in-kips

$\phi M_n = 0.70 \times 83.9 = 58.7$ in-kips.

FACTORED ULTIMATE MOMENT.

$E_m = 1000 f'_m = 1000 \times 1800 = 1{,}800{,}000$ psi

$n = E_s / E_c = 29{,}000{,}000 / 1{,}800{,}000 = 16.1$

$I_{cr} = n A_{se} (d-c)^2 + bc^3/3$

$= 16.1 \times .357 (4.5 - 1.372)^2 + 12 \times 1.372^3/3 = 66.57 \text{ in.}^4$

$\Delta_n = \dfrac{5 M_n h_c^2}{48 E_m I_{cr}} = \dfrac{5 \times 83.9 \times \overline{27 \times 12}^2}{48 \times 1800 \times 66.57} = 7.66$ in.

$M_u = w_u h_c^2/8 + P_{u1} e/2 + (P_{u1} + P_{u2}) \Delta_n$

$= 37.9 \times 27^2 \times 12/8 + 336 \times 8.0/2 + 1612 \times 7.66$

$= 41{,}440 + 1{,}340 + 12{,}350 = 55{,}130$ in-lbs. $= 55.13$ in-kips

The capacity of $58.7 > 55.13$ so section is O.K.

CHECK ON DEFLECTION LIMITATION

Max. $\Delta_s = h_c/100 = 3.24''$ at unfactored service loads

$f_r = 2.0\sqrt{f'_m} = 2.0\sqrt{1800} = 85$ psi; $I_g = bt^3/12 = 729 \text{ in.}^4$

$M_{cr} = f_r I_g / y_t = .085 \times 729/4.5 = 13.77$ in-kips

$\Delta_{cr} = \dfrac{5 M_{cr} h_c^2}{48 E_m I_g} = \dfrac{5 \times 13.77 \times \overline{27 \times 12}^2}{48 \times 1800 \times 729} = 0.115$ in.

At unfactored service loads, assume $\Delta = h_c/100 = 3.24$ in.

$M_s = w h_c^2/8 + P_1 e/2 + (P_1 + P_2) \Delta$

$= 27 \times 27^2 \times 12/8 + 320 \times 8.0/2 + 1535 \times 3.24$

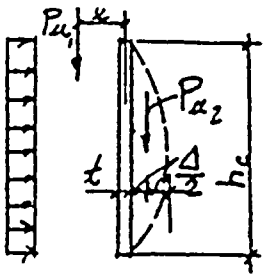
$= 29{,}520 + 1280 + 4970 = 35{,}770$ in-lb. $= 35.8$ in.-kips

$\Delta_s = \Delta_{cr} + \dfrac{M_s - M_{cr}}{M_n - M_{cr}} (\Delta_n - \Delta_{cr})$

$= .115 + \dfrac{35.8 - 13.77}{83.9 - 13.77} (7.66 - .115)$

$= .115 + 2.48 = 2.60$ in.

The service deflection is within limits, section is O.K.



Material : Concrete Wall on continuous footing

f'_c = 2000 psi

f_y = 60000 psi

P_1 = 320 plf

e = 6.25 in.

h_c = 21.0 ft.

Trial : 5 1/2" wall Rebar : #4 @ 9" O.C

t = 5.5 in. d = t/2 = 2.75 in.

A_s = 0.267 sq. in./ft. A_g = b x t = 66.0 in.^{2}

Panel wt. = 68.8 psf ρ_g = 0.405 %

P_2 = 68.8 x 21.0/2 = 722 plf Wall wt. @ mid ht.

Seis. w = 0.3 x 68.8 = 20.6 psf

Ultimate loads :

P_{u1} = 320 x 1.4 x 0.75 = 336

P_{u2} = 722 x 1.4 x 0.75 = 758

P_u = P_{u1} + P_{u2} = 1094 plf

w_u = 20.6 x 1.87 x 0.75 = 28.9 psf

Nominal Moment Strength

A_s Eff = (P_u + A_s f_y) / f_y = $\frac{1094 + 0.267 \times 60000}{60000}$ = 0.285 sq.in.

a = (P_u + A_s f_y) / .85f'_cb = $\frac{17100}{0.85 \times 2000 \times 12}$ = 0.84 in.

c = a / .85 = 0.84 / .85 = 0.99 in.

Ø = 0.90 x 0.8 = 0.72

M_n = A_s Eff f_y (d - a/2) = 0.285 x 60 $(2.75 - \frac{0.84}{2})$ = 39.8 "k

Ø M_n = 0.72 x 39.8 = 28.7 "k

Factored Moment

$E_c = 57000\,(f'_c)^{\frac{1}{2}} = 57000 \times (2000)^{\frac{1}{2}} = 2.55 \times 10^6$ psi

$n = E_s / E_c = 29000 / 2550 = 11.4$

$I_{cr} = n\,A_{sEff}\,(d - c)^2 + bc^3/3$

$= 11.4 \times 0.285\,(2.75 - 0.99)^2 + 12 \times (0.99)^3 \div 3$

$= 10.1 + 3.9 = 14.0$ in.^{4}

$\Delta_n = 5\,M_n\,h_c^2 \div (48\,E_c I_{cr}) = \dfrac{5 \times 39.8 \times \overline{21.0}^2 \times 144}{48 \times 2550 \times 14.0} = 7.38$ in.

$M_u = w_u h_c^2 \times 1.5 + P_{u1}\,e/2 + (P_{u1} + P_{u2})\,\Delta_n$

$= 28.9 \times \overline{21.0}^2 \times 1.5 + 336 \times \dfrac{6.25}{2} + 1094 \times 7.38$

$= 19.12 + 1.05 + 8.07 = 28.2$ "k $< \phi M_n = 28.7$ "k

Wall section is (~~is not~~) adequate for strength requirement.

Check Deflection Limitation

$f_r = 5 \times (f'_c)^{\frac{1}{2}} = 5 \times (2000)^{\frac{1}{2}} = 224$ psi

$I_g = \frac{1}{12} \times 12 \times t^3 = t^3 = 166$ in.^{4}

$M_{cr} = f_r \times I_g / y_t = 0.224 \times 166 / 2.75 = 13.5$ "k

$\Delta_{cr} = 5\,M_{cr} h_c^2 \div (48\,E_c\,I_g) = \dfrac{5 \times 13.5 \times \overline{21}^2 \times 144}{48 \times 2550 \times 166} = 0.21$ in.

At service load: Assume $\Delta = h_c / 100 = 252/100 = 2.52$ in.

$M_s = w\,h_c^2 \times 1.5 + P_1\,e/2 + (P_1 + P_2)\,\Delta$

$= 20.6 \times (21.0)^2 \times 1.5 + 320 \times \dfrac{6.25}{2} + 1094 \times 2.52$

$= 13.63 + 1.00 + 2.76 = 17.4$ "k

$\Delta_s = \Delta_{cr} + \dfrac{M_s - M_{cr}}{M_n - M_{cr}}\,(\Delta_n - \Delta_{cr})$

$= 0.21 + \dfrac{17.4 - 13.5}{39.8 - 13.5} \times (7.38 - 0.21)$

$= 0.21 + 1.06 = 1.27$ in. $< h_c / 100$

Wall section is (~~is not~~) adequate for slenderness requirements.

Factored Moment

$E_m = 1000\ f'_m = 1000 \times 1500 = 1.5 \times 10^6$ psi

$n = E_s / E_m = 29000 \div 1.5 \times 10^3 = 19.3$

$I_{cr} = n\ A_{s\ eff}\ (d - c)^2 + b\ c^3 \div 3$

$= 19.3 \times 0.173 (3.81 - 0.80)^2 + \frac{12}{3} \times (0.80)^3$

$= 30.3 + 2.0 = 32.3$ in.^{4}

$\Delta_n = 5M_n h_c^2 \div (48\ E_m\ I_{cr}) = \frac{5 \times 36.0 \times (18.0)^2 \times 144}{48 \times 1500 \times 32.3} = 3.61$ in.

$M_u = w_u\ h_c^2 \times 1.5 + P_{u1}\ e / 2 + (P_{u1} + P_{u2})\ \Delta_n$

$= 33.6 \times (18.0)^2 \times 1.5 + 336 \times \frac{7.3}{2} + (1092) \times 3.61$

$= 16.33 + 1.23 + 3.94 = 21.5$ "k $< \phi M_n = 23.0$

Wall section is (~~is not~~) adequate for strength requirement.

Deflection Limitation

$f_r = 2.5\ (f'_m)^{\frac{1}{2}} = 2.5 \times (1500)^{\frac{1}{2}} = 96.8$ psi

$I_g = b\ t^3 / 12 = \frac{1}{12} \times 12 \times (7.62)^3 = 442$ in.^{4}

$M_{cr} = f_r \times I_g / y_t = 96.8 \times \frac{442}{3.81} = 11.2$ "k

$\Delta_{cr} = 5M_{cr} h_c^2 \div (48\ E_m I_g) = \frac{5 \times 11.2 \times (18.0)^2 \times 144}{48 \times 1500 \times 442} = 0.08$ in.

At service load: Assume $\Delta = h_c / 100 = 12 \times 18 / 100 = 2.16$ in.

$M_s = w\ h_c^2 \times 1.5 + P_1 e/2 + (P_1 + P_2)\Delta$

$= 24.0 \times (18.0)^2 \times 1.5 + 320 \times \frac{7.3}{2} + (320 + 720) \times 2.16$

$= 11.66 + 1.17 + 2.25 = 15.1$ "k

$\Delta_s = \Delta_{cr} + \frac{M_s - M_{cr}}{M_n - M_{cr}} (\Delta_n - \Delta_{cr})$

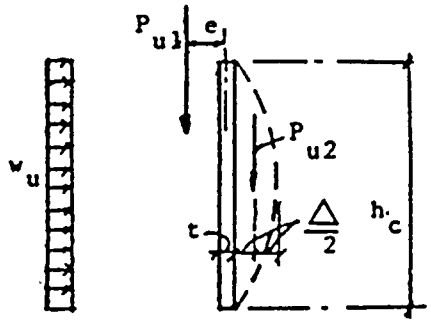
$= 0.08 + \frac{15.1 - 11.2}{36.0 - 11.2} \times (3.61 - 0.08)$

$= 0.08 + 0.56 = 0.64$ in. $<< h_c/100$

Wall section is (~~is not~~) adequate for slenderness requirement.

DESIGN EXAMPLE NO.7

Work Sheet
1 of 2



Material : CMU

f'_m = 1500 psi (Called Inspection)

f_y = 60,000 psi

P_1 = 320 plf

e = 7.3 in.

h_c = 18.0 ft.

Trial : 8" CMU Rebar : #5 @ 24

t = 7.62 in. d = t/2 = 3.81 in.

A_s = 0.155 sq. in./ft. A_g = b x t = 7.625 x 12 = 91.5 in.^{2}

Panel wt. = 80 psf ρ_g = 0.169 % < 0.32

P_2 = 80 x 18.0 ÷ 2 = 720 plf Wall wt. @ mid-height

Seis. w = 80 x 0.3 = 24.0 psf

Ultimate loads :

P_{u1} = 1.4 x 320 x 0.75 = 336

P_{u2} = 1.4 x 720 x 0.75 = 756

P_u = P_{u1} + P_{u2} = 1092 plf

Seis. w_u = 1.4 x 24.0 = 33.6 psf

Nominal Moment Strength

$A_{s\ Eff} = (P_u + A_s f_y)/f_y = \frac{1092 + 0.155 \times 60000}{60000} = 0.173$ in.^{2}

$a = (P_u + A_s f_y)/(.85 f'_m b) = \frac{10390}{.85 \times 1500 \times 12} = 0.68$ in.

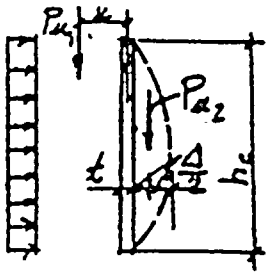
c = a ÷ 0.85 = 0.68 ÷ 0.85 = 0.80 in.

Ø = 0.80 x 0.8 = 0.64

$M_n = A_{s\ Eff}\ f_y\ (d - a/2)$

$= 0.173 \times 60\ (3.81 - \frac{0.68}{2}) = 36.0$ "k

$Ø\ M_n$ = 0.64 x 36.0 = 23.0 "k



Material : Concrete Wall on continuous footing

f'_c = 3,000 psi

f_y = 60,000 psi

P_1 = 320 plf

e = 7.0 in.

h_c = 28.0 ft.

Trial : 7" wall . Rebar : #5 @ 11

t = 7.0 in. d = t/2 = 3.5 in.

A_s = 0.338 sq. in./ft. A_g = b x t = 84.0 in.^{2}

Panel wt. = 87.5 psf ρ_g = 0.403%

P_2 = 87.5 x 28 /2 = 1225 plf Wall wt. @ mid ht.

Seis. w = 0.3 x 87.5 = 26.3 psf

Ultimate loads :

P_{u1} = 320 x 1.4 x 0.75 = 336

P_{u2} = 1225 x 1.4 x 0.75 = 1286

P_u = P_{u1} + P_{u2} = 1622 plf

w_u = 26.3 x 1.4 = 36.8 psf

Nominal Moment Strength

A_s Eff = (P_u + A_s f_y) / f_y = $\frac{1622 + 0.338 \times 60000}{60000}$ = 0.365 sq.in.

a = (P_u + A_s f_y) /.85f'_cb = $\frac{21900}{0.85 \times 3000 \times 12}$ = 0.72 in.

c = a / .85 = 0.72 / .85 = 0.84 in.

Ø = 0.90 - 2.0P_u/$f'_c A_g$ = 0.90 - $\frac{2.0 \times 1622}{3000 \times 84.0}$ = 0.887

M_n = A_s Eff f_y (d - a/2) = 0.365 x 60 (3.50 - $\frac{0.72}{2}$) = 68.8 "k

Ø M_n = 0.887 x 68.8 = 61.0 "k

Factored Moment

$E_c = 57000\ (f'_c)^{\frac{1}{2}} = 57000 \times (3000)^{\frac{1}{2}} = 3.12 \times 10^6$ psi

$n = E_s / E_c = 29000 / 3120 = 9.3$

$I_{cr} = n\ A_{sEff}\ (d - c)^2 + bc^3/3$

$= 9.3 \times 0.365\ (3.5 - 0.84)^2 + 12 \times (0.84)^3 \div 3$

$= 24.0 + 2.4 = 26.4$ in.^{4}

$\Delta_n = 5\ M_n\ h_c^2 \div (48\ E_c I_{cr}) = \dfrac{5 \times 68.8 \times \overline{28.0}^2 \times 144}{48 \times 3120 \times 26.4} = 9.82$ in.

$M_u = w_u h_c^2 \times 1.5 + P_{u1}\ e / 2 + (P_{u1} + P_{u2})\ \Delta_n$

$= 36.8 \times \overline{28}^2 \times 1.5 + 336 \times \dfrac{7.0}{2} + 1622 \times 9.82$

$= 43.3 + 1.2 + 15.9 = 60.4$ "k $< \phi M_n = 61.0$

Wall section is (~~is not~~) adequate for strength requirement.

Check Deflection Limitation

$f_r = 5 \times (f'_c)^{\frac{1}{2}} = 5 \times (3000)^{\frac{1}{2}} = 274$ psi

$I_g = \frac{1}{12} \times 12 \times t^3 = t^3 = 343$ in.^{4}

$M_{cr} = f_r \times I_g / y_t = 274 \times 343 / 3.5 = 26.8$ "k

$\Delta_{cr} = 5\ M_{cr} h_c^2 \div (48\ E_c\ I_g) = \dfrac{5 \times 26.8 \times \overline{28}^2 \times 144}{48 \times 3120 \times 343} = 0.30$ in.

At service load: Assume $\Delta = h_c / 100 = 336 / 100 = 3.36$ in.

$M_s = w\ h_c^2 \times 1.5 + P_1\ e / 2 + (P_1 + P_2)\ \Delta$.

$= 26.3 \times (28)^2 \times 1.5 + 320 \times \dfrac{7.0}{2} + 1545 \times 3.36$

$= 30.9 + 1.1 + 5.2 = 37.2$ "k

$\Delta_s = \Delta_{cr} + \dfrac{M_s - M_{cr}}{M_n - M_{cr}} (\Delta_n - \Delta_{cr})$

$= 0.30 + \dfrac{37.2 - 26.8}{68.8 - 26.8} \times (9.82 - 0.30)$

$= 0.30 + 2.36 = 2.66$ in. $< h_c / 100$

Wall section is (~~is not~~) adequate for slenderness requirements.

SECTION 9

CONCLUSIONS, RECOMMENDATIONS, AND OTHER CONSIDERATIONS

9.1 CONCLUSIONS

1. Buckling. There was no evidence of elastic and inelastic lateral instability (buckling) for the load ranges tested, which were primarily lateral loads with axial loads less than 1/10 of the short column axial capacity.

2. Pe Moment Effect. The significance of the eccentric moment from the applied simulated light framing roof load was small.

3. PΔ Moment Effect. The significance of the P-Δ moment was most pronounced in the thinner panels but did not produce lateral instability in the load ranges tested. Panel weight was the largest component of secondary moments. Secondary moments accounted for approximately 20% of the total moment at yield of the reinforcement.

4. Load Deflection. Load deflection characteristics of the panels can be approximated by three straight lines representing the uncracked stage, the cracked stage, and the postyielding stage. The intersection points of these lines are a function of the moment capacity at modulus of rupture of the concrete or of the mortar joints in masonry and the moment capacity of the wall section at yielding of the reinforcement.

Excellent correlation of panel midspan moment versus deflection plots can be obtained with test resuls by drawing a straight line from the origin to the moment at first crack and the moment at initial yield of reinforcement. The lines represent the uncracked to cracked to yield deflection stages.

5. Load Moment Curves. The interaction P-M (Load-Moment) curves for short columns provided an adequate predicted moment capacity envelope for both masonry and concrete panels when loaded with relatively low axial loads that are much smaller than balance point on the P-M curve.

6. EI Value. Tests showed that the product of the cracked transformed section moment of inertia and the code modulus of elasticity was useful in predicting midspan deflection of the panel at yield level of the reinforcement. This was true for both concrete and masonry walls.

7. Residual Deflection. Although the panels exhibited adequate strength at and beyond the yield point, the rebound study indicates that a midpoint permanent deflection can be expected for panels loaded to the yield level of the reinforcement.

8. No h/t Limitation. The tests demonstrated that there was no validity for fixed height-to-thickness limits, but they did reveal the need for deflection limits to control potential residual deflection in panels after service loads experience.

9.2 RECOMMENDATIONS

1. Minimum Reinforcement. Moment capacity of cracked section at yielding of reinforcement should be greater than or at least equal to the moment capacity of the uncracked section based on a gross section tensile strength of $5\sqrt{f'_c}$ for concrete, $2.5\sqrt{f'_m}$ for hollow block masonry, and $2.0\sqrt{f'_m}$ for solid brick masonry.

2. Deflection Control. The adoption of deflection control is a new feature and is used to assure a wall of reasonable straightness after a service level loading. It should prevent

excessive deflection service load level, and also use of a panel with excessive flexibility. It is recommended that midheight deflection be limited to height divided by 100, that is $\Delta = \frac{h}{100}$.

3. ϕ Factor. The ϕ factor has been introduced to reflect effective quality control relating to material and construction practices. It is suggested the ϕ factor be used to account for the differences in construction with continuous and noncontinuous inspection, for both concrete and masonry construction. It is recommended that a ϕ factor of 80% of the factor for special inspected work be used for noncontinuous inspection.

4. Maximum Amount of Steel. The maximum flexural steel ratio, ρ_g, based on gross area should be limited to the value given in Table 7-1, Maximum Design ρ_g for the Units and Material Shown. This limitation on the amount of steel is to assure that there will be a ductile yielding condition and never a brittle failure of the concrete or masonry.

9.3 OTHER CONSIDERATIONS

1. Design and Construction without Continuous Inspection.

All the test panels were constructed under close supervision and continuous inspection. For construction that does not have continuous inspection, the current code requires that the allowable stresses in masonry design be arbitrarily reduced by one-half. In essence this provides for an increased factor of safety to cover for the uncertainty of the quality of the construction and material.

Some of the concerns were the strength of the materials and the location and yield strength of the steel. Computer studies were made to determine the capacity of numerous walls where

these variables were considered. For all economically practical configurations, these computer studies conclusively showed that compressive stresses in the masonry and concrete were not the limiting criteria. However, the amount, the location, and the yield strength of the reinforcement always governed the ultimate capacity of the walls.

The consensus of the Committee was that construction without continuous inspection can be handled by an adjustment of the ϕ factor in the design. By reducing the ϕ factor, the adjustment is made on the resistance side of the design equation. This more properly reflects the original intent of the ϕ factor, rather than changing the loading side of the design equation. The suggested ϕ factor has been reduced to an amount that will adjust for these variations of materials and workmanship, including steel location. The resulting design penalizes uninspected walls by approximately 15% in height. This means that for the same amount of steel, the uninspected walls would have to be 15% shorter than inspected walls of the same thickness.

This suggested method of design recognizes the large amount of construction built throughout the United States where continuous inspection is not available or is not used, and provides a guide to architects and engineers for the design of concrete and masonry walls without continuous inspection.

2. Partially Grouted Walls. The masonry walls tested were all solid grouted. In pactice, many walls are not solid grouted but are only grouted at the steel location. Although all walls tested were solid grouted, the same principles of design and performance should prevail for walls that are only partially grouted. From the analysis it was evident that the stress block usually falls within the face shells only and thus the lack of grout in some of the cells does not influence the performance of

the wall. The cells containing steel would be grouted and thus, through the grout, the steel would be functioning. This is the mechanism by which even solid grouted walls function.

For partially grouted walls, the actual moment of inertia should be used for uncracked sections, while the transformed computed moment of inertia used for cracked sections can be used even for partially grouted walls.

The depth of the compression stress block should be calculated to determine that it is completely within the face shell of the masonry unit. If it isn't, computations can be made to evaluate the section based on T-beam configurations in which the face shell is the flange and the grouted cell is the stem.

3. Moment Determination. The analysis of the test panels properly considered that the walls were pinned at the top and the bottom. Therefore, the lateral moment on the wall was $WH^2/8$ and this became the basic design moment to which the Pe and $P\Delta$ effect were added. The designer may wish to modify these moments and deflections to take into account any conditions of restraint.

4. Precast vs. In-Place Construction. It is important to note that this test program covered both the precast elements (concrete) and the construction in-place elements (masonry walls), and they were all on continuous footing. Therefore, the scope of the program applies to the prefabricated and precast walls, and to the constructed in-place and cast-in-place walls. No differences were noted in the test performance characteristics between the precast concrete panels and the built in-place masonry panels.

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5. Openings in Walls. The walls tested were solid walls, and it is recognized that walls with openings and other configurations may introduce special problems. This publication and the research program did not cover these considerations, but it is suggested that the engineer and designer use basic principles for the analysis of walls with openings. It should be recognized that walls with openings will have a change of stiffness caused by the openings in the walls. The design of opening jambs deserves special consideration.

6. Isolated Footings. The test program was conducted so that all walls had continuous support. A technique of designing walls on isolated footings is given in Reference 30. Until further testing is done, this appears to be an acceptable approach for evaluating slenderness effects in the design of tall, slender walls on isolated footings. Special considerations should be given to the increased loading on the wall elements over the isolated pads.

10.3 CONTRIBUTORS

Numerous organizations and individuals provided materials, technical services, and money that made this project possible. Listed below are the organizations and individuals who provided this support.

Associated Concrete Products

Baldi Brothers Construction Co.
Bethlehem Steel Corporation
Wallace Bonsall
C.E. Buggy, Inc.

Cabot, Cabot & Forbes
California Field Iron Workers Trust Fund
California Gunite Co.
California Portland Cement Co.
Ted Christensen, S.E.
Concrete Coring Company
Concrete Masonry Association of California & Nevada
Conrock Company
Correia & Christian

Dominion Construction

O.K. Earl Corporation
Eide Industries
Emkay Development

Foster Sand & Gravel.

Hillman, Biddison & Loevenguth, S.E.
Higgins Brick Co.

Incentive Builders
Investment Building Group

Tom Kamei & Associates, S.E.
Kariotis & Associates, S.E.
Keller Construction Co.

Masonry Institute of America
McLean & Schultz, S.E.
Millie and Severson
Mr. Crane, Inc.
Mutual Building Materials

Osborne Laboratories

Charles Pankow Co.
Portland Cement Association

Ramtech Laboratories
Rockwin Corporation

Sanchez and Hernandez
S & H Steel
Smith-Emery Co.
Spancrete of California
Stolte, Inc.
Donald Strand, S.E.
Superior Concrete Accessories, Inc.
Syart Construction Co.
Synetic Designs Company

Thompson and LaBrie, S.E.
Transit Mix Concrete
Triangle Steel
Trus Joist Corporation
Twining Testing Laboratory

Davis Walker Corporation
Warner Company
Watson Land Development Co.
Paul Winter, S.E.
Wray Construction Co.
Wheeler & Gray, S.E.

10.4 DOCUMENTARY FILM

The Masonry Institute of America made a special contribution by producing a documentary film on the test project. MIA engaged White Productions to make a cinematic record of all phases of the project: committee meetings, laboratory testing, instrumentation, testing of panels, and descriptions of uses for masonry and concrete walls. The films have been reproduced so that many organizations throughout the United States can show this research program to thousands of architects, engineers, building officials and constructors across the country.

10.5 STAFF SUPPORT

The Task Committee is pleased to recognize the special dedication and support given the project by the staff members of the Masonry Institute of America, namely Juan Giron, staff engineer, for computer studies, drafting, designing, and other technical assistance, and Bella Sokoloff, for secretarial and editing support. They worked unstintingly for the project from its beginning to the final report production. Our success shows up in their work.

10.6 VOLUNTEERS

There was much discussion in the committee as to whether to engage a testing laboratory to conduct the total test under the supervision of the committee or to have the committee do the testing itself. Under the guidance of project director Ralph S. McLean, structural engineer, of the firm McLean & Schultz, the program was set up and the members of the committee volunteered to serve as his assistants to conduct the program. Jim Amrhein, member of the committee, was present at the testing of almost every wall. Robert G. Thomas of the Masonry Institute of America and Robert Tobin of Portland Cement Assoc. (and member of the committee) were at the test site as often as possible.

In addition to the basic cadre of committee members, volunteers from numerous offices assisted in the program. The volunteers were members of the Structural Engineers Assoc. of Southern California or the Southern California Chapter of American Concrete Institute or interested engineers and building officials who wished to participate in the program. The Masonry Institute of America scheduled these volunteers so that there would be approximately three at the test site for each test. Without these volunteers this program could not have come to fruition. They helped set up the test panels and equipment, conduct the tests, and read the many gages, dials, and controls.

Listed below is a partial list of the volunteers. We apologize to those whom we missed, but we have included all the names that we have available.

Acosta, Julio
Aleem, Syed
Allen, Jr., C.K.
Amanullah, Saeed
Austin, Mary

Baumann, Hanns
Bayer, Jack
Beer, Robert
Bliel, Richard
Choy, Phil

Corbett, John
Crask, Lloyd
Christensen, Ted
Czapski, Dan
Daniels, Robert

Dickey, Walter
DiMundo, George
Eggenberger, Byrne
Freyermouth, Ed
Gutierrez, Sr., Ramon

Halwani, Sammy
Harder, Ben
Hart, Stanley
Hatalsky, Pete
Hobbs, Leonard

Jasper, Richard
Jones, Mike
Juvier, Arizmendiz
Kamai, Tom
Kariotis, John

Kobbitz, Steve
Kobzeff, John
Korman, Ben
Lam, Howard
Lambe, Michael

Lamont, Wayne
Leuer, John
Lue, Raymond
Mann, Cliff
Martin, Melvin

Meeker, Edward
Meier, Mike
Milczewski, Juergen
Mills, Gary
Muneno, Don

Narver, David
Negen, Tom
Nieblas, Gerald
Ochoa, Ignacio
Osborne, Howard

Osborne, Raymond
Pach, Greg
Powers, J.C.
Pfefferle, Warren
Pullman, George

Ramos, Rick
Rao, J.K.
Reyes, Joel
Rez, Don
Richardson, R.C.

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Daniels, Robert

Dickey, Walter
DiMundo, George
Eggenberger, Byrne
Freyermouth, Ed
Gutierrez, Sr., Ramon

Halwani, Sammy
Harder, Ben
Hart, Stanley
Hatalsky, Pete
Hobbs, Leonard

Jasper, Richard
Jones, Mike
Juvier, Arizmendiz
Kamai, Tom
Kariotis, John

Kobbitz, Steve
Kobzeff, John
Korman, Ben
Lam, Howard
Lambe, Michael

Lamont, Wayne
Leuer, John
Lue, Raymond
Mann, Cliff
Martin, Melvin

Meeker, Edward
Meier, Mike
Milczewski, Juergen
Mills, Gary
Muneno, Don

Narver, David
Negen, Tom
Nieblas, Gerald
Ochoa, Ignacio
Osborne, Howard

Osborne, Raymond
Pach, Greg
Powers, J.C.
Pfefferle, Warren
Pullman, George

Ramos, Rick
Rao, J.K.
Reyes, Joel
Rez, Don
Richardson, R.C.

Sanders, Hank
Schneider, Robert
Scott, Jock
Seltzer, Warren
Shakeri, Sirous

Sinbel, Aly
Spencer, Allan
Stockinger, Herb
Stapenhill, Jim
Tandoc, Rosalinda

Taubman, David
Tawfik, Mohamed
Test, C. Taylor
Teal, Edward
Thomas, Robert G.

Thomsen, Steen
Ting, Raphael
Traw, Jon
Valenzuela, Louis
Warren, Jack

White, Dean
Wong, Ken
Woody, Hal
Wu, Jackson
Wyatt, Jesse

Yaguchi, John
Yee, Ben
Yeung, Herb

10.6 REFERENCES

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