POST-TENSIONED SINGLE WYTHE CONCRETE MASONRY WALLS

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ABSTRACT

This study investigates post-tensioning as a means of increasing the flexural strength of single wythe concrete masonry walls. Wall panels are constructed of conventional concrete masonry units and mortar, the properties of which are determined. The wall panels are post-tensioned and grouted solid. Three groups of wall panels are post-tensioned with a different amount of post-tensioning force. A reinforced masonry wall panel is also constructed. The panels are tested in flexure. Each post-tensioned panel is tested twice. Based on strain data obtained during the first test, the cracking moment is determined. In the second test, several panels are tested to failure. The flexural behavior of reinforced and post-tensioned masonry observed during testing is discussed. The deformation of the masonry caused by grouting, shrinkage and creep in two post-tensioned wall panels and one non-post-tensioned wall panel is monitored for a period of 90 days. The procedure used to design the post-tensioned wall panels is discussed. The construction of a post-tensioned wall in actual practice is examined. Recommendations are made for future research in the area of post-tensioned masonry.

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CHAPTER 1

INTRODUCTION

Masonry is one of the oldest and most extensively used construction materials, but its structural behavior is said to be the least understood. For centuries, masonry walls were recognized to be stable, provided they conformed to empirical rules pertaining to height, unsupported span, and thickness(21). While the use of empirical rules in the design of masonry structures proved to be adequate, limits were placed on the height of masonry structures due to the required massiveness of walls. Moreover, masonry structures designed in accordance with empirical "rules of thumb" did not perform well in situations where the structures were subjected to substantial lateral loads caused by seismic and wind forces. Procedures for the design of reinforced masonry have been developed and codified, and structures designed in accordance with these procedures have performed quite well(2). Design procedures for the rational design of plain masonry have enabled the construction of numerous load-bearing wall structures of up to 20 stories in height(16). The procedures used in the rational design of masonry, commonly called Engineered Masonry, are sound; however, they are not without shortcomings. For example, reinforced masonry design is quite conservative, and engineered plain masonry relies to a certain extent on axial loads to provide resistance to lateral loads. These short-comings yield a potentially inefficient design in situations

where a masonry wall is subjected to little or no axial load while at the same time being subjected to lateral and/or eccentric axial loads. Examples would be tall masonry curtain walls typically found in commercial structures, load-bearing walls in structures where a lightweight floor framing system is used, retaining walls, and prefabricated masonry wall panels. In the aforementioned situations, the application of an axial load would increase the ability of the masonry wall to resist lateral loads. Post-tensioning the masonry wall would fulfill this requirement, and is the focus of this study.

1.1 Historical Background

The concept of post-tensioning masonry dates back to 1825, when Brunel employed post-tensioned brickwork in the construction of air shafts for the Thames River Tunnel(16). The literature provides no further mention of post-tensioned masonry walls until 1970, when Hanlon reported the use of post-tensioned concrete masonry in the construction of several one-and-two story buildings and one six-story building in New Zealand(13). The one- and-two story buildings utilized walls constructed of 8" concrete masonry units with 3/8" diameter 7-wire strands, sheathed in plastic garden hose, in the cores. The cores containing the strands were filled with mortar as the wall was laid up.

In the case of the six-story building, a cavity wall was employed, and the sheathed strands were run free in the cavity and anchored atop the concrete roof slab. Anchorages

on the roof were protected with a bituminous coating, and left exposed in order to enable a second post-tensioning at the end of 18 months. Hanlon tested two wall panels under simulated earthquake loading, and determined that even after mortar joints cracked under extreme loading, it was practical to repair the joints and in doing so, restore the wall to its original strength.

In Great Britain, several structures utilizing post-tensioned masonry walls have been constructed. British experience has been primarily with "diaphram" walls, which are essentially cavity walls, and "fin" walls, which are walls containing deep pilasters and designed as "T" beams[8,9]. In the case of the diaphram wall, post-tensioning rods are placed in the cavity of the wall and anchored to the foundation. They are extended up through a concrete beam, which rests on top of the masonry wythes, to the roof anchorage. The cavity is typically ungrouted. In fin walls, the post-tensioning rod is placed between the masonry wythes of the fin, and is typically grouted after post-tensioning. Post-tensioning is accomplished through the use of a torque wrench[7,8].

While the prestressing of masonry structures in Great Britain does not appear to be widespread, Haseltine indicates that the latest draft of British Standard Code BS 5628:Part 2 addresses prestressed masonry; however, it states only that the general principles of prestressed concrete

design, supplemented as necessary for masonry, are to be utilized(14).

1.2 Purpose and Scope

This study is concerned with post-tensioned concrete masonry as a potential construction system. The purpose of the study is to examine the constructibility, behavior in flexure, and economy of post-tensioned concrete masonry walls with respect to conventional concrete masonry construction. Three broad areas are examined. The first is the feasibility of post-tensioning conventional concrete masonry walls using commonly available tools and materials. The second is the general behavior of post-tensioned concrete masonry in flexure with respect to both the design procedure and conventionally reinforced masonry. The third is an examination of post-tensioned concrete masonry as compared to reinforced concrete masonry.

In order to carry out the study, several wall panels were constructed using conventional concrete masonry units and mortar. A post-tensioning system using readily available tools and materials was devised, the panels were post-tensioned, and tested in flexure. Appropriate tests to determine the physical properties of the masonry components were performed.

The panels were constructed of 8" concrete masonry units and Type S mortar, materials typically used in load-bearing concrete masonry construction. Each panel consisted of eight concrete masonry units in a stack bond.

Panels were post-tensioned concentrically with a 5/8" diameter continuously threaded prestressing rod, commonly used as rock anchor, and were grouted solid following post-tensioning. Figure 1.1 depicts a typical post-tensioned wall panel. Deformation of the concrete masonry units under application of post-tensioning force was recorded, and lock-off losses were determined. Three groups of panels were post-tensioned, each with a different amount of post-tensioning force. One post-tensioned panel from each of two groups, and one non-post-tensioned panel were monitored for time-dependent deformation due to shrinkage and creep, which has a direct relationship to the loss of post-tensioning force. One wall panel, conventionally reinforced and grouted, was constructed. When the panels were tested in flexure, loads, corresponding tensile and compressive strains in the concrete masonry units, and deflections were recorded.

The actual behavior of the wall panels is compared to the behavior predicted by the design technique. In order to determine the practicality of post-tensioned masonry, construction details are discussed, and a comparison of post-tensioned versus conventionally reinforced concrete masonry is performed.

1.3 Limitations

There are several limitations to tnis study. The height to thickness ratio of the panels was such that slenderness is not a factor, whereas in actual construction

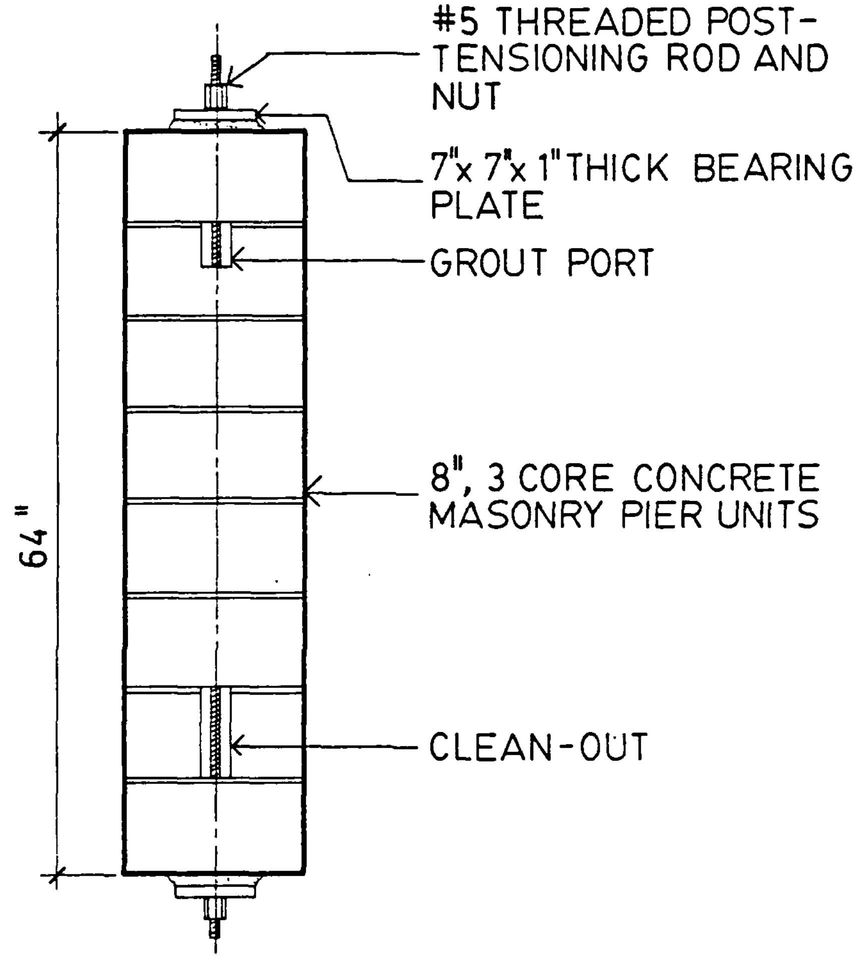


Figure 1.1. Typical Post-Tensioned Masonry Wall Panel.

practice it might be. The panels were constructed in a stack bond, and thus the effect of post-tensioning on a wall constructed with a running bond was not examined. The concrete masonry units were fully bedded in mortar, which would be difficult to achieve in a wall constructed with a running bond where typically only the face shells are bedded in mortar. Due to time constraints, shrinkage and creep were only monitored for a period of 90 days and thus provide only a partial indication of time-dependent prestress losses. Finally, the wall panels were allowed to cure for a period of 45 days prior to post-tensioning, a situation which would in all likelihood not be possible in actual construction practice. In light of these limitations, this study should be viewed as a feasibility study of the concept of post-tensioned masonry and a basis for further development.

CHAPTER 2

CODES AND GENERAL PRACTICE

2.1 Codes and General Practice

American building codes do not contain guidelines for the use of prestressed masonry. British Standard Code BS 5628:Part 2 permits the use of prestressed masonry but states only that the general principles of prestressed concrete are to be used, supplemented as necessary for masonry[14].

Masonry design is quite conservative, and in order to point out the potential benefits of post-tensioned masonry, Current American codes pertaining to the design of plain and reinforced masonry will be discussed. The BOCA Basic Building Code, the National Building Code, and the Standard Building Code all incorporate the following masonry design standards[4,17,23]:

The American Concrete Institute Building Code for Concrete Masonry Structures, ACI 531-79[5]

The American National Standards Institute Building Code Requirements for Masonry, ANJI A41.2[1]

The National Concrete Masonry Association Specification for the Design and Construction of Load-bearing Concrete Masonry, NCMA TR75-B-1970[22]

The Uniform Building Code[24] outlines specific requirements and procedures which are similar in scope to these standards, but does not refer specifically to them. All four of the codes mentioned incorporate American Society for Testing and Materials standards pertinent to masonry materials. There are essentially two design techniques

utilized in masonry construction. The first is the empirical technique, where wall thicknesses and height to thickness ratios are specified in the code. The second is based on rational analysis for both reinforced and plain masonry where the masonry is sized and/or reinforcement selected based on a structural analysis for the specific loads to be carried.

2.1.1 Reinforced Masonry

Reinforced hollow masonry is a construction system in which steel reinforcement is imbedded in grout within the concrete block such that the masonry, grout and steel act together to resist the applied forces. The hollow concrete units are laid up in mortar such that their alignment forms a series of continuous vertical cavities. Horizontal cavities, called bond beams, are created through the use of special concrete blocks. The vertical and horizontal cavities within the wall contain properly positioned steel reinforcement and are grouted solid, forming a bonded, composite structural system(21).

Allowable stresses and design formulas used in the design of reinforced masonry vary slightly among the previously mentioned standards, but all of the requirements are based on the same principles of design and analysis. The working-stress method for concrete design is utilized in all flexural computations.

The following is a discussion of the design of reinforced concrete masonry walls under combined axial and

lateral loading using design criteria specified by the Uniform Building Code[24]. The allowable and compressive stress is a function of the ultimate compressive strength of the masonry and the unsupported height and thickness of the wall. This relationship is given by the equation:

$$F_a = 0.2\, f'_m \left[1 - \left(\frac{h}{40t}\right)^3\right] \tag{2.1}$$

where: F_a = allowable unit axial stress, psi

f'_m = ultimate compressive stress of masonry, psi

h = unsupported wall height, in.

t = nominal wall thickness, in.

The allowable flexural compressive stress, F_b, is:

$$F_b = .33\, f'_m \text{ (900 psi maximum)} \tag{2.2}$$

Stresses generated through combined axial loads and flexure must be such that the following interaction equation is satisfied:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1 \tag{2.3}$$

where

f_b = actual flexural compressive stress

f_a = actual axial compressive stress

Schneider and Dickey[21] suggest that this procedure is quite conservative since the stress-reduction component of equation (2.1) is somewhat of a holdover from the days of empirical design. They point out that the code does permit the h/t ratio to be increased and the wall thickness to be

decreased when supporting calculations are provided. They also point out that the interaction formula given by equation (2.3) is conservative since it does not account for the influence of axial compressive force in reducing tensile stresses developed as the result of lateral loads.

The conservative nature of equation (2.3) can be illustrated by comparing an interaction diagram based on an elastic column analysis with an interaction diagram based on equation (2.3). Figure 2.1 presents such a comparison schematically. For purposes of this discussion, slenderness is not considered a factor, and the code stipulated values of 300 psi for axial compression, F_a, and 500 psi for flexural compression, F_b are used. At point 1 in Figure 2.1, the masonry is under axial load only, and is stressed to 300 psi, the allowable axial compressive stress. The allowable flexural compressive stress, however, is 500 psi, and thus the masonry at this axial load is capable of resisting a moment which would cause an increase in the maximum compressive stress of 200 psi. As moment is applied while load is held constant between points 1 and 2, the stress at the compression face of the wall increases, and the compressive stress at the tension face is decreased, until the maximum compressive stress of 500 psi is reached at point 2. The conservative nature of equation 2.3 is readily apparent. It is also apparent that an increase in axial load up to the balance point enables the wall to resist a greater moment.

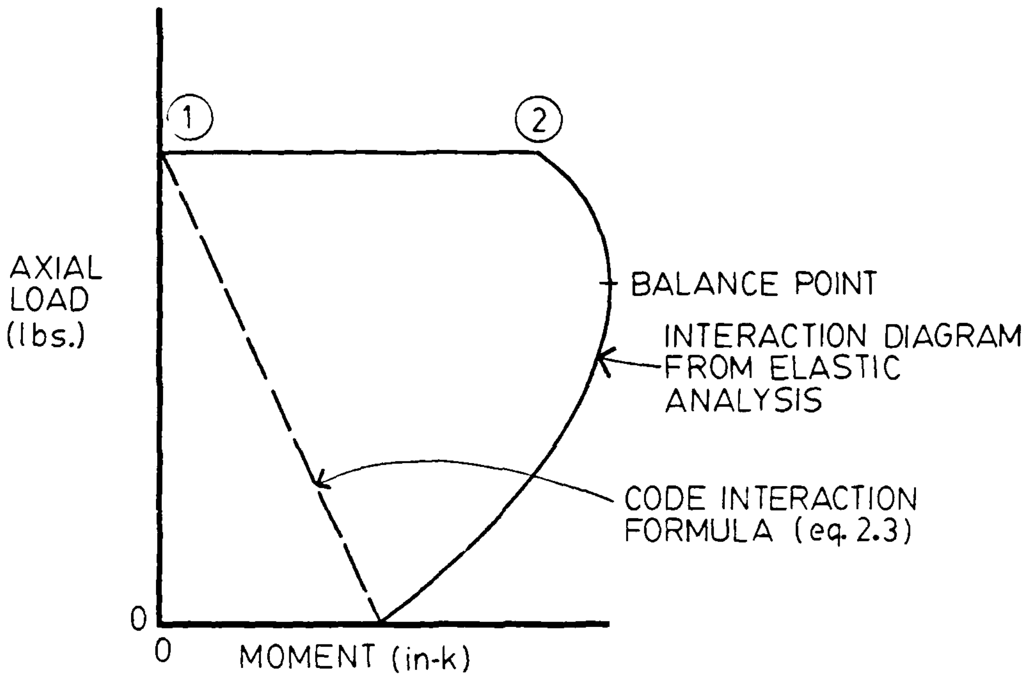


Figure 2.1. Reinforced Masonry Interaction Diagram.

2.1.2 Plain Masonry

The engineered design of plain masonry acknowledges the fact that masonry has some tensile strength and that the flexural strength increases with the amount of axial load present[6]. This concept was validated by Yokel, Mathey and Dikkers[26] in an extensive series of tests performed for the Department of Commerce. The Specification for the Design and Construction of Load-Bearing Concrete Masonry, NCMA TR 75-B-1970, will be used as the basis for this discussion. The allowable axial compressive stress is computed in accordance with equation (2.1), and the allowable tensile stresses permitted are based on the type of concrete masonry units used (hollow or solid), the type of mortar (Type M or S, Type N) and the direction of stress, (parallel to bed joints or perpendicular to bed joints). For concrete masonry built with Type M or S mortar and grouted solid, the allowable tensile stress perpendicular to the bed joints is limited to 39 psi[22].

The theory behind the engineered design of plain masonry walls subjected to combined loading is as follows. The axial load, P, is distributed over the area of the wall, A, and produces compressive stress, f, in the masonry. Lateral loads impose flexural compressive and tensile stresses in the wall in addition to the compressive stresses caused by the axial load. Flexural stresses are computed from the equation $f = P/A \pm M/S$, where f is the flexural compressive or tensile stress, M is the moment due to

lateral loads, and S is the section modulus of the wall. The compressive stress in the wall, f_c, due to combined loading is the sum of the axial and flexural compressive stresses, $f_c = (P/A) + (M/S)$, and is limited by the interaction formula, equation (2.3). The tensile stress due to combined loading, f_t, is the difference b tween the compressive stress due to axial load and the computed tensile stress due to application of lateral load, $f_t = (P/A) - (M/S)$. Tensile stress is limited by code stipulated values, as mentioned previously. A schematic interaction diagram for engineered plain masonry is shown in Figure 2.2.

2.2 Post-Tensioning

From observation of Figures 2.1 and 2.2, it is apparent that the capacity of a wall to resist moments increases as the axial load on the wall is increased up to the balance point. In certain situations, the axial load necessary to increase moment capacity is not present, and the use of thicker walls, pilasters, or increased steel reinforcement, all of which potentially add to the cost construction, is called for.

Another technique by which the moment capacity of a wall could be increased is by post-tensioning the wall. When a wall is post-tensioned, the masonry is placed under axial compression such that the compressive stress resulting from post-tensioning is sufficient to overcome anticipated tensile stresses caused by the application of a lateral

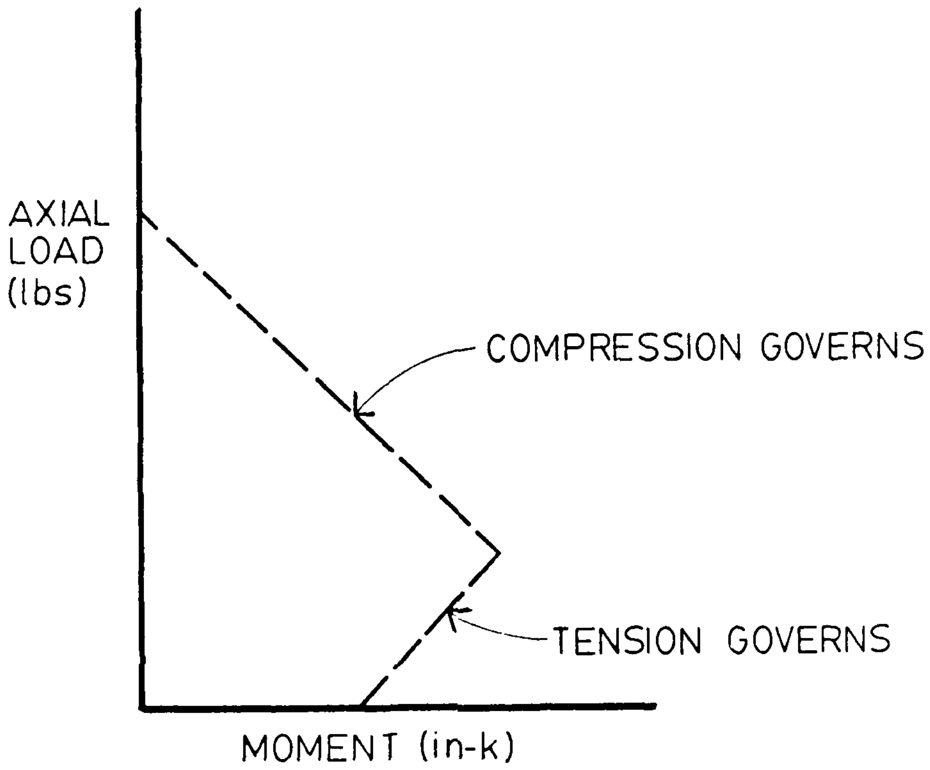


Figure 2.2. Plain Masonry Interaction Diagram.

load[15]. Compressive stresses in a single wythe concrete masonry wall, post-tensioned concentrically, are identical to compressive stresses in a single wythe concrete masonry wall supporting a concentrically applied axial load. Stresses resulting from the application of a post-tensioning forceand lateral loads can be determined in the same manner as the stresses for engineered plain masonry subjected to axial and lateral load, that is $f = (P/A) \pm (M/S)$, the only difference being that P in this case is the post-tensioning force. Figure 2.3 shows the theoretical stress distribution in a post-tensioned member.

There is, however, a difference in the behavior of the walls. Axially loaded conventional masonry walls are subject to column action, where any eccentricities in the application of the axial load cause the wall to deflect. Increases in load cause increases in deflection, eventually causing the wall to buckle. The code accounts for the effect of buckling through the stress reduction factor $1 - (\frac{h}{40t})^3$ in equation (2.1), and by limiting the height to thickness ratios of masonry walls.

In a tall, slender masonry wall, where the post-tensioning rod is held in position only at the top and bottom of the wall, the application of a post-tensioning force could cause the wall to buckle. Libby[15] points out that if the post-tensioning rod is in contact with the member at points between the ends of the member, the tendency to buckle is reduced to a significant degree, and

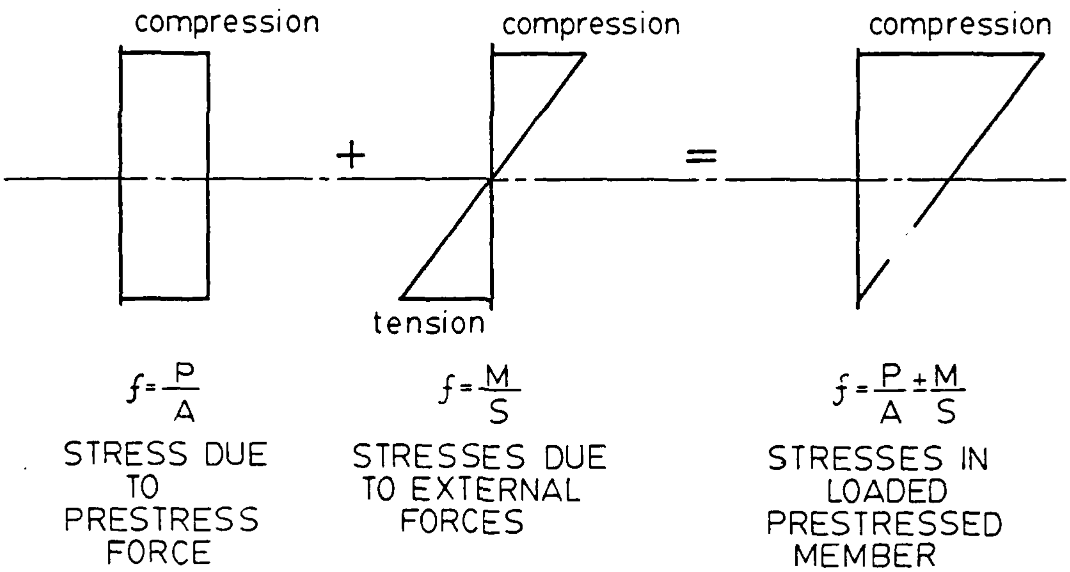


Figure 2.3. Theoretical Stress Distribution in a Post-Tensioned Member.

if the post-tensioning rod is completely in contact with the member throughout its entire length, there is no possibility of buckling. The design of a post-tensioned wall should therefore, include provisions for placing the post-tensioning rod into contact with the wall at either points between the top and bottom of the wall or continuously throughout the length of the wall.

Post-tensioned masonry and conventional masonry also differ in that the effects of time-dependent deformation due to the shrinkage and creep of masonry are much more important in a post-tensioned wall than in a conventional wall. Shrinkage of concrete masonry is a change in dimension from a moist condition to a dry condition. The Standard Specification for Hollow Load-Bearing Concrete Masonry Units, ASTM C90-75 establishes limits for linear shrinkage of concrete masonry units, which range from 0.03% to 0.065%, based on the quality of the unit and the humidity conditions at the location of use. Creep on the other hand is much more difficult to determine. Very little is known about creep in concrete masonry and test data vary widely. Sahlin(20) indicates that, based on tests performed by Nylander and Ericson, the maximum deformation due to creep approaches three times the instantaneous deformation. British Standard Code BS 5628: Part 2(14) indicates that in the absence of better information, a numerical value of two times the elastic deformation of the masonry under the prestress force shall be used. Curtin, Shaw, Beck, and

Bray[7] indicate that in their experience the effects of shrinkage and creep in a post-tensioned wall can be compensated for by an increase in the effective post-tensioning force of 20%. While the magnitude of time dependent losses may be difficult to determine accurately, the fact is that concrete masonry under compressive stress will shorten with time, and this decrease in length is accompanied by a loss of post-tensioning force. If post-tensioned masonry is to be used as a construction system, the phenomenon of time-dependent losses must be evaluated.

CHAPTER 3

MATERIALS

3.1 Concrete Masonry Units

The concrete masonry units selected for use in this study were load-bearing, three-core pier units, with nominal dimensions of 8" x 8" x 16". The units were manufactured by E. DeVecchis & Sons, Inc., of State College, Pennsylvania. They were manufactured of normal weight concrete with a crushed limestone aggregate and according to the manufacturer, conform to the Standard Specifications for Hollow Load-Bearing Concrete Masonry Units, ASTM C90-75. Figure 3.1 shows a typical three-core pier unit used for the panels.

Three units were selected from the full lot of units used in the construction of the panels as representative of units from the full lot. These units were used for testing in accordance with the Standard Methods of Sampling and Testing Concrete Masonry Units, ASTM C140-75 (1980). The purpose of the tests performed under ASTM C140-75 (1980) is to determine if the physical characteristics pertaining to dimensions, absorption, moisture content and compressive strength are in conformance with the Standard Specifications for Hollow Load-Bearing Concrete Masonry Units, ASTM C90-75. In order to determine these physical properties, it is necessary to determine other physical properties, specifically, unit weight, area, volume and density. The following is a discussion of the tests of the three sample

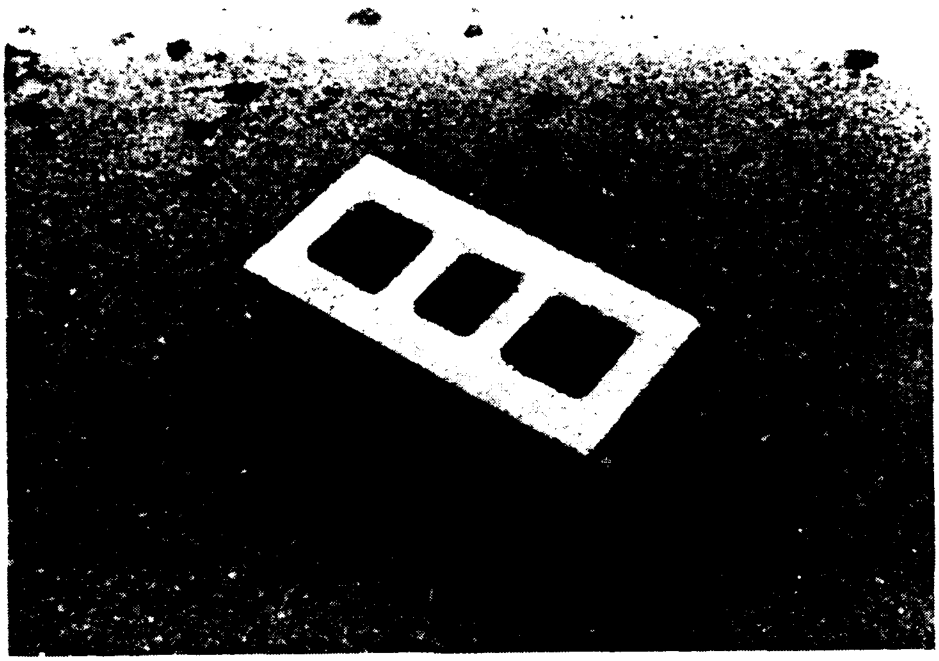


Figure 3.1. Three Core Concrete Masonry Pier Unit.

concrete masonry units performed in accordance with ASTM C140-75 (1980).

3.1.1 Measurements and Dimensions

Average measurements are used to determine exterior dimensions of each unit. Length is measured along the longitudinal centerline of each face, width across the top and bottom bearing surfaces at midlength, and height on both faces at midlength. Face shell and web thicknesses are measured at the thinnest point of each element, 1/2" above the mortar bed plane. Equivalent web thickness (in inches per linear foot of specimen) is determined by multiplying the sum of the measured thicknesses of all webs in the unit by 12 and dividing by the length of the unit.

Table 3.1 presents the average length (L), width (W), height (H), minimum face shell thickness (FST), minimum web thickness (WT) for end webs and interior webs, and equivalent web thickness of each sample unit. The average for all three samples is also shown. ASTM C90-75 permits a maximum variation of 0.125 inches from the standard overall dimensions of 15.625 inches (length), 7.625 inches (width), and 7.625 inches (height). The specification allows a minimum face shell thickness of 1.25 inches, a minimum web thickness of 1.0 inches, and a minimum equivalent web thickness of 2.25 inches per linear foot. All of the units tested met this portion of the specification.

Table 3.1. Dimensions of Concrete Masonry Units.

Unit	Avg. Length L(in.)	Avg. Width W(in.)	Avg. Height H(in.)	Avg. Min. Face Shell Thickness FST(in.)	Avg. Minimum Web Thickness WT.(in.) End Web	Avg. Minimum Web Thickness WT.(in.) Interior Web	Equivalent Web Thickness (in. per foot)
1	15.594	7.625	7.594	1.578	1.313	1.063	3.564
2	15.594	7.594	7.594	1.297	1.313	1.063	3.564
3	15.563	7.625	7.594	1.328	1.313	1.000	3.470
Avg. for three units	15.584	7.615	7.594	1.395	1.313	1.041	3.533

3.1.2 Absorption

ASTM C140-75 (1980) defines absorption of concrete masonry as the amount of water absorbed by the concrete masonry units after immersion in water at approximately 70° for a period of 24 hours. Absorption affects the workability of the mortar. If a masonry unit absorbs water from the mortar too quickly, the mason will not have enough time to set and adjust the block before the mortar stiffens, and a strong mortar to block bond will not be achieved. ASTM C90-75 therefore limits absorption to a maximum of 13 lb./ft.^{3}. Absorption of the three sample units, as shown in Table 3.2, varied from 9.3 lb./ft.^{3} to 10.14 lb./ft/^{3}, below the maximum absorption of 13 lb./ft.^{3} specified.

3.1.3 Unit Weight and Moisture Content

Table 3.3 shows unit weight and moisture content for the three sample units. ASTM C90-75 does not establish standards for unit weights, but does set limits on moisture content. The moisture content requirements are intended to indicate whether a unit is sufficiently dry for use in wall construction. Concrete shrinks slightly with the loss of moisture down to an air dry condition. If excessively moist units are placed in a wall, cracks will develop as the concrete shrinks. The average moisture content of the samples, 27.5% of the total absorption, is below the maximum of 30% of total absorption allowed by the specification.

Table 3.2. Absorption of Concrete Masonry Units.

Unit	A Wet Weight (lbs.)	B Dry Weight (lbs.)	C Suspended Immersed Weight (lbs.)	D Absorption (lb./ft.^{3})	Absorption (%)
1	43.26	40.33	24.47	9.73	7.26
2	43.20	40.17	24.56	10.14	7.54
3	43.00	40.25	24.10	9.30	7.03
Avg. for three units	43.18	40.25	24.38	9.72	7.28

where,

Absorption, lb./ft.^{3} = [(A-B)/(A-C)] x 62.4

Absorption, % = [(A-B)/B] x 100

Table 3.3. Unit Weight and Moisture Content.

Unit	A As Sampled Weight (lbs.)	B Dry Weight (lbs.)	C Wet Weight (lbs.)	Moisture Content &
1	41.11	40.33	43..26	26.6
2	41.08	40.17	43.20	30.0
3	40.98	40.25	43.08	25.8
Avg. for three units	41.06	40.25	43.18	27.5

where,
Moisture Content, % = [(A-B)/(C-B)] x 100

3.1.4 Area, Volume and Density

The average gross and net area, volume, and density for the three sample concrete masonry units are reported in Table 3.4. ASTM C90-75 does not establish standards for these properties. The average net area for the three samples is 68.46 in.^{2}.

3.1.5 Compressive Strength of Concrete Masonry Units

The three sample concrete masonry units were utilized for the compression test described in ASTM C140-75 (1980). Each unit was capped with gypsum capping plaster, which was allowed to cure for 24 hours. Machine surfaced steel plates, 1" thick, were utilized as bearing blocks. Each sample was placed in the testing machine and positioned approximately concentrically.

The size and weight of the bearing blocks and the sample made exact positioning difficult; however, measurements indicated that the load was applied within 1/4" of the centroid of the concrete masonry unit. ASTM C140-75 (1980) specifies that up to one-half of the anticipated maximum load shall be applied at any convenient rate, after which the rate of loading shall be adjusted such that the remaining load is applied in not less than one minute, nor more than two minutes. Ultimate loads were higher than anticipated, and thus, an average of approximately five minutes was utilized for each test. The mode of failure in each case was a shear cone resulting in a more or less rectangular shaped "hourglass" section. Figure 3.2 depicts

Table 3.4. Area, Volume and Density of Concrete Masonry Units.

Unit	A Net Volume ($ft.^2$)	B Gross Volume ($ft.^3$)	C Dry Weight of Unit (lb.)	D Density ($lb./ft.^3$)
1	.299	.523	40.33	134.5
2	.300	.520	40.17	133.9
3	.304	.522	40.25	132.3
Avg. for three units	.301	.522	40.25	133.6

Unit	E Wet Weight of Unit (lb.)	F Immersed Weight of Unit lb.	G Avg. Net Area (%)	H Avg. Net Area (in.2)
1	43.26	24.47	57.17	67.97
2	43.2	24.56	57.69	68.32
3	43.08	24.1	58.23	69.10
Avg. for three units	43.18	24.38	57.70	68.46

where,

Net Volume, (A) = C/D

Gross Volume, (B) = (L x W x H)/1728

Density, (D) = [C/(E-F)] x 62.4

Average Net Area, % (G) = (A/B) x 100

Average Net Area, $in.^2$ (H) = G x L x W

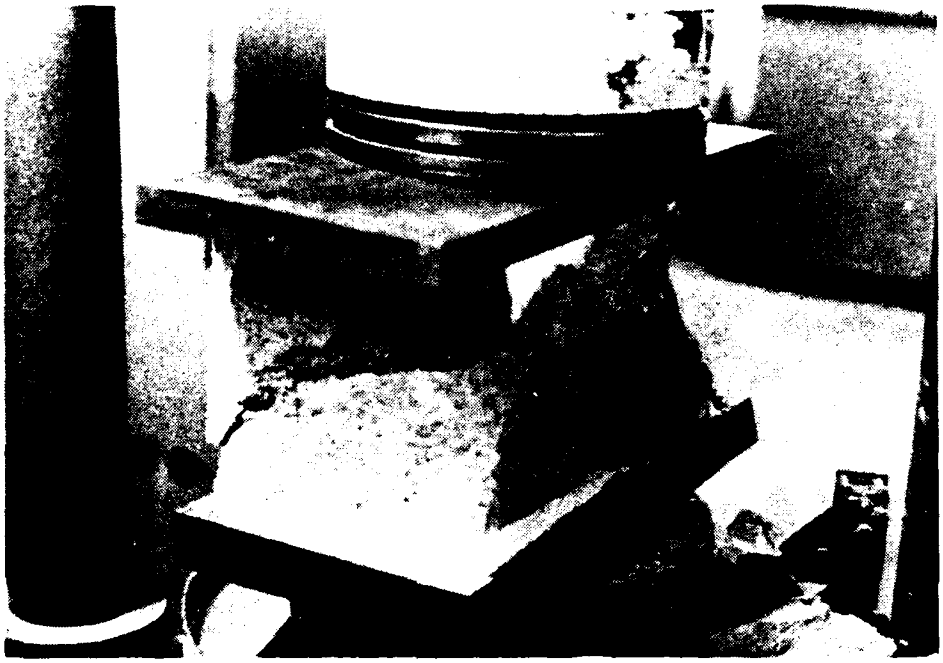


Figure 3.2. Compression Failure of Concrete Masonry Unit.

Table 3.5. Compressive Strength of Concrete Masonry Units.

Unit	Ultimate Load (lbs.)	Gross Area Area (in.2)	Gross Area Ultimate Strength (psi)	Net Area Area (in.2)	Net Area Ultimate Strength (psi)
1	215,000	118.90	1808	67.97	3163
2	205,000	118.42	1731	68.32	3001
3	237,000	118.66	2001	69.10	3437
Avg. for three units	219,167	118.66	1847	68.46	3201

a typical failure. ASTM C90-75 specifies a minimum average compressive strength of 1000 psi based on the gross area of the unit. The average compressive strength of the units tested, as shown in Table 3.5., was 1847 psi based on the gross area, and 3201 psi based on the net area.

3.2 Mortar

Mortar was mixed according to the Standard Specification for Mortar for Unit Masonry, ASTM C270-80a. A portland cement-lime mix was utilized for Type S mortar. ASTM C270-80a permits the use of masonry cement; however, since masonry cements are proprietary mixes of lime, cement, air-entraining agents and other additives, usually in unspecified proportions, masonry cement was not used. The Portland cement-lime mix enabled quantities of components to be accurately measured, thus allowing reproducibility of the physical characteristics of the mortar.

The Portland cement used wa~ Type I, conforming to the Standard Specification for Portland Cement, ASTM C150-80. The lime was a Type N hydrated lime conforming to the Standard Specification for Hydrated Lime for Masonry Purposes, ASTM C207-79. The sand used in both mortar and grout was a commercially available, manufactured, white silica sand. The results of three sieve analyses of the sand are presented in Table 3.6. The first sieve analysis was on a sample taken from the material brought to the job site, and was performed after the panels had been constructed. The sand did not conform to the Standard

Table 3.6. Sieve Analysis of Sand.

Sieve Size	SAMPLE 1 (as built) Cum. % retained	Cum. % passing	SAMPLE 2 (stockpile) Cum. % retained	Cum. % passing	SAMPLE 3 (stockpile) Cum. % retained	Cum. % passing	ASTM C144-80 % passing
4	0	100	0	100	0	100	100
8	1	99	1	99	1	99	99-100
16	2	98	3	97	3	97	70-100
30	5	95	7	93	6	94	40-75
50	49	51	60	40	54	46	20-40
100	94	6	96	4	95	5	10-25
200	98	2	99	1	98	2	0-10

Specification for Aggregate for Masonry Mortar, ASTM C144-81, with excessive amounts passing the number 30 and 50 sieves, and an insufficient amount passing the number 100 sieve. The following day, two additional samples were taken from the sand stockpile in the vendor's yard. Tests on these samples yielded similar results. Generally speaking, mortar made of finer sand tends to be more workable, but has greater porosity and is weaker[19]. This did not prove to be a problem in this study.

The proportions, by volume, of the dry materials utilized for mortar and grout are presented in Table 3.7. They conform to ASTM C270-80a for the mortar proportions and to the Standard Specification for Grout for Reinforced and Non-reinforced Masonry, ASTM C476-80 for grout proportions. Table 3.8 converts these volume proportions to equivalent weights, in pounds.

Mortar was batched by hand in a steel mortar box. A wood box with a 1/2 cubic foot volume, graduated at 1/8 cubic foot and 1/4 cubic foot increments, was used to measure the dry materials. Four batches of mortar were produced, each containing 1 1/2 cubic feet of dry materials. Potable water was added to the dry mixes until the mason felt that the mortar was of a suitable consistency. The mason's judgment was the sole determinant of mortar workability. Table 3.9 indicates the quantity of water used in each batch of mortar.

Table 3.7. Proportion for Mortar and Grout by Volume.

Type	Portland Cement	Hydrated Lime	Sand
Type S mortar	1	1/2	4 1/2
Fine aggregate grout	1	0	3

Table. 3.8. Proportions for Mortar and Grout by Weight Per Cubic Foot.

Type	Portland Cement	Hyerated Lime	Sand
Type S Mortar	15.67	3.34	60.00
Fine aggregate grout	23.50	0	60.00

The weights per cubic foot of the component materials as provided in ASTM C270-80a are as follows:

Material	Weight, lb./ft.^{3}
Portland Cement	94
Hydrated Lime	40
Sand	80

Table 3.9. Quantity of Water in Mortar.

		Water per ft.^{3} of dry materials	
qts.	lb.	qts.	lb.
7.5	15.58	5	10.39
7.5	15.58	5	10.39
7	14.54	4.67	9.69
7	14.54	4.67	9.69

Three mortar cubes were made from each batch of mortar. Cubes were made in accordance with the Standard Method for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry, ASTM C780-80, and the Standard Test Method for Compressive Strength of Hydraulic Cement Mortars, ASTM C109-80. Samples of mortar were taken approximately 15 minutes after the mortar batch was mixed. Cubes were molded in metal gang molds which were then placed in airtight plastic bags and left undisturbed at the job site for a period of 24 hours. The molds were then removed from the plastic bags and placed in a moisture room for another 24 hours. At the end of this period, the cubes were removed from the molds, marked for identification, and returned to the moisture room. Total moist curing time was 28 days.

At the end of the curing period, the cubes were tested for compressive strength. The results of the test for each cube and the average strength for each mortar batch are shown in Table 3.10. None of the mortar cubes tested reached the compressive strength of 1800 psi specified in ASTM C270-80. Two problems encountered during the mixing of the mortar may account for the discrepancy. First, the moisture from the sand migrated into the wood measuring box, causing the cement and lime to adhere to the walls of the measuring box, thus obscuring the graduations. Lime in particular was difficult to measure accurately. The amount of lime present in the mortar has a direct bearing on the

Table 3.10. Compressive Strength of Mortar Cube Specimens.

Batch	Compressive Strength of Individual Cubes, (psi)	Average Strength of Mortar For each Batch (psi)
1	1371	
	1420	1407
	1432	
2	964	
	1004	1003
	1042	
3	1305	
	1078	1180
	1159	
4	1432	
	1367	1444
	1455	
Average for four batches		1259

compressive strength of the water; a relatively small increase in lime content can significantly lower the strength of the mortar[3,19,20,21]. This effect would be quite significant given the small quantity of mortar in each batch. The second problem pertains to the water added to batch 2. Less water is required for a "wet box" (a mortar box which has already had mortar mixed in it) then a "dry box." Due to the inexperience of the mason tender, the same amount of water was added to batches 1 and 2. The mortar from batch 1 was considered "a bit dry" by the mason, and the tender felt that the same quantity of water would produce a more workable mortar. Initially the mason judged the mortar usable; however, after constructing two wall panels, the mason felt that the mortar was too "wet," and the remaining mortar from this batch was discarded. The low compressive strength of the mortar did not appear to affect the flexural strength of the wall panels.

3.3 Masonry Prisms

Concrete masonry prisms were constructed in accordance with the Standard Test Methods for Compressive Strength of Masonry Prisms, ASTM E447-74. Method B, which is used to determine the compressive strength of masonry built at the job site with the same materials and workmanship present in the structure, was followed.

Three prisms were constructed, utilizing mortar from batches 2, 3 and 4. Each prism consisted of two concrete masonry units in stack bond, with a mortar joint identical

to that present in the wall panels. Prisms were left at the job site and allowed to cure for 28 days. The capping procedure utilized gypsum plaster, and was modified due to the weight, bulk and fragility of the prisms. Capping was accomplished by pouring the capping plaster on the prism and setting the plate glass on the plaster. Acceptable capping was achieved.

In addition to testing for compressive strength, deformation under load was measured. Steel angles were fastened to both face shells of each concrete masonry unit with an epoxy adhesive. This provided an 8" gauge length across the mortar joint. Dial indicators with a least reading of 1×10^{-4} in. were mounted to the angles. Since the mortar and concrete masonry units have different strength characteristics, this method of measurement was considered to provide an accurate indication of masonry deformation. Figure 3.3 depicts the manner in which deformation was measured.

Prisms were tested at the end of the 28-day cure period. Prisms were placed in the testing machine and the dial indicators mounted. ASTM E447-74 specifies that up to one half of the ultimate load may be applied at any rate and that the remainder of the load be applied in not less than one nor more than two minutes. Strength of the prisms was higher than anticipated, and the time criterion could not be met. As was the case with the compressive tests of the concrete masonry units, the average time to test each prism

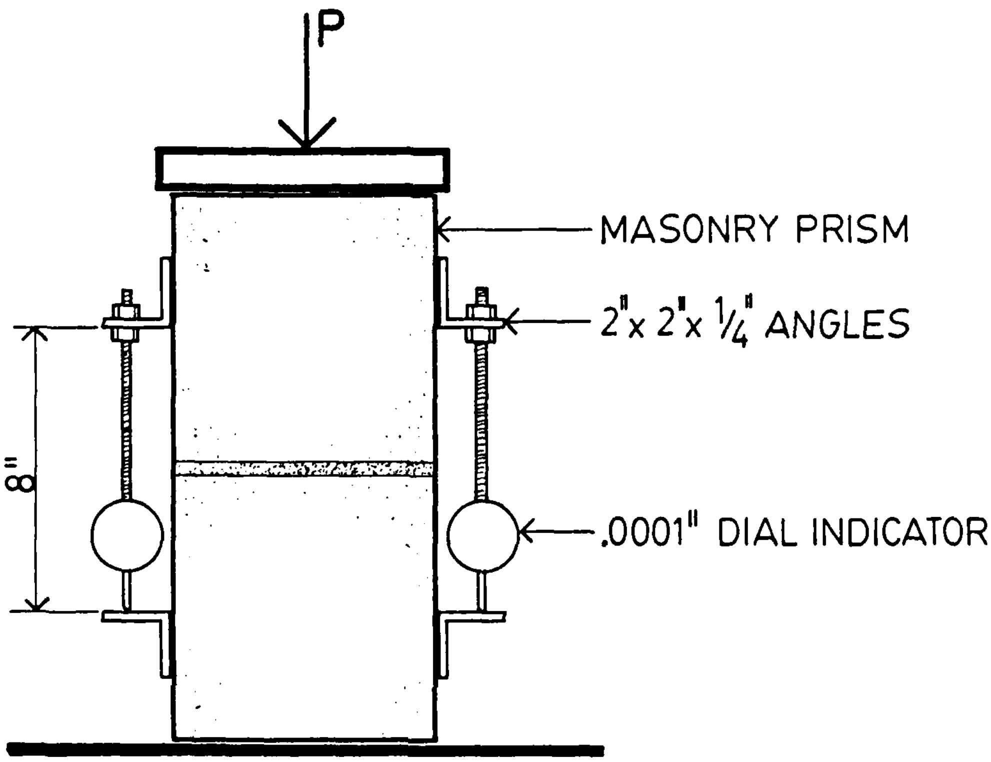


Figure 3.3. Method Used to Measure Deformation of Masonry Prism Tested in Compression.

was approximately five minutes. The results of the prism compressive strength tests are reported in Table 3.11, and a typical prism failure is shown in Figure 3.4. The average compressive strength for the three prisms tested was 2829 psi based on the net area, and 1636 psi based on the gross area. The assumed value for the ultimate strength of hollow unit masonry in the Uniform Building Code is 1350 psi, based on the net area[24]. A comparison of prism strength shown in Table 3.11 with concrete masonry unit strength shown in Table 3.5 and the strength of mortar cubes as shown in Table 3.10 does not indicate a strong relationship between variations in mortar cube strength and masonry strength. Based on net area, the average masonry prism strength of 2829 psi was 2.25 times the average mortar cube strength of 1259 psi. Citing data from tests performed on brick masonry, Sahlin[20] states that there is a direct correlation between masonry strength and mortar strength. The data cited, however, indicates that the effect is more pronounced at mortar strengths below approximately 750 psi where the lime content of the mortar is relatively high. In the 1000-2500 psi range the effect tends to be very slight.

The National Concrete Masonr Association indicates that when full mortar bedding is used, as was the case with the prisms tested, prism strength for hollow concrete masonry units should be within 80%-90% of the concrete masonry unit strength[18]. The average strength of the three prisms, 2829 psi, was 88% of the average concrete

Table 3.11. Compressive Strength of Concrete Masonry Prisms.

Prism	Mortar Batch	Ultimate Load (lbs.)	Net Area Masonry Strength f'm (psi)	Gross Area Masonry Strength f'm(psi)
1	2	175,000	2556	1478
2	3	206,000	3009	1740
3	4	200,00	2921	1689
Avg. of three prisms		193,667	2829	1636

Average net area, $in.^2$ = 68.46
Average gross area, $in.^2$ = 118.40

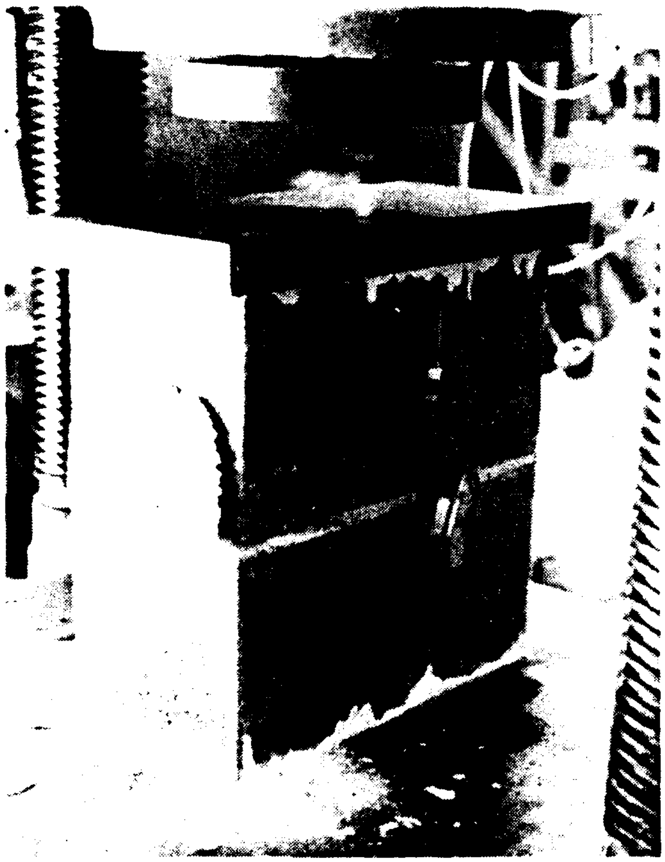


Figure 3.4. Compression Failure of Concrete Masonry Prism.

masonry unit strength of 3201 psi. Beal[3] suggests that mortar strength has little effect on prism strength, and indicates that an increase in mortar strength of 130% results in an increase in prism strength of only 10%. The data obtained from the three prisms tested support these statements. A possible reason for this is that the failure mode of the mortar cubes and mortar joints is different. The mortar cube failure is essentially a shear failure resulting in an "hourglass" fracture[10], while the mortar joint failure is one of crushing. At higher stress levels, the mortar in the joint begins to break down. Because the mortar is confined by the concrete masonry units, as it breaks down it becomes compacted, thus permitting stresses to be increased[20].

3.3.1 Modulus of Elasticity

Load and displacement data obtained from the prism tests were converted to unit stress and unit strain, and plotted against one another. Figure 3.5 shows the stress-strain curve for the three prisms tested.

The modulus of elasticity, E_m specified by the Uniform Building Code is 1000 f'_m, with a maximum value of 3,000,000 psi[24]. The National Concrete Masonry Association and American Concrete Institute also specify a value of 1000 f'_m for E_m, but limit the maximum to 2,500,000 psi[22,5]. Figure 3.6 depicts a fairly linear stress-strain relationship to approximately 750 psi for prism 1 and approximately 1000 psi for prisms 2 and 3. A comparison of

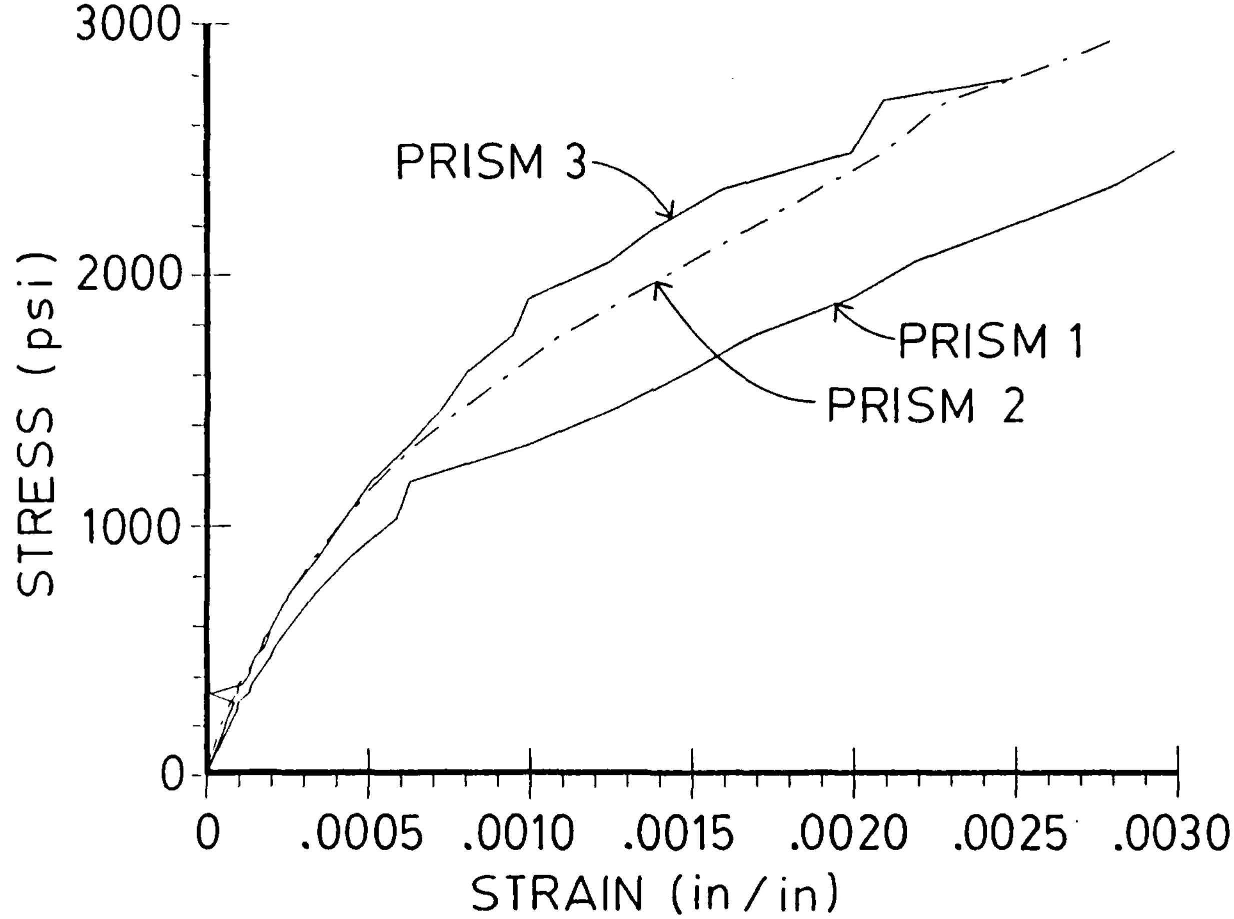


Figure 3.5. Stress-Strain Curves for Masonry Prisms.

the Modulus of Elasticity computed from code criteria and from test data at the maximum flexural compressive stresses permitted by the codes is presented in Table 3.12. Inspection of Table 3.12 indicates that the code specified values of E_m yield slightly conservative values at the maximum flexural compressive stress values allowed by the codes.

3.4 Grout

The fine aggregate grout was batched by hand in a steel mortar box. Water was added until the grout reached a fluid consistency and yielded a slump of 10", as measured with a standard slump cone using the procedure described in the Standard Test Method for Slump of Portland Cement Concrete ASTM C143. The Standard Specification for Grout for Reinforced and Non-Reinforced Masonry, ASTM C476-80, does not provide a range of acceptable slump. The National Concrete Masonry Institute recommends a slump range of 8" for units with low absorption to 10" for units with high absorption(12). The grout was required to have a fluid consistency such that it could be poured through 1" diameter holes in the top of each specimen, and the 10" slump provided the required consistency. The amount of water required to achieve the 10" slump varied with the "wetness" of the mortar box, and ranged from 8 quarts per cubic foot of dry materials to 6.5 quarts per cubic foot of dry materials.

Table 3.12. Comparison of Code Specified Values of Modulus of Elasticity, E_m, with Modulus of Elasticity Values Computed From Test Data.

Sample	Value of E_m computed from Test Data		Code Specified Value of Em (psi)
	900 psi	1200 psi	
1	1,903,000	1,692,000	1,750,000
2	2,536,000	2,182,000	2,060,000
3	2,478,000	2,265,000	2,000,000
Average of three values	2,300,000	2,046,000	1,937,000

where,

Modulus of Elasticity, E_m (psi) computed from test data = stress (psi)/strain (in./in.)

Modulus of Elasticity, E_m, (psi) computed from code criteria = 1000 f'_m

900 psi = maximum flexural compressive stress permitted by *Uniform Building Code and National Concrete Masonry* Association (e,f)

1200 psi = maximum flexural compressive stress permitted by American Concrete Institute 531-79

The ASTM specifications do not provide a method for the field sampling of grout. The National Concrete Masonry Association[12] suggests making a mold by intersecting concrete masonry units to form a 3" x 3" x 6" mold which is lined with absorptive paper to provide a bond break. Figure 3.6 depicts such a mold.

Grouting was performed on two successive days and two grout samples were taken from each days production. Samples were left in the molds, at the job site, for a curing period of seven days. Samples were then removed from the molds, marked for identification, measured to obtain an average cross-sectional area, and capped with gypsum capping plaster. Since the intent was to determine the strength of the grout at the time the wall panels were tested in flexure, the grout was tested when flexural testing of the wall panels was approximately 50% complete. The age of the grout at this point was either eight or nine days, depending on the day the panels were grouted. The average strength of the grout, as reported in Table 3.13, was 2627 psi.

The Standard Specification for Grout for Reinforced and Non-Reinforced Masonry, ASTM C476-80, does not specify a minimum compressive strength for grout. In the design of masonry structures, the assembly of masonry units, mortar and grout is assumed to result in a material of uniform compressive strength[21,22]. Ultimate masonry stresses are based on either code assumed values or masonry prism tests. If code assumed values are utilized, the strength of the

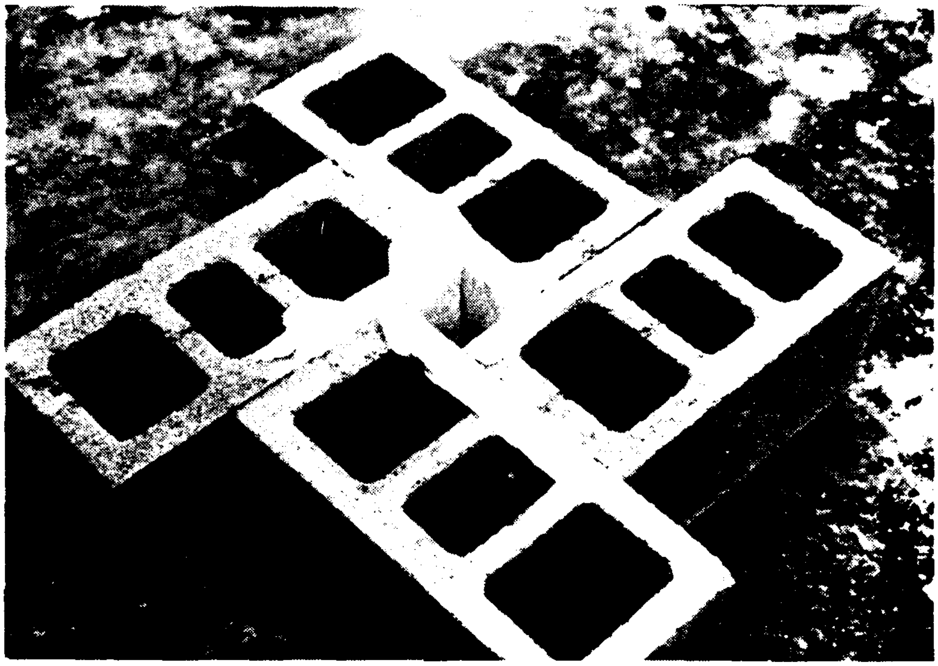


Figure 3.6. Grout Mold.

Table 3.13. Compressive Strength of Grout Specimens.

Specimen	Age (days)	Average Cross-Sectional Area (in.^{2})	Maximum Load (lbs.)	Compressive Strength (psi)
1	9	8.81	21,000	2486
2	9	9.57	21,000	2194
3	8	9.38	25,900	2761
4	8	9.38	28,750	3065
Avg. for four samples				2627

grout should meet or exceed the assumed strength. If the ultimate strength of the masonry is determined from prism tests, the strength of the grout should be similar to the strength of the prism. In either case, since allowable stresses are only a fraction of the ultimate stress of the masonry, it would appear that the strength of the grout is not critical, and that the testing of grout is undertaken primarily for quality control purposes. The average compressive strength of the grout, at an age of 8-9 days, was 2627 psi. This compared favorably with the 28-day net-area average masonry strength of 2829 psi.

3.5 Post-Tensioning Steel

The post-tensioning steel utilized in this study was a continuously threaded, 5/8" diameter hot-rolled and proof-stressed alloy steel conforming to the Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete, ASTM A722-75, manufactured by Dywidag Systems International, of Lincoln Park, New Jersey. Properties of the steel as quoted by the manufacturer are shown in Table 3.14. Nuts manufactured for use with the post-tensioning rod were employed in this study. Dywidag post-tensioning rods are readily available and are used in post-tensioned concrete construction and as rock anchors. Figure 3.7 shows a short length of the post-tensioning rod and nuts used in the study.

Table 3.14. Prestressing Steel Properties.

Nominal Diameter	5/8"
Cross-Sectional Area	0.28 in.^{2}
Ultimate Stress	157,000 psi
Modulus of Elasticity	29,500,000 psi
Maximum Jacking Force	34,800 lbs.
Maximum Lockoff Force	30,500 lbs.
Maximum Thread Diameter	0.693 in.

Figure 3.7. Post-Tensioning Steel and Accessories.

CHAPTER 4

WALL PANELS

4.1 Description

Thirteen wall panels were constructed of three-core, concrete masonry pier units and Type S mortar. Each panel consisted of eight concrete masonry units in a stack bond.

Twelve wall panels were post-tensioned. Prior to constructing the wall panels, the cores of the top and bottom concrete masonry units were filled with grout except for holes formed in the cores. The middle core of each bottom block and each top block contained a 7/8" diameter hole through which the post-tensioning rod extended. The top block contained 1" diameter holes in the first and third cores through which grout was poured after the wall panel was post-tensioned.

Filling in the cores served three purposes. First, the holes formed in the center core ensured accurate positioning of the post-tensioning rod. Second, the grout poured into the cores after post-tensioning was contained by the bottom unit. Third, the filled in cores provided greater bearing area for the bearing plate anchorages and also provided more uniform load distribution. The top and bottom blocks were "stack-cast" to insure uniformity. Oiled, 7/8" diameter PVC piping was used to form the center holes, and 1" diameter cardboard tubes were used to form the holes in the first and third cores of the top units. Figure 4.1 shows the manner in which the units were stack cast, and Figure 4.2 shows

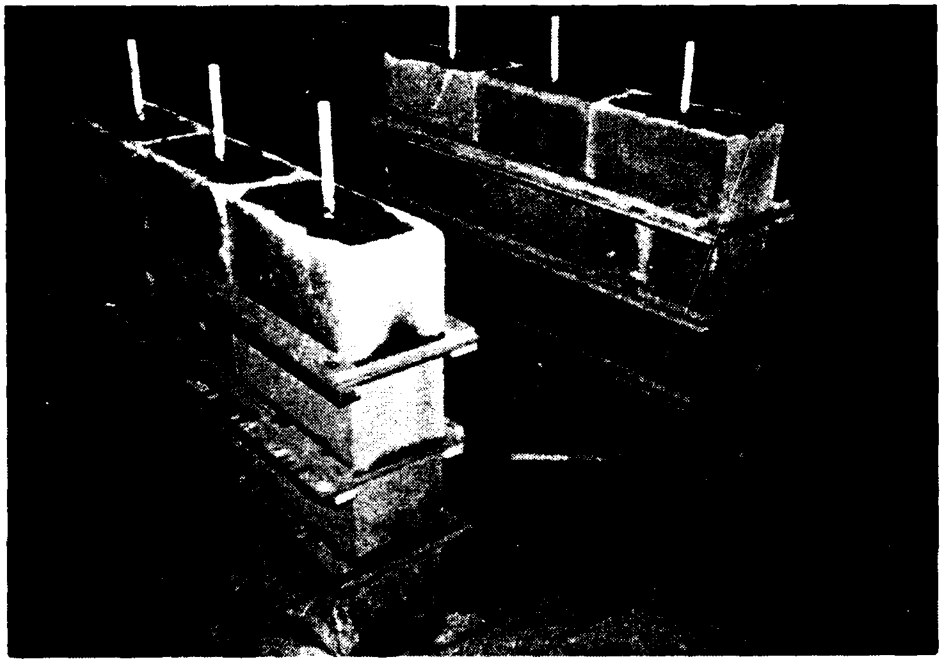


Figure 4.1. Stack-casting of Bottom and Top Concrete Masonry Units.

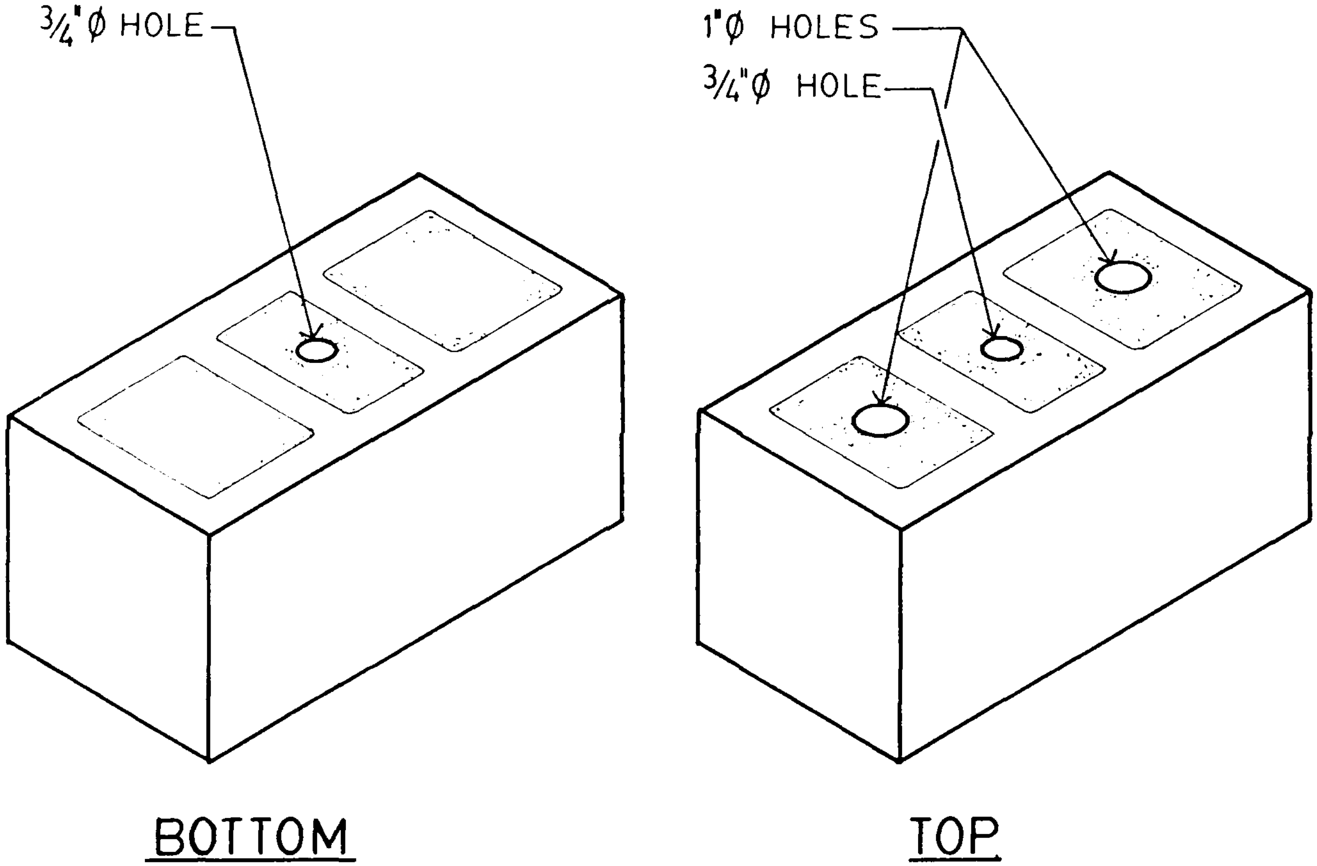


Figure 4.2. Bottom and Top Concrete Masonry Units.

typical bottom and top units. In order to insure that the hole in the bottom unit remained unobstructed and to facilitate threading the post-tensioning rod through the hole, a portion of the center core face shell was saw cut from the unit directly above the bottom unit. A portion of the center core face shell in the unit directly below the top unit was removed in similar fashion to enable grout to be placed in the center core. A 7" x 7" x 1" steel bearing plate with a 7/8" diameter hole was provided at the bottom and top of all twelve wall panels. Each of the post-tensioned wall panels contained a continuously threaded, 5/8" diameter prestressing rod, with nuts and washers resting against the bearing plates. Figure 4.3 shows a typical post-tensioned wall panel.

One conventionally reinforced wall panel was constructed. This wall panel consisted of eight, three-core concrete masonry pier units, and was reinforced with one #5, grade 40 reinforcing rod in the center core, grouted in place. Figure 4.4 depicts the conventionally reinforced wall panel.

4.2 Construction of Wall Panels

The wall panels were constructed on 4' x 4'pallets made of plywood and 2 x 6 lumber. Ledgers were provided on the side of each 2 x 6 to properly position the bottom bearing plate. Three wall panels were constructed on each pallet, a detail of which is shown in Figure 4.5.

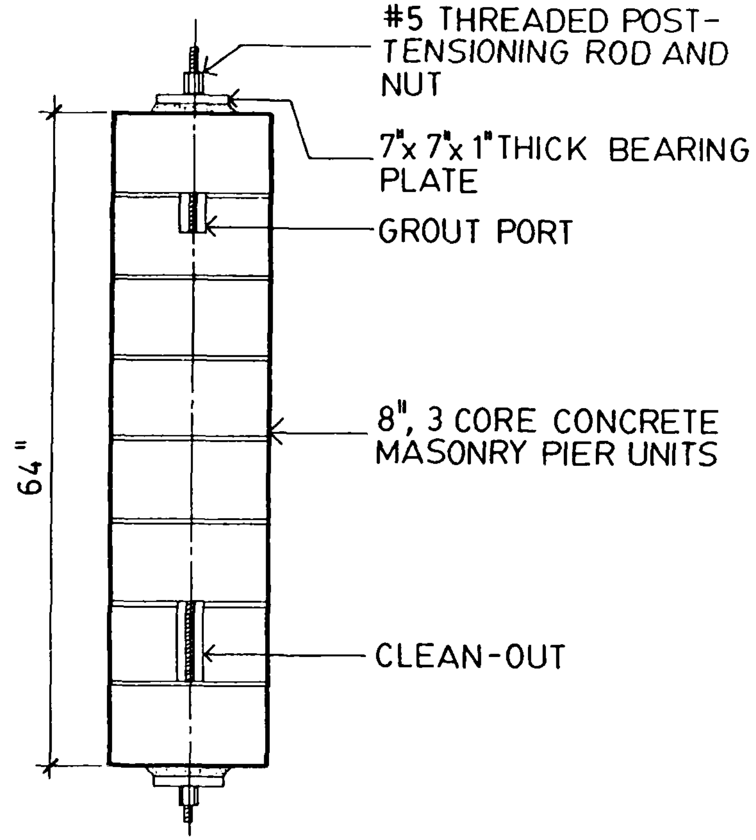


Figure 4.3. Typical Post-Tensioned Concrete Masonry Wall Panel.

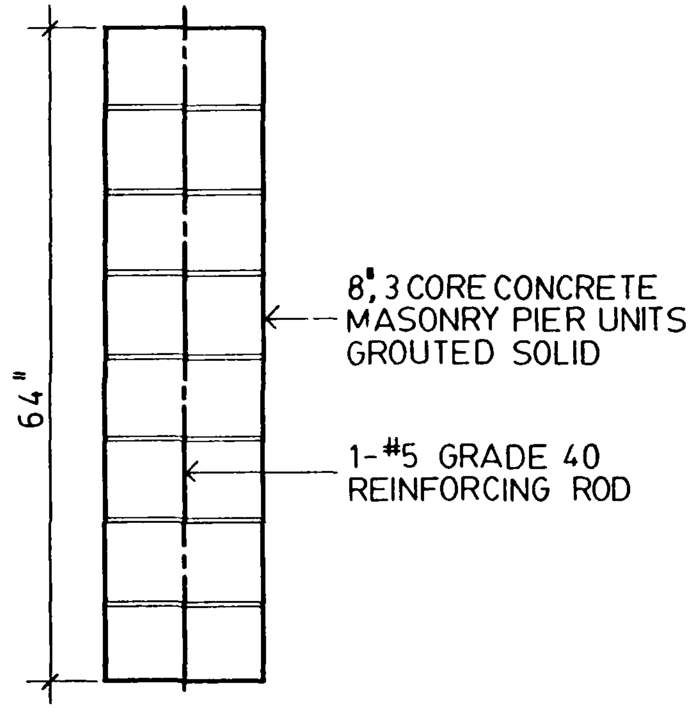


Figure 4.4. Reinforced Masonry Wall Panel.

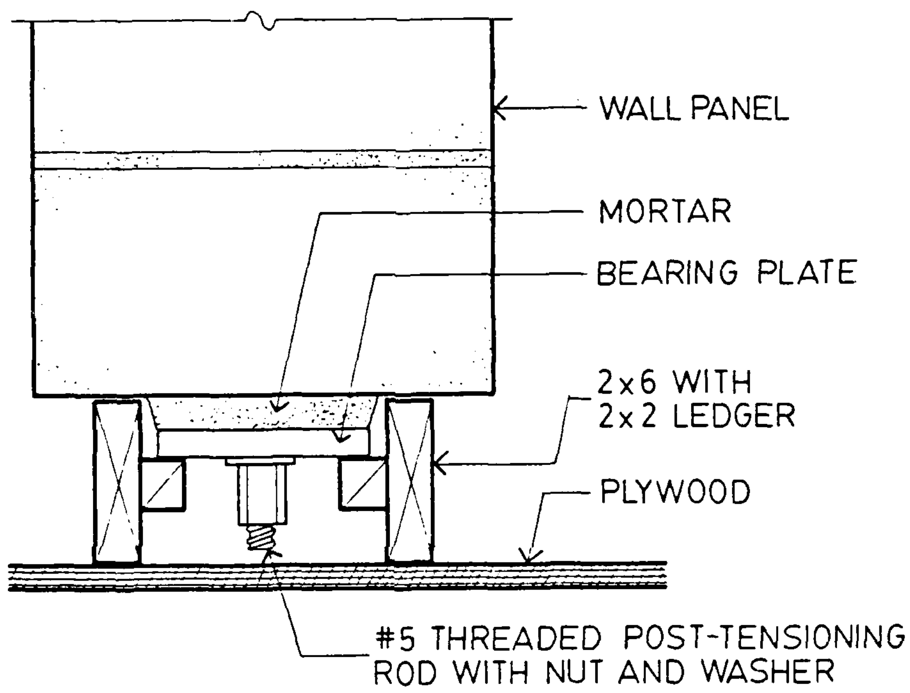


Figure 4.5. Pallet Detail.

All wall panels were constructed by an experienced mason in the following sequence. The bottom bearing plate was positioned on the pallet and a short length of PVC piping was placed in the hole in the bearing plate, a full bed of mortar was placed on the bearing plate and the bottom block set in place, with the PVC pipe insuring proper alignment of holes. The remainder of the blocks were then set in conventional fashion. All concrete masonry units were fully bedded in mortar, and the joints were struck flush with the face of the unit. Joint thickness was approximately 3/8". After the top block was set in place, a length of PVC piping was placed in the hole in the center core of the top block, the top bearing plate was set in a full bed of mortar and tamped down with a trowel. Proper alignment of the holes was provided by the PVC pipe. The PVC piping was removed, and approximately one week later, the post-tensioning rods were placed in the wall panels. No difficulties were encountered during construction of the wall panels. Figure 4.6 depicts the wall panels prior to insertion of post-tensioning rods. Approximately 40 days elapsed between the construction and post-tensioning of the wall panels.

4.3 Instrumentation of Wall Panels

Each wall panel was provided with two wire strain gauges, one on each face. Strain gauges were mounted vertically along the centerline of the fourth block from the top. Due to the manner in which the gauges function, gauges

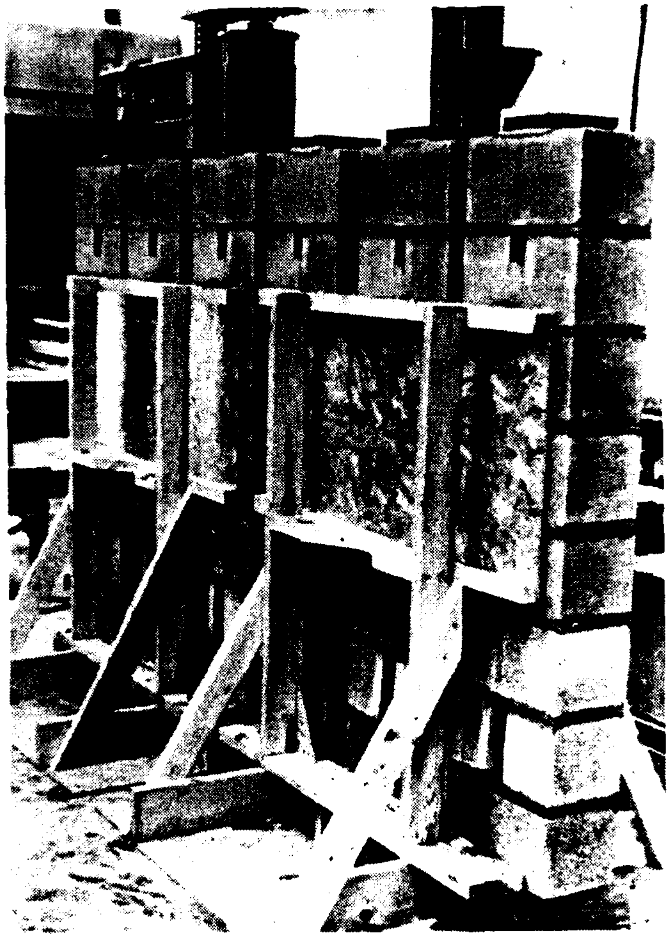


Figure 4.6. Wall Panels Prior to Installation of Post-Tensioning Rods.

could not span a mortar joint and therefore measured strain only in the concrete masonry unit. Gauges were mounted as follows. A strip of expoxy was trowelled on the masonry unit and allowed to cure. The epoxy was sanded down with an orbital sander until a smooth surface was achieved, and then cleaned with gauze pads and isopropyl alcohol. A bead of the recommended glue was placed on the epoxy, and the strain gauge was set into the glue. Pressure was applied on the strain gauge with a wood block until the glue dried. Strain gauges were capable of measuring deformations of 1×10^{-6} in./in.

Three of the wall panels were monitored for time-dependent deformation with a Whittemore Extensometer. Gauge targets were mounted on the centerline of one face of each of the three panels with an epoxy adhesive. Gauge length was 10" and spanned the mortar joints, thus providing an accurate indication of masonry deformation. Five staggered gauge lengths were placed within the middle four masonry units. Deformations were averaged and divided by the gauge length, 10", resulting in unit strains. The Whittemore Extensometer has a least reading of 1×10^{-5} in./in. Figure 4.7 shows a wall panel equipped with a wire strain gauge and Whittemore Gauge targets. A Whittemore Extensometer and accessories are shown in Figure 4.8.

4.4 Post-Tensioning

Wall panels were divided into three groups of four panels each. All of the wall panels in groups 1 and 3 and

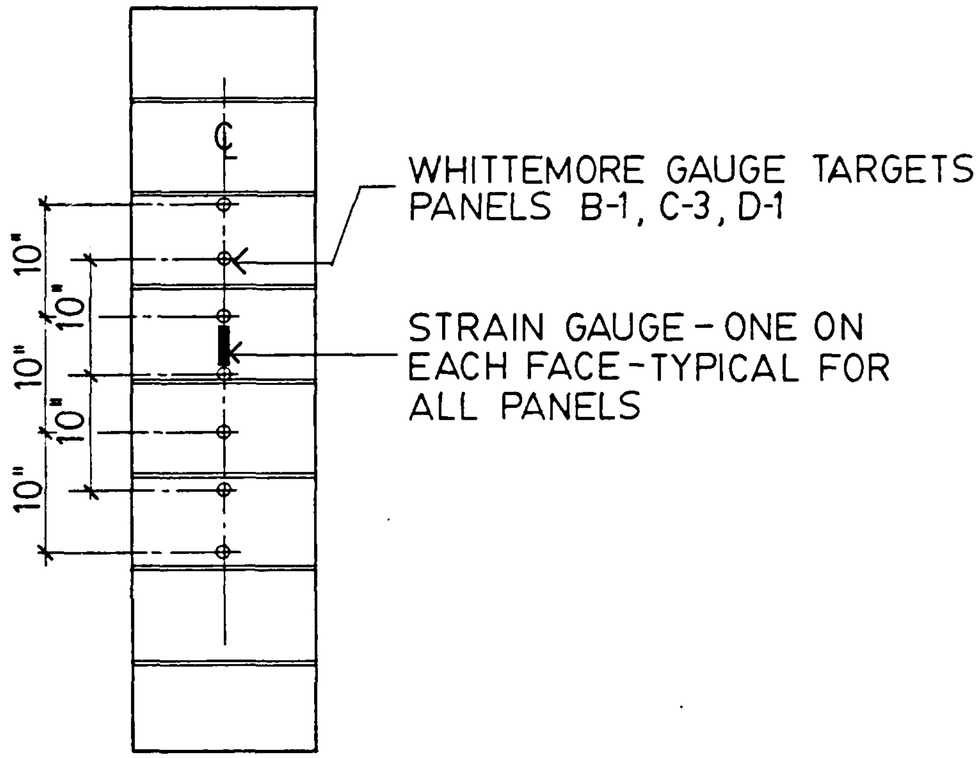


Figure 4.7. Wall Panel Instrumentation.

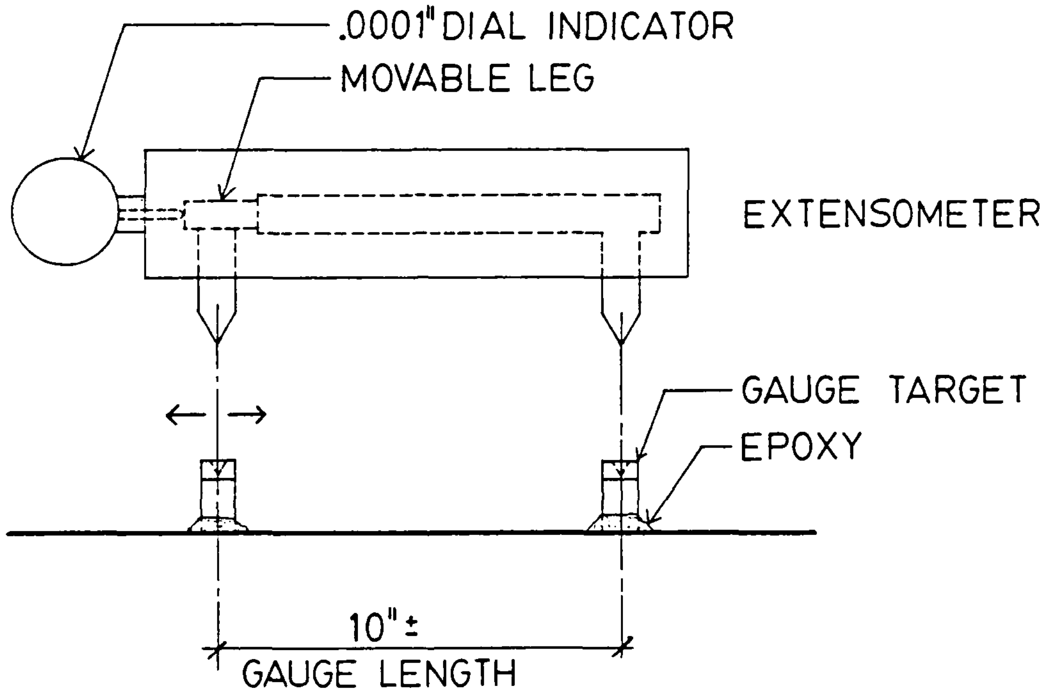


Figure 4.8. Whittemore Extensometer.

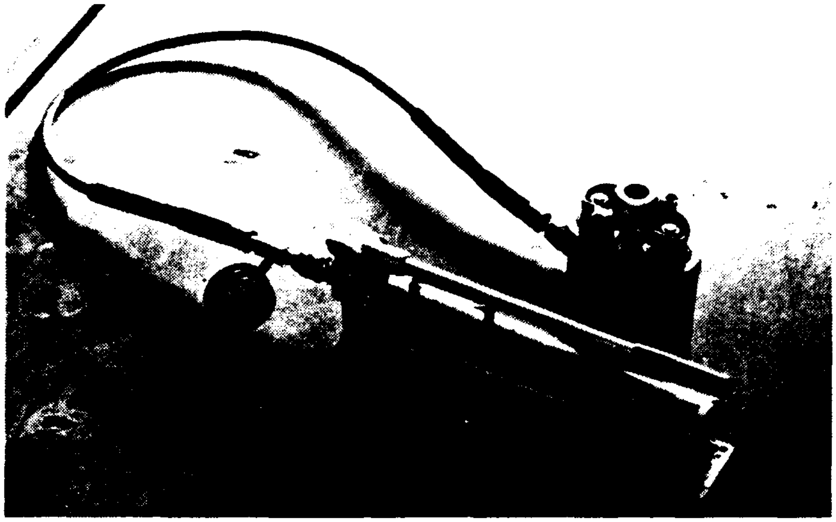


Figure 4.9. Hydraulic Pump and Jack.

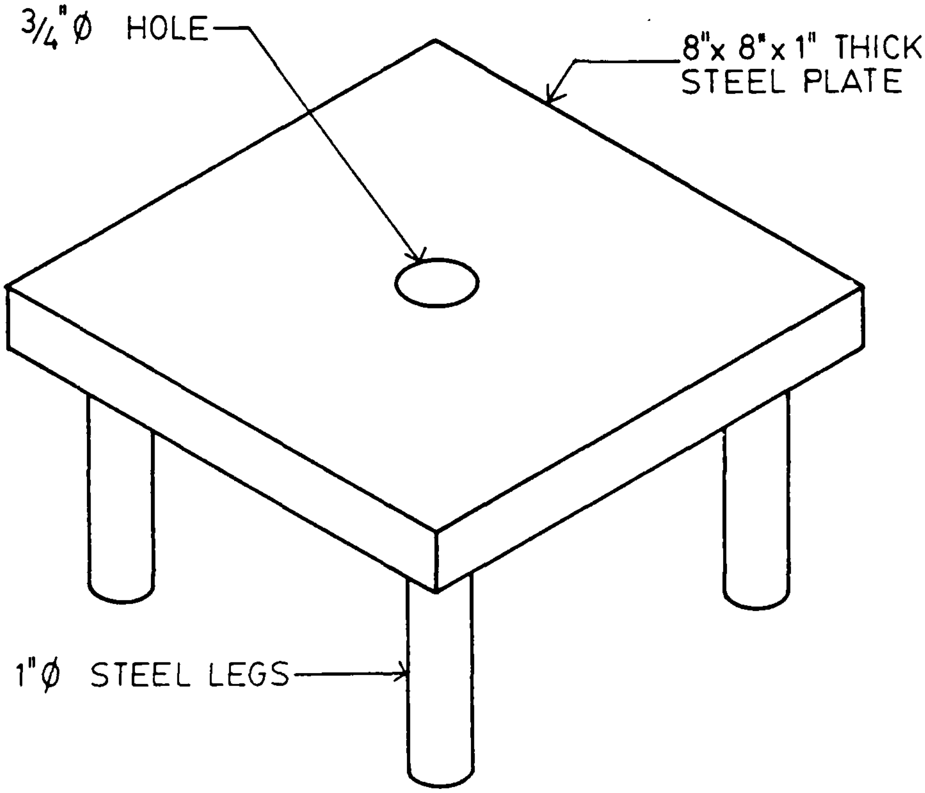


Figure 4.10. Jacking Table.

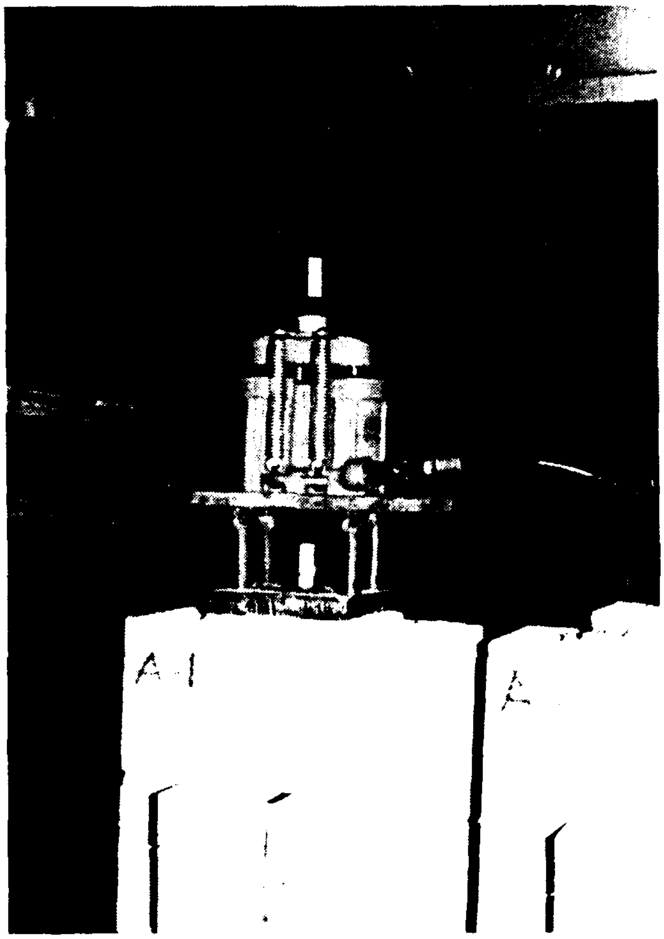


Figure 4.11. Hydraulic Jack and Jacking Table on Wall Panel.

three of the panels in group 2 were post-tensioned. The fourth panel in group 2 was used as a control for the measurement of shrinkage and creep.

The post-tensioning system developed for this project utilized a 30-ton, hole-in-the-center jack, operated by a two-speed hydraulic pump. The jack was placed on a jacking table fabricated of 3/4" thick steel plate and 1" diameter steel legs. The bottom and top nuts and washers were threaded on to the post-tensioning rod and fastened finger tight. A box wrench was placed on the top nut, and the jacking table was set in place on the bearing plate. The hole-in-the-center jack was then set on the jacking table, and a 3" x 3" x 3/4" thick steel bearing plate and nut were threaded down the post-tensioning rod, coming to rest on the jack. As the post-tensioning force was applied, the nut against the top bearing plate was tightened with the box wrench. Post-tensioning force was monitored with a hydraulic gauge on the pump. Prior to post-tensioning, the gauge was calibrated in a testing machine and found to be accurate to within 200 pounds. Figures 4.9 and 4.10 show the hydraulic pump and jack and the jacking table, and Figure 4.11 shows the post-tensioning system in place on a wall panel. Post-tensioning was accomplished without difficulty.

Strain in the concrete masonry units was measured during the application of post-tensioning force and after the nut bearing against the top bearing plate had been

tightened with the box wrench and the load on the jack released. Table 4.1 presents the post-tensioning forces, computed compressive stresses, strains in the masonry units, and lockoff losses. It is interesting to note that although the post-tensioning force applied by the jack was measured on a gauge with a least count of tons, the average measured strain in the masonry units resulting from the application of the post-tensioning force was quite consistent within each group, varying by 7×10^{-6} in./in. in group 2 and 3×10^{-6}in./in. in group 3. The only significant variation in measured masonry unit strain under initial applications of post-tensioning force was in panel A-1, the first panel post-tensioned, where the measured masonry unit strain of 45×10^{-6} in./in. was significantly below the recorded strains of 57×10^{-6} and 60×10^{-6} for the remaining panels in the group. Table 4.1 also shows average measured strain in the masonry units at lockoff. The change in strain at lockoff varied from +4% for panel A-1 to -13.3% for panel A-3. Based on the change in measured strain, the average lockoff losses are 1.93% for group 1, 5.96% for group 2, and 5.88% for group 3. Lock-off losses were significantly affected by the manner in which the nut was tightened. A torque wrench would provide better control and would enable lock-off losses of less than 5% to be consistently achieved.

The Whittemore Extensometer was used to measure masonry strain at lock-off in panels B-1 and C-3. Table 4.2 presents a comparison of the average measured strains in the

Table 4.1. Compressive Strain Due to Post-Tensioning Forces and Lockoff Losses.

Group	Wall Panel	Post-Tensicn. force (lbs.)	Calculated Compressive Stress in Masonry Unit (psi)	Avg. Meas. Strain in Masonry Unit Due to Post-Tensioning (in./in.)	Avg. Meas. Strain in Masonry Unit at Lockoff (in./in.)	Change in Measured Strain at Lockoff (%)	Avg. Lockoff Loss for Group (%)
	A-1	12,000	175	.000045	.000047	+4	
1	A-2	12,000	175	.000057	*	*	1.93
	A-3	12,000	175	.000060	.000052	-13.3	
	B-1	12,000	175	.000057	.000059	+3.5	
	B-2	17,000	248	.000073	.000069	-5.5	
2	B-3	17,000	248	.000069	.000065	-5.8	5.96
	D-3	17,000	248	.000076	.000071	-6.6	
	C-1	23,000	336	.000087	.000086	-1.1	
3	C-2	23,000	336	.000086	.000082	-4.7	5.88
	C-3	23,000	336	.000084	.000075	-10.7	
	D-2	23,000	336	.000085	.000079	-7.0	

*Data not obtained

Table 4.2. Strain in Masonry Compared with Strain in Masonry Unit.

Wall Panel	Post-Tensioning Force (lbs.)	Average Measured Strain in Masonry Unit at Lockoff (in./in.)	Average Measured Strain in Masonry at Lockoff (in./in.)
B-1	12,000	.000059	.000054
C-3	23,000	.000075	.000074

masonry units with the average measured strains in the masonry. For panel B-1, the measured masonry strain was 5×10^{-6} in./in. less than the measured masonry unit strain of 59×10^{-6} in./in., a difference of 8.5%. For panel C-3, the measured masonry strain was 1×10^{-6} in./in. less than the measured masonry unit strain of 75×10^{-6} in./in., a difference of 1%. Although only two panels can be compared, the data suggest that the strain in the masonry is closely approximated by the strain in the masonry units.

4.5 Grouting

Grouting was accomplished following post-tensioning. Cleanouts at the bottom of the wall panel were closed with pieces of 1/2" plywood and held in place with stovepipe wire. "Funnels" constructed of plywood and 3/4" thick lumber were fabricated and tied in place in front of the grout port in the center core at the top of the panel. Grout was poured through the 1" diameter holes in the top unit utilizing buckets and a steel funnel, and was consolidated with a puddling stick. Consolidation of grout in the center core was accomplished with a spring type plumbers snake. No problems were encountered during grouting.

CHAPTER 5

DESIGN PROCEDURES

5.1 Post-Tensioned Wall Panels

The design procedure used in this study employed the basic principle used in the design of prestressed concrete, where the compressive stresses caused by the application of a prestress force are used to overcome tensile stresses caused by the application of external loads. The stress in the outermost fiber of a conventionally reinforced member is determined from equation (5.1):

$$f = \frac{P}{A} \pm \frac{M}{S} \tag{5.1}$$

where

f = stress in outermost fiber, psi

P = prestress force, lbs.

A = area, $in.^2$

M = moment due to external forces, in-k.

S = section modulus, in^3

5.1.1 Allowable Moment

For purposes of this study, the allowable moment is defined as that moment which will cause a tensile stress equal to zero in the outermost masonry fiber. The allowable moments for panels post-tensioned with forces of 12K, 17K, and 23K, and computed in accordance with equation (5.1), are 26.3 in-k, 37.2 in-k and 50.4 in-k, respectively. Calculations are shown in Appendix A. These moments were

based on the assumption that the grout placed in the wall panels possessed a measure of flexural tensile strength. While there is no accepted design value for the modulus of rupture for grout, Winter and Nilson (25) indicate that for sand and gravel concrete, the modulus of rupture is approximately 0.5 to 0.7 times the split cylinder strength, and that the split cylinder strength is approximately 6 to 7 times the square root of the ultimate strength of the concrete. Using this criterion and an assumed value of 2500 psi for the compressive strength of the grout, the modulus of rupture was estimated at 150 psi to 245 psi. This was felt to be sufficiently high to justify the assumption that the grout would not crack prior to the allowable moment. The section modulus, S, was computed based on a rectangular cross-section using the formula $S = \frac{bh^2}{6}$ and was equal to 150 in.^{3}. Figure 5.1 shows the theoretical stress distributions in the post-tensioned wall panels at the respective allowable moments.

5.2 Conventionally Reinforced Wall Panel

The allowable moment for the conventionally reinforced wall panel was computed using the working stress design technique and found to be 21.3 in-k. In this technique, the masonry is assumed to possess no tensile strength, the neutral axis is determined on the basis of a cracked, transformed section, and code stipulated stresses for masonry in compression and steel in tension are used to determine the masonry and steel moments. The lower moment,

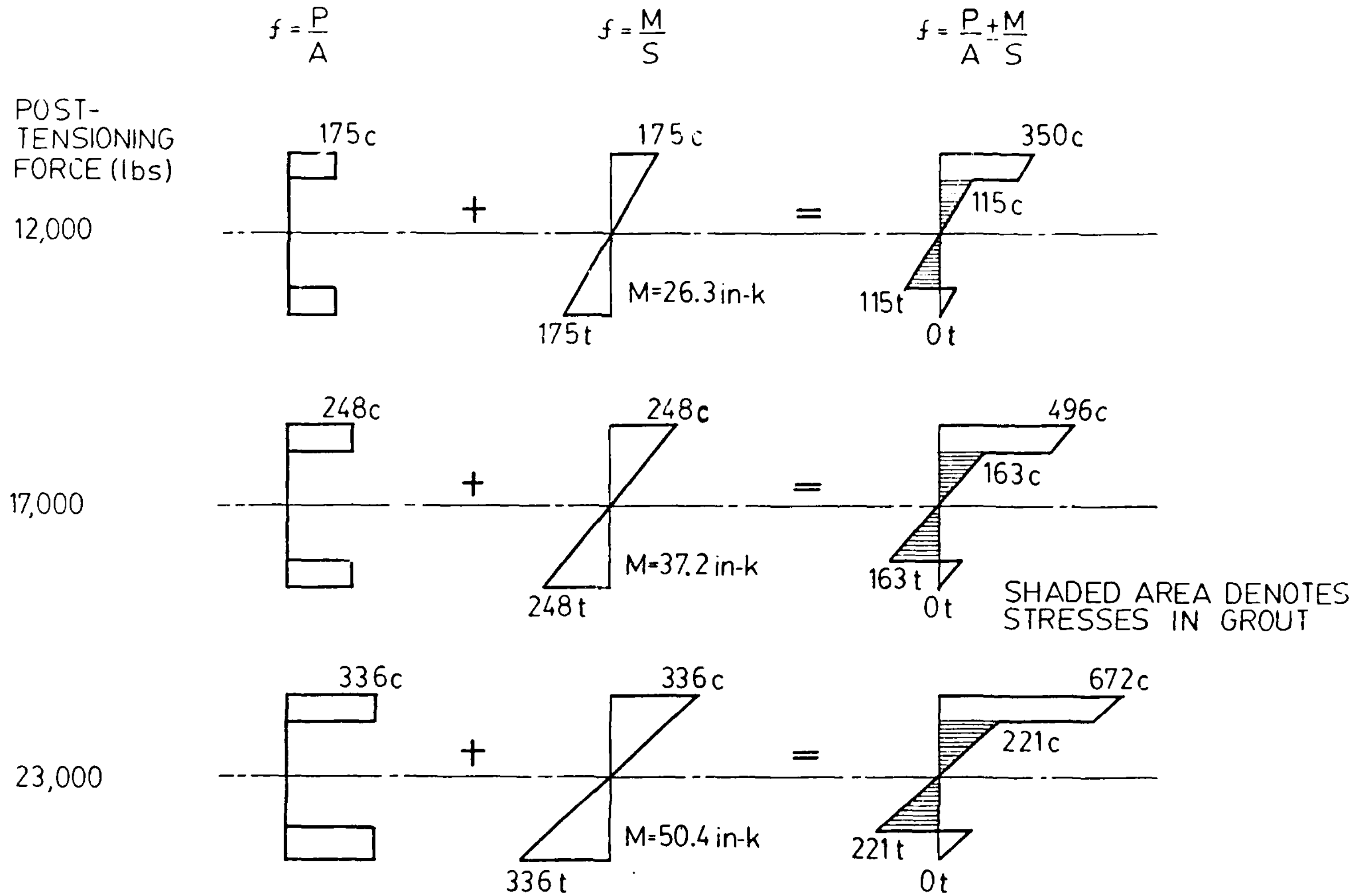


Figure 5.1. Theoretical Stress Distributions in Post-Tensioning Masonry Wall Panels.

in this case the moment due to steel stress, governs the design. The computations are shown in Appendix A.

5.3 Ultimate Moment

In relatively lightly post-tensioned members, the cracking moment may be greater than the moment the member can withstand after it has cracked. This condition is most likely to occur in members that are prestressed concentrically with small amounts of steel[15]. It is prudent, therefore, to compute the ultimate moment. The working stress design technique, with its assumed triangular compressive stress distribution, will not accurately predict ultimate moments. Citing data from tests on brick masonry performed by Withey, Sahlin[20] indicates that the Whitney formula for the ultimate moment is a reinforced concrete beam will accurately predict the ultimate moment in a reinforced masonry beam[20]. The formula cited by Sahlin is:

$$m = q(1 - 0.59q) \leq 0.4 \tag{5.2}$$

where:

$$m = \frac{M}{bd^2 f'_m} \quad \text{and} \quad q = \frac{\rho f_y}{f'_m}$$

b = width of beam

d = effective depth of beam

M = ultimate moment

f'_m = ultimate prism strength of masonry in compression

ρ = percentage of steel

f_y = yield stress of steel

Sahlin indicates that the value of m is limited to approximately 0.4 due to the brittleness of the masonry tested. Utilizing the formulas cited by Sahlin, the ultimate moment for the post-tensioning masonry wall panel was computed to be 141.1 in-k, and for the conventionally reinforced wall panel, 45.1 in-k. Calculations are provided in Appendix B. The rather significant difference in ultimate moments is primarily due to the difference in the yield strength of the post-tensioning steel, which at 157,000 psi is much higher than the Grade 40 steel yield strength of 40,000 psi.

CHAPTER 6

TESTING OF WALL PANELS

6.1 Test Frame

Figures 6.1 and 6.2 show the test frame and loading diagram utilized in this study. Loads were applied through a hydraulic jack with a 40,000 lb. capacity and monitored on a scale graduated in 50 lb. increments. The hydraulic jack was connected to a transfer beam with two loading points 18 in. apart. The 18 in. spacing between loading points was used in order to apply loads at one-third points of the 54 in. span, which would create a uniform bending moment between loading points.

6.2 Instrumentation

Each wall panel was provided with two-wire type strain gauges, one on each face shell of the fourth block from the top of the panel, which were used to monitor tensile and compressive strains in the masonry unit. Deflections were monitored with dial indicators with a least reading of 1×10^{-3} in.

6.3 Loading Procedure

Flexural testing of wall panels started seven days after post-tensioning and grouting, and was completed within a five-day period. Following placement of the wall panel in the test frame, the transfer beam was adjusted manually until it came into contact with the wall panel. Strain gauges were then connected to the strain indicator and the

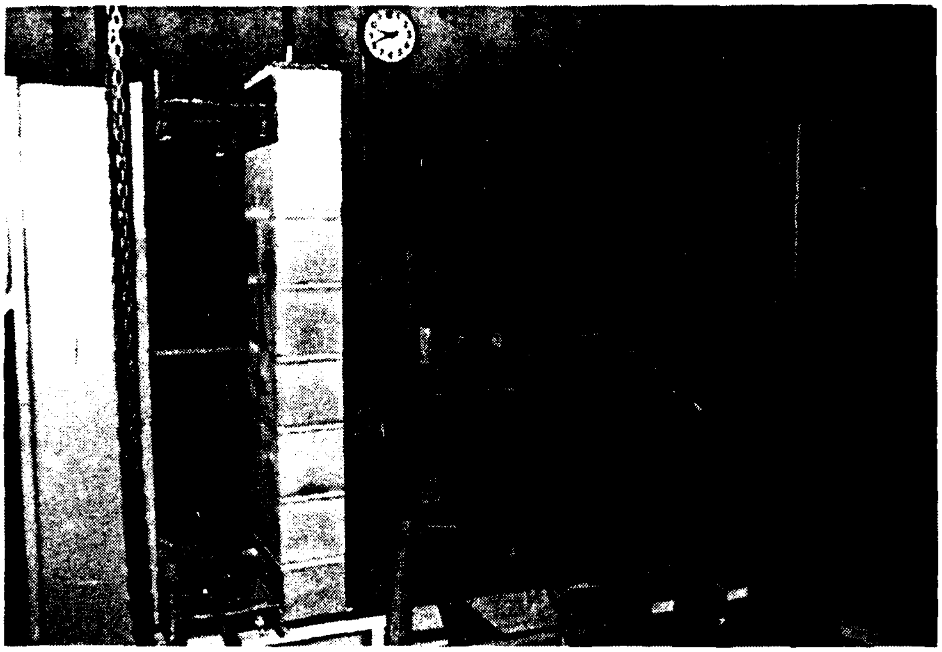


Figure 6.1. Test Frame.

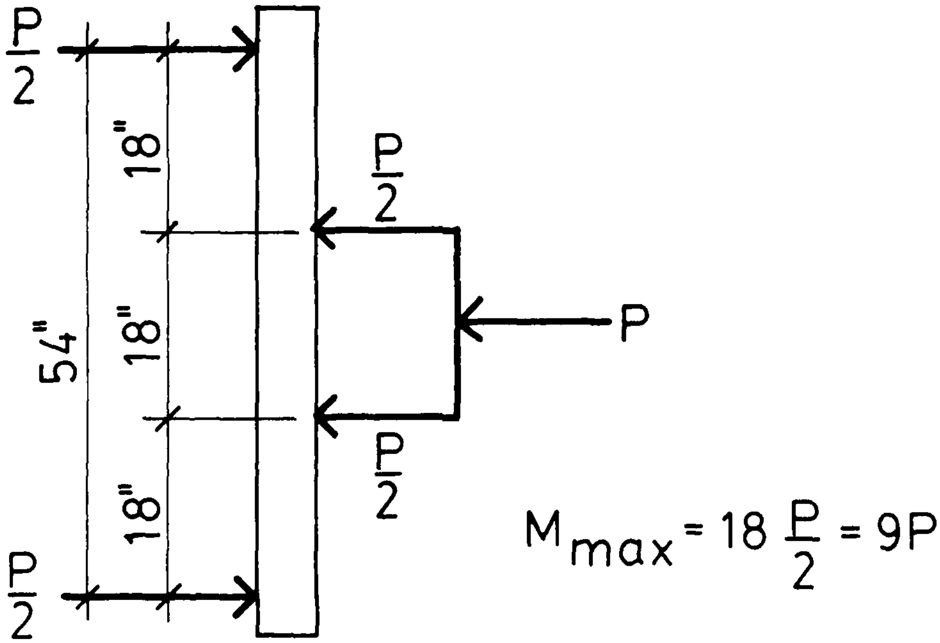


Figure 6.2. Loading Diagram.

dial gauges used to measure deflection were set in place. Loads were applied in 1-kip increments, and were held constant while visual observations, strains, and deflection were recorded. Loads were applied and behavior was monitored until the cracking moment was reached, at which point loads were released, instrumentation reset, and the loading cycle repeated in identical fashion for a second time. Each loading cycle took approximately 20 minutes. The stresses caused by application of a moment are a function of the section modulus. Since cracking of the masonry changes the section modulus, the behavior of the panel in the cracked condition will be different from the behavior of the panel in the uncracked condition. The second loading cycle was intended to examine this difference.

The computed allowable moment was known at the time that testing commenced; however, the cracking moment was not. Three panels were tested before it was noted that there was an abrupt change in the rate of deflection that corresponded with a sharp reduction and leveling-off of measured tensile strains. This was assumed to be the cracking moment, and the initial loading cycle of the remaining panels was terminated when two successive tensile strain gauge readings indicated either a leveling-off or decrease in measured tensile strain. Five of the panels were reloaded to produce a bending moment close to the calculated ultimate moment and the load then released. The behavior of the panels during loading and unloading was observed. Four of the panels were

reloaded until the steel was perceived to yield. The conventionally reinforced panel was loaded to destruction in one loading cycle. Table 6.1 shows the magnitude of load applied to each wall panel during each loading cycle.

6.4 Problems Encountered During Testing

Several problems were encountered during the testing of the wall panels. Due to the weight of the wall panels and the configuration of the test frame it was not possible in all cases to immediately seat the panel against the reactions. In addition, all of the panels were slightly twisted along their longitudinal axis to some degree. This prevented complete bearing against reactions until a portion of the load was applied. Based on visual observation all of the panels were completely seated after application of a 1 kip load. Deflection was measured at the same distance from the bottom reaction on each panel; however, it was not possible to measure deflection at the center of the span. The dial indicators were observed to "stick" on occasion which proved to be a significant problem for two of the panels tested, and may be the cause of slight inconsistencies in deflection data recorded for other panels. The transfer beam was positioned accurately initially· however, as loads were applied slight eccentricities in the test frame caused the beam to shift, evidently causing unequal application of load and variations in moment. This was observed only at loads in the 15-16 kip range, when panel deflection became extreme, and as a result, the

Table 6.1. Wall Panel Loading Cycles.

Panel	Post-Tens. Force (kips)	Allow. Moment (in-k)	FIRST LOADING CYCLE			SECOND LOADING CYCLE			THIRD LOADING CYCLE		
			Load (k)	Moment (in-k)	Percentage of allow. Moment	Load (k)	Moment (in-k)	Percentage of allow. Moment	Load (k)	Moment (in-k)	Percentage of allow. moment
A1	12	26.25	7	63	240	7	63	240	14	126	480
A2	12	26.25	8	72	274	14	126	480	-	-	-
A3	12	26.25	12	108	411	16	144	548	-	-	-
B2	17	37.20	13	117	315	16	144	387	-	-	-
B3	17	37.20	8	72	194	16*	144	387	-	-	-
D3	17	37.20	8	72	194	16*	144	387	-	-	-
C1	23	50.40	11	99	196	16*	144	286	-	-	-
C2	23	50.40	9	81	160	16	144	286	-	-	-
D2	23	50.40	10	90	179	16*	144	286	-	-	-
E1	0	21.3	6*	54	253						

*indicates load above which steel yielded.

ultimate moments of panels tested to destruction should only be considered approximations.

6.5 Test Data

Tensile strain, compressive strain and deflection readings were plotted against moments for each group of post-tensioned panels, and for the conventionally reinforced wall panel. The allowable moment for each group of panels, is shown on each set of plots. Due to problems encountered in seating the wall panels against the reaction points, deflections are shown as the change in deflection from the equilibrium position for the first increment of load.

6.5.1 12,000 lb. Post-Tension Force

Figure 6.3 shows measured tensile strain, deflection and compressive strain plotted against moments for the panels post-tensioned with a 12,000 lb. force. The allowable moment for this group of panels was 26.3 in-k. Panel A-1 was the first panel tested, and due to problems with the test-frame and a faulty dial indicator, deflection data was not obtained for the first loading cycle. In order to obtain deflection data for two loading cycles, panel A-1 was subjected to two reloads. Panel A-3 was the second panel tested, and since the flexural behavior of the post-tensioned panels was not understood at the time, panel A-3 was subjected to a moment of 108 in-k during the first loading cycle. During the first loading cycle, all of the panels in this group exhibited similar elastic behavior. An increase in moment was accompanied by a linear increase in

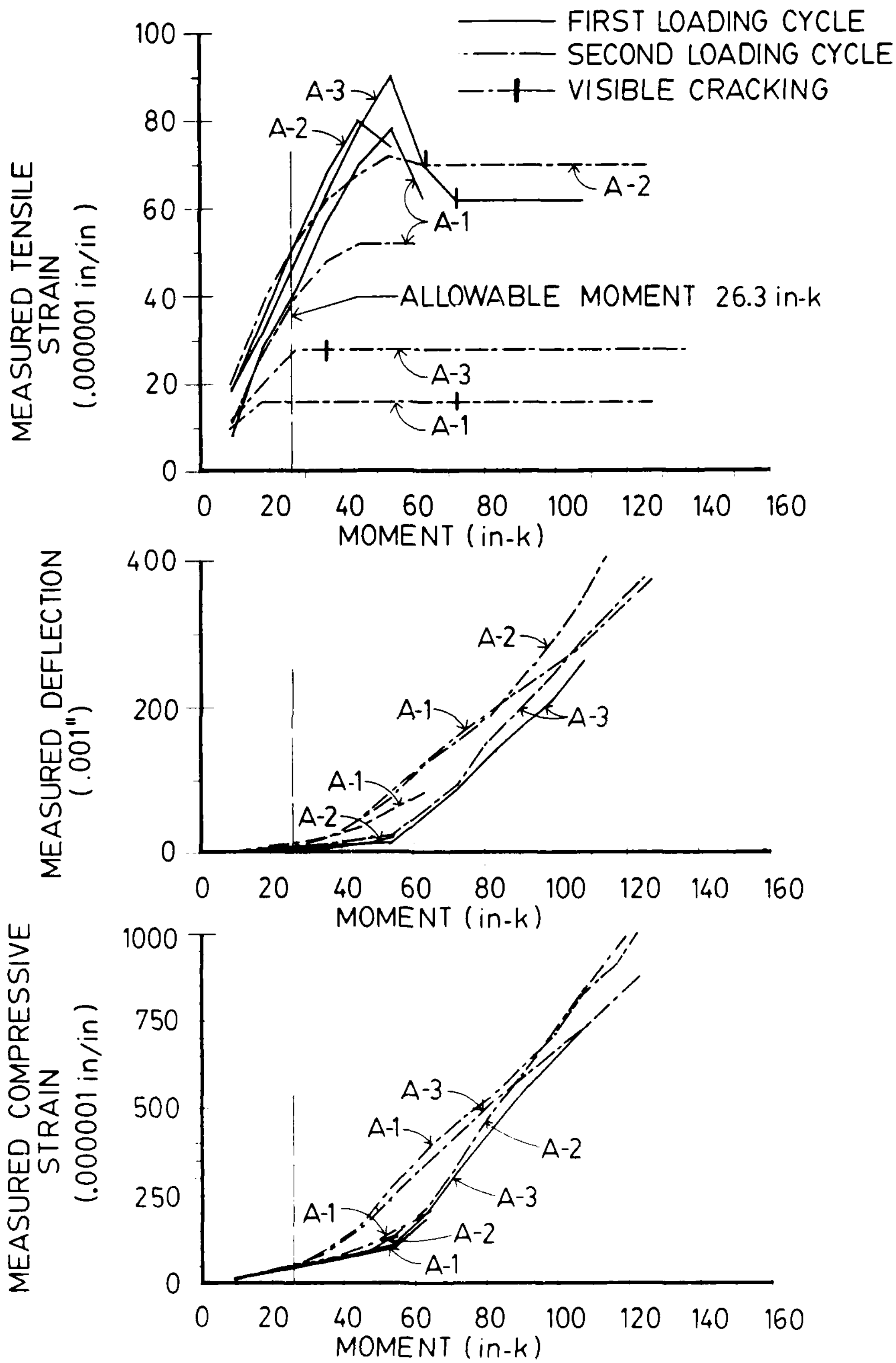


Figure 6.3. Test Data, 12,000 lb. Post-Tension Force.

tensile and compressive strain and deflections. This elastic behavior continued to a moment of 45 in-k for panel A-2 and 54 in-k for panels A-1 and A-3. At these moments a marked decrease in tensile strains was recorded, which was accompanied by corresponding and sharp increases in deflection and compressive strain for each of the panels. Under the continued application of moments above 63 in-k, panel A-3 exhibited constant tensile strain, and linear increases in deflection and compressive strains. Cracks in the mortar joints became visible at a moment of 72 in-k. For the panels in this group the decreases in tensile strain and corresponding changes in deflection and compressive strain occurred at 18 in-k and 27 in-k above the allowable moment of 26.3 in-k.

During the second loading cycle, the behavior of the panels was again elastic; however, changes in the rate of increase of tensile strain began to occur at approximately the allowable moment. For panel A-1, tensile strain increased gradually to a moment of 45 in-k, beyond which it became constant. For panel A-2, tensile strain increased gradually to a moment of 54 in-k, and then decreased slightly, becoming constant at 73 in-k. Tensile strain for panel A-3 became constant at 27 in-k. During the second loading cycle changes in deflection and compressive strain were less distinct but generally corresponded to changes to tensile strain for each panel. Cracks became visible in the mortar joints of panel A-3 at 36 in-k, and in the joints of

panel A-2 at a moment of 63 in-k. The maximum moments applied to panels A-2 and A-3 during the second loading cycle were 126 in-k and 144 in-k, respectively. Upon release of load, the joints closed. Hairline cracks were visible in the joints, but visual inspection and tapping of the mortar with a hammer indicated that the mortar was otherwise sound. Panel A-1 was subjected to a third loading cycle. Tensile strain became constant at 18 in-k, at which point there was a gradual change in compressive strain. A distinct change in compressive strain and deflection occurred at 36 in-k, and cracks became visible at 72 in-k. A maximum moment of 126 in-k was applied, and upon release of loads, the cracks closed, with no apparent additional damage to the mortar.

During the first loading cycle, all of the panels behaved elastically to a moment of 18 in-k to 27 in-k above the calculated allowable moment. At the allowable moment, the calculated outermost masonry fiber tensile stress was assumed to be zero. The continued application of load evidently caused tensile stresses to develop in the masonry similar to the manner in which tensile stresses would develop in a reinforced masonry member. The difference between the allowable moment and the moment at which a decrease in tensile strain was recorded appears to be related to the tensile strength of the masonry.

The behavior of the panels under subsequent loading cycles tends to support the assumption that the decrease in

tensile strain is indicative of cracking and resulting change in section properties. In the first loading cycle, panels A-1 and A-2 were loaded until a decrease in tensile strain was recorded, and the load was released, while panel A-3 continued to be loaded beyond the moment at which the decrease in tensile strains was recorded. During the second loading cycle, tensile strains for panels A-1 and A-2 began to change at the allowable moment, but continued to increase gradually, before becoming constant, while the tensile strain for panel A-3 became constant at the allowable moment. This suggests that panels A-1 and A-2 did not crack completely during the first loading cycle. The deflection and compressive strain curves for panels A-1 and A-2 show linear behavior to approximately the allowable moment, a gradual transition, and then essentially linear behavior to the maximum moment. The transition appears to be related to section properties changing as a result of additional cracking. The transition for panel A-3 is much less pronounced and is apparently the result of more extensive cracking during the initial loading cycle.

6.5.2 17,000 lb. Post-Tension Force

Figure 6.4 shows measured tensile strains, deflections and measured compressive strain plotted against moments, for the panels post-tensioned with a 17,000 lb. force. The allowable moment for this group of wall panels is 37.2 in-k. Panel B-2 was tested before the procedure by which cracking moments could be determined became evident, and was

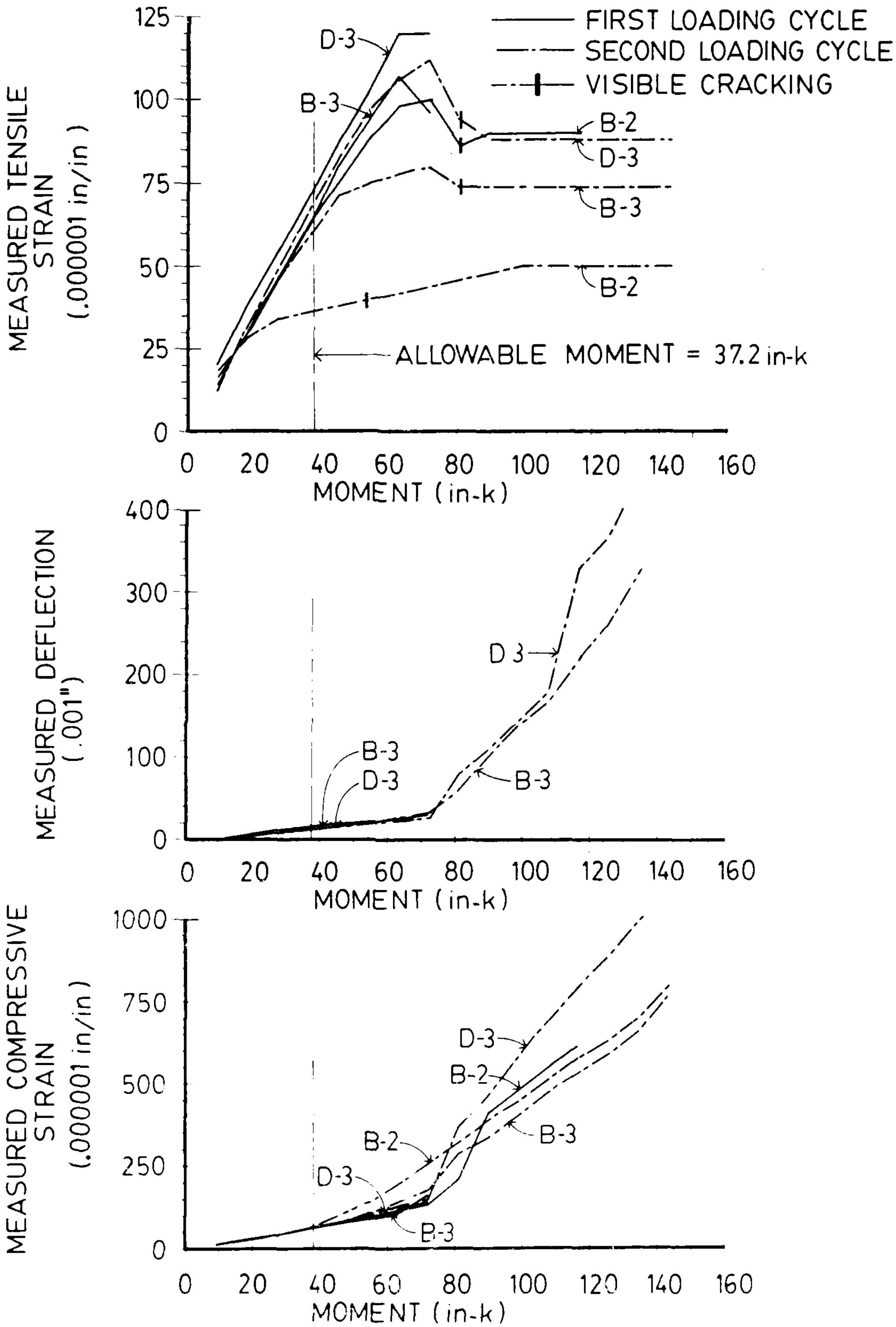


Figure 6.4. Test Data, 17,000 lb. Post-Tension Force.

not tested in accordance with the loading procedure described in section 6.3. In addition, due to faulty dial indicators, deflection data for panel B-2 were not obtained.

In the first loading cycle, all of the panels exhibited essentially identical elastic behavior. Tensile strain for panel B-2 was linear to 54 in-k, but did not decrease until 72 in-k, at which point compressive strain changed distinctly. Cracks appeared in the mortar joints at 81 in-k. Panel B-2 was loaded to a moment of 117 in-k, and when the load was released, the joints closed, with no cracks visible. Panels B-3 and D-3 exhibited linear increases in tensile strains to a moment of 63 in-k, following which a decrease in tensile strain was recorded for B-3, while no change in tensile strain was recorded for D-3. A change in deflection at 63 in-k was noted for panel B-3, but deflection for D-3 remained linear, as did compressive strains for both panels. The cracking moment for this group of panels, assumed to be the moment at which tensile strains began to decrease, was approximately 27 in-k for panel B-3 and 36 in-k for panels B-2 and B-3 above the allowable moment of 37.2 in-k. During the second loading cycle, panel B-2 exhibited a non-linear increase in tensile strain, which became constant at 99 in-k, although cracks became visible in the mortar joints at 54 in-k. Compressive strain increased linearly to the allowable moment, and then increased linearly at a different rate to the maximum moment. Panel B-2 was loaded to a maximum moment of 144 in-k. When the load was released the

panel returned to its original position. Cracks were visible at the joint between mortar and masonry unit, as shown in Figure 6.5. The behavior of panel B-2 suggests that a significant amount of cracking occurred during the first loading cycle. Tensile strain in panel B-3 was linear to 45 in-k, and decreased between 63 in-k and 72 in-k, where cracks became visible in the mortar joints. Deflection changed at 63 in-k, and a gradual change in compressive strain was noted at 54 in-k, and became more pronounced at 72in-k. Panel D-3 showed linear tensile strain to 54 in-k, and a decrease in tensile strain between 72 in-k and 81 in-k, when cracks became visible. Deflection was linear up to 72 in-k then changed abruptly. While compressive strain was linear to 63 in-k and increased abruptly at 72 in-k. Panels B-3 and D-3 were tested to destruction by applying a moment in excess of 144 in-k. In the case of both panels, the failure mode was a yielding of the steel accompanied by spalling of the masonry at the mortar joint. Figure 6.6 shows a typical failure.

As was the case with the wall panels post-tensioned with the 12,000 lb. force, behavior during the first loading cycle was elastic to the cracking moment, and behavior during the second loading cycle appeared dependent upon the extent of the change in section properties due to cracking.

6.5.3 <u>23,000 lb. Post-Tensioning Force</u>

Figure 6.7 shows measured tensile strain, deflection and measured compressive strain plotted against moments for

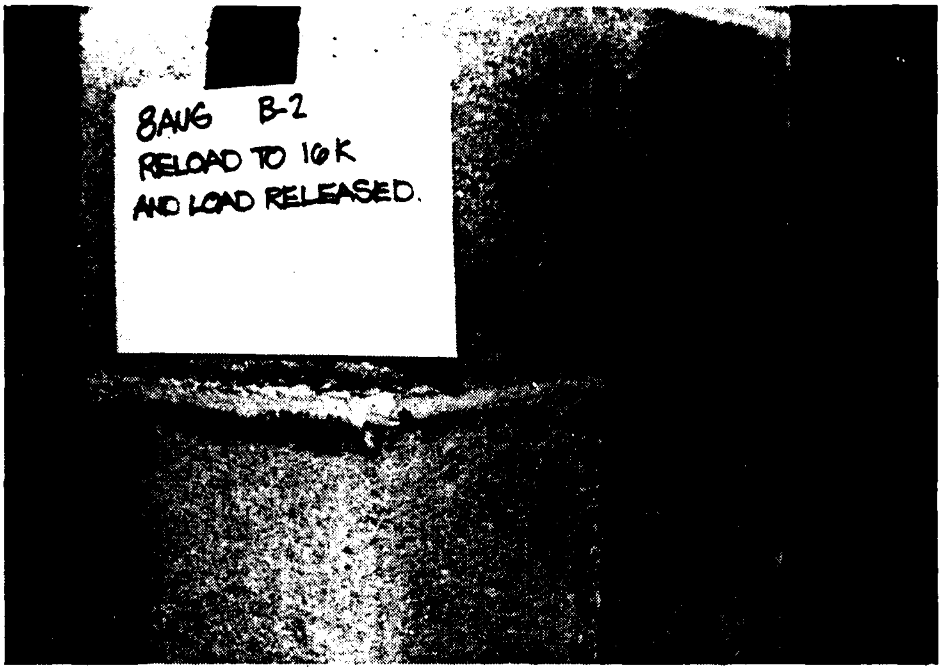


Figure 6.5. Panel B-2 Following Application of 144 in-k Moment.

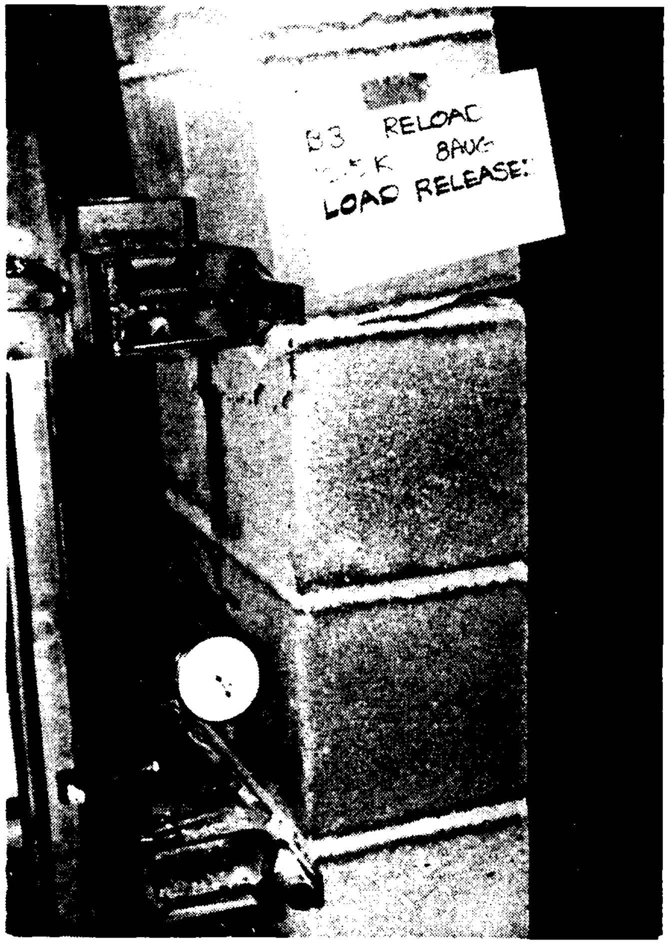


Figure 6.6. Panel B-3 Tested to Destruction.

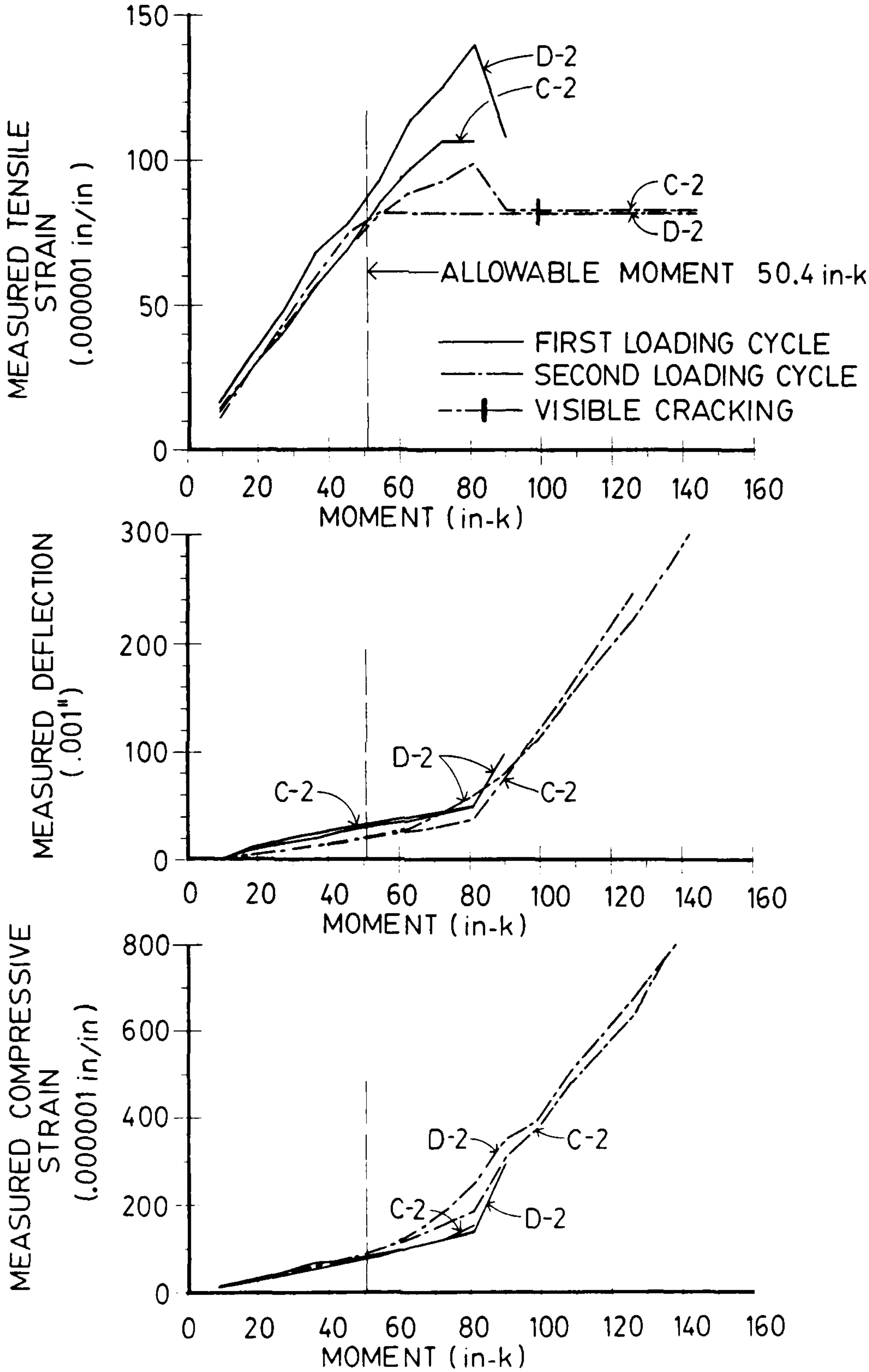


Figure 6.7. Test Data, 23,000 lb. Post-Tension Force.

the panels post-tensioned with a 23,000 lb. force. The allowable moment for these panels was 50.4 in-k. All of the panels were tested in accordance with the loading procedure described in paragraph 6.3. The strain gauges mounted on panel C-1 were found to function improperly, and as a result data obtained for panel C-1 is not shown. In the first loading cycle, tensile strain for panel C-2 increased linearly to 72 in-k and remained constant to 81 in-k. Strain and deflection were both linear to 81 in-k, suggesting that the cracking moment had been reached but that only a limited amount of cracking had occurred.

Strains and deflection for panel D-3 were linear to a moment of 81 in-k, where an abrupt decrease in tensile strain, and a corresponding increase in deflection and tensile strain occurred. During the second loading cycle, tensile strain in both panels was linear to 54 in-k. Panel C-2 continued to show an increase in tensile strain to 81 in-k, where a decrease in tensile strain was recorded. An increase in compressive strain was noted at 54 in-k, and in deflection at 72 in-k. A marked increase in compressive strain was noted at 81 in-k, and in deflection at 90 in-k. Cracks became visible in the mortar joints at 99 in-k. In the case of panel D-2, tensile strain became constant at 54 in-k, and was accompanied by a change in both deflection and compressive strain. Cracks became visible at 99 in-k. Panel C-2 was loaded to 144 in-k, and when the load was released, the panel returned to its original position, with

hairline cracks visible in the joints. Panel D-3 was loaded to a moment in excess of 144 in-k, and failed when the steel yielded.

6.5.4 Conventionally Reinforced Wall Panel

Figure 6.8 shows measured tensile and compressive strains and deflection plotted against moments for panel E-1. The allowable moment for this panel, which was tested to destruction in one loading cycle, was 20.3 in-k. Tensile and compressive strain and deflection were essentially linear to a moment of 27 in-k, at which point there was a marked change. Cracks in the mortar became visible at 36 in-k, and the steel yielded at approximately 45 in-k. The data indicate that at a moment of 27 in-k, the behavior of panel E-1 changes from that of an uncracked, elastic member to that of a cracked member.

6.6 Discussion of Test Results

6.6.1 Wall Panels in Flexure

During the first loading cycle, all of the post-tensioned panels behaved elastically until the applied moment exceeded the allowable moment. At this load, change in tensile strain, deflection and compressive strain was noted. Figure 6.9 shows tensile strains plotted against moments for all of the post-tensioned panels. For each group of panels, it can be see that the decease in tensile strain occurred at an applied moment approximately 27 in-k greater than the allowable moment. Evidently, once the allowable moment is reached, tensile stresses begin to

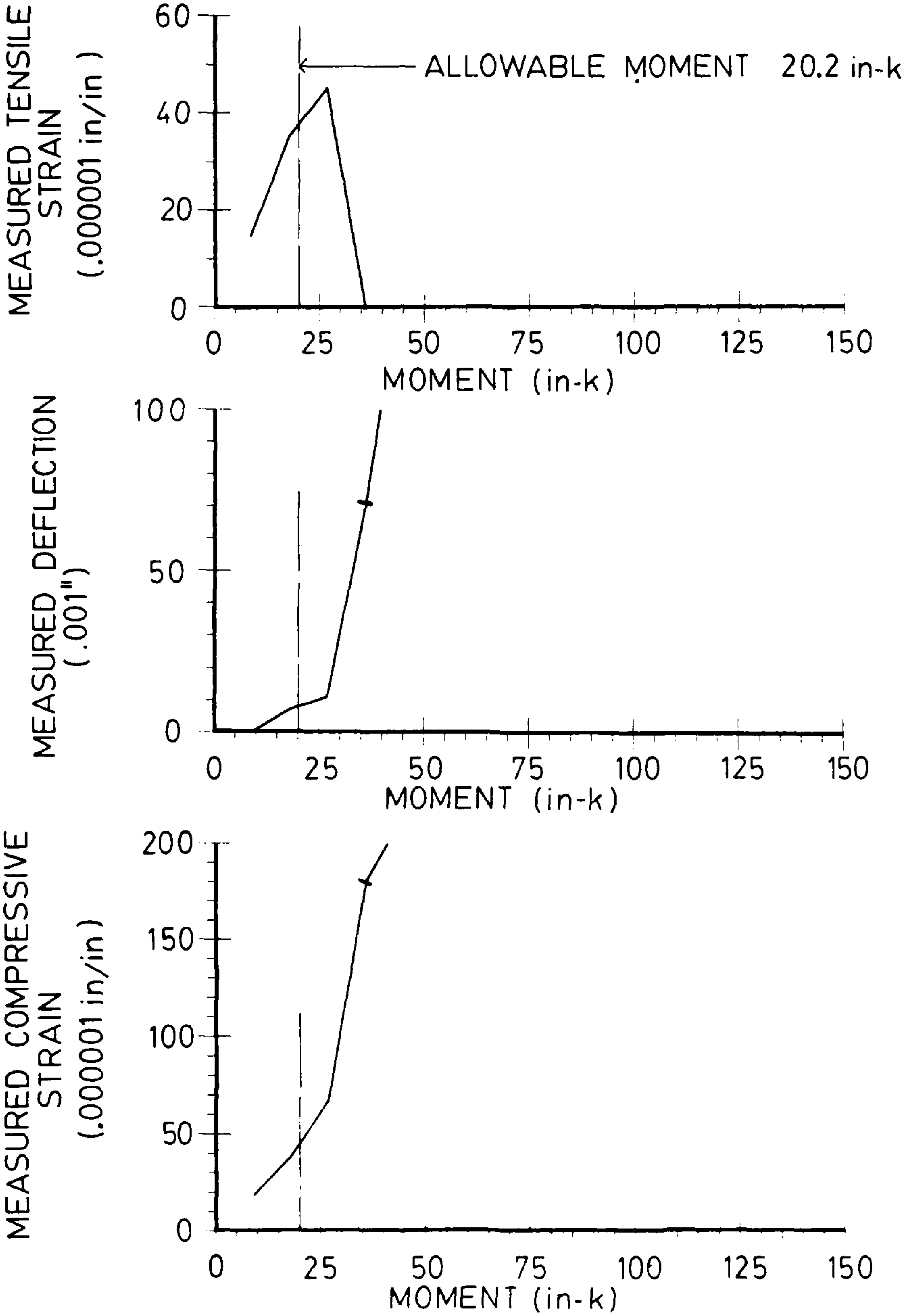


Figure 6.8. Test Data, Reinforced Masonry Wall Panel.

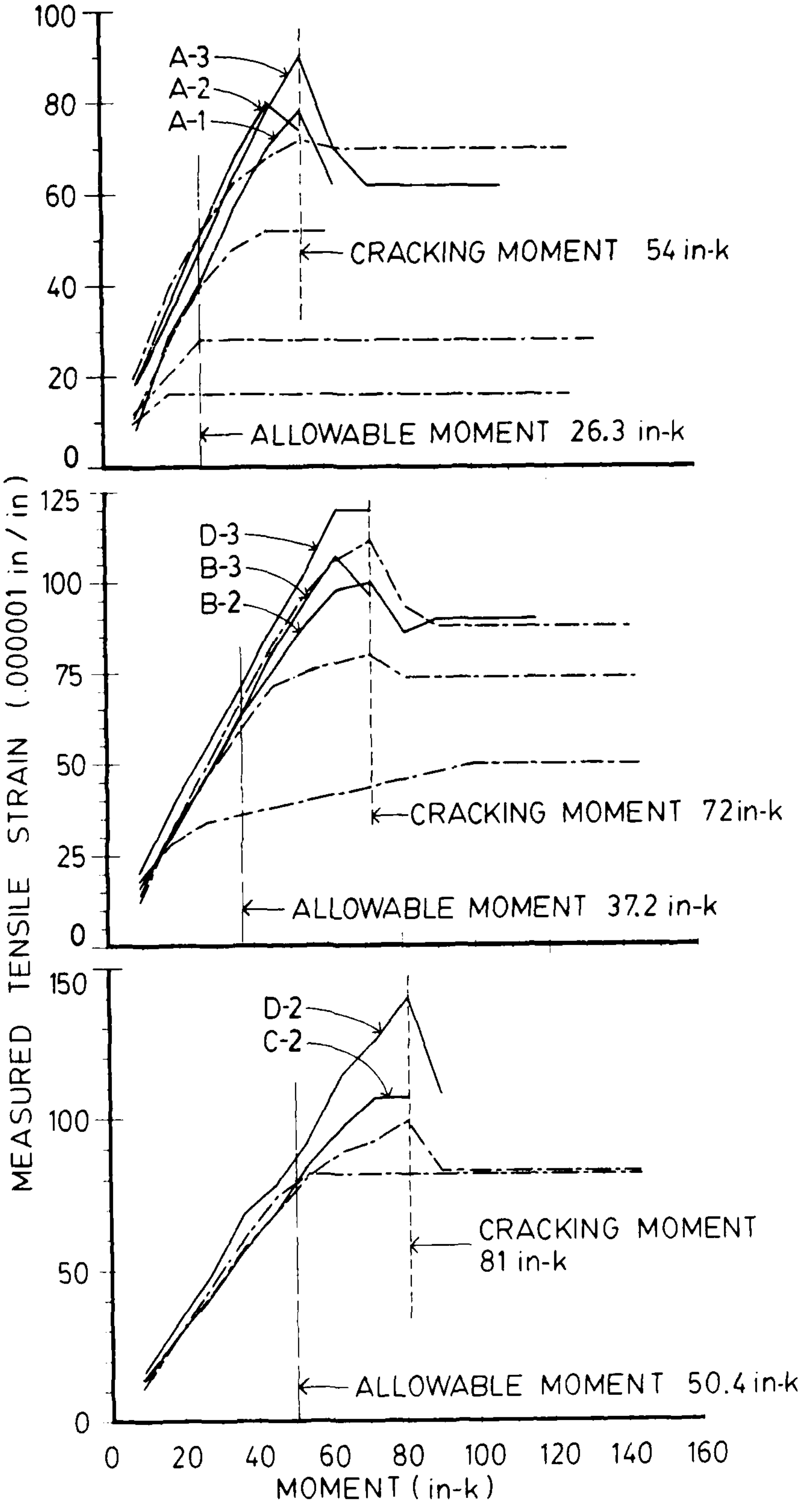


Figure 6.9. Comparison of Measured Tensile Strains for Post-Tensioned Masonry Wall Panels.

develop in the masonry and increase until the modulus of rupture is reached. The behavior of the conventionally reinforced masonry wall panel, which apparently changed from that of an uncracked, elastic member to that of a cracked member at a moment of 27 in-k, tends to support this belief. The change in behavior of the conventionally reinforced wall panel, based on visual observation of both the wall panel and the strain gauge indicator, appeared to be sudden. The cracking of both the masonry and the grout appeared to occur at the same time. This is not necessarily true of the post-tensioned panels, as evidenced by their behavior during the second loading cycle. During the second loading cycle, with the exception of panel D-2, all of the post-tensioned wall panels which were initially loaded to the cracking moment exhibited an increase in tensile strain after allowable moment had been reached. This was generally accompanied by a gradual change in both deflection and compressive strain. Since the strain gauges were mounted on the concrete masonry units, it can be reasonably assumed that a decrease in tensile strain is indicative of cracking in the mortar joint. What is not clear, however, is the extent to which the grout had cracked, if it had cracked at all, when the decrease in tensile strain was noted. In any case, the data suggest that the design procedure was adequate for the panels tested.

The test data also confirm the accuracy of the procedure used to compute the ultimate moment. The ultimate

moment of the post-tensioned wall panels could not, for reasons discussed earlier, be accurately determined, but occurred at a moment somewhat in excess of 144 in-k. This is only slightly greater than the ultimate moment of 141 in-k predicted by the Whitney type procedure cited by Sahlin. The procedure also proved to be accurate in predicting the ultimate moment of 45 in-k for the conventionally reinforced wall panel.

6.6.2 Shrinkage and Creep

Two of the post-tensioned panels, panels B-1 (12,000 lb. post-tensioning force) and C-3 (23,000 lb. post-tensioning force), and the non-post-tensioned panel, panel D-3, were measured for time dependent deformation due to shrinkage and creep. Figure 6.10 shows the deformation of the three wall panels for a 90-day period following post-tensioning.

The panels were stored indoors; however, neither the temperature nor the relative humidity was controlled in the space. Deformation was measured with a Whittemore Extensometer, and a temperature compensation bar was used to adjust data. The deformation of the masonry as depicted in Figure 6.10 is assumed to be due to creep and changes in humidity. Due to time constraints, it was not possible to measure shrinkage and creep for more than 90 days. It is realized that the data does not represent a true indication of the magnitude of time dependent deformation, but it does

provide some insight into the behavior of the post-tensioned wall panels.

All of the panels expanded following grouting. This was the result of the masonry units absorbing water present in the grout. Panel B-1 expanded by 114 x 10^{-6} in/in, and panel C-3 by 91 x 10^{-6} in/in. This is greater than the elastic shortening due to post-tensioning of 54 x 10^{-6} in/in for panel B-1 and 74 x 10^{-6} in/in for panel C-3. In order to minimize the amount of post-tensioning force required, the masonry should be post-tensioned prior to grouting. Grouting causes the masonry to expand, increasing the tensile stress in the steel and the compressive stress in the masonry. The design procedure should, therefore, take this expansion into consideration. Figure 6.10 also indicates that as the post-tensioning force is increased, the amount of expansion due to grouting is decreased.

Between days 28 and 81, the deformation of panel D-1 remained fairly constant, while panels B-1 and C-3 shortened at an essentially identical linear rate, evidently due to creep. Between days 81 and 90, panel B-1 exhibited an increase in length of 1 x 10^{-5} in/in from the 81 day length, while panel C-3 exhibited an increase of 1.5 x 10^{-5} in/in from the 81-day length. Panel D-1 showed a decrease in length of 3 x 10^{-6} in/in during this same period. This behavior cannot be explained, and may be due to inaccuracies in data collection.

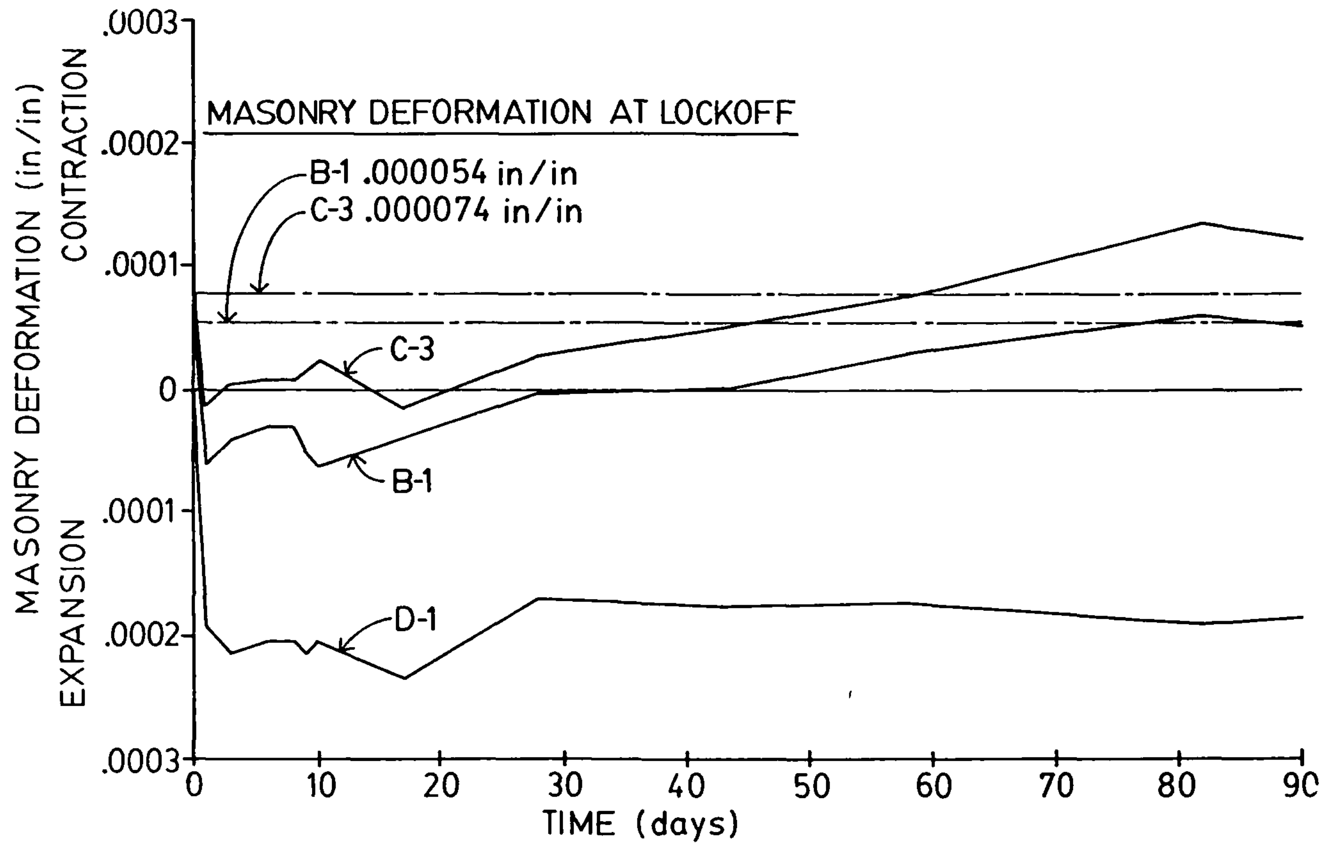


Figure 6.10. Test Data, Shrinkage and Creep.

The rate of creep in concrete masonry generally decreases with age and eventually ceases. The age at which this occurs is not known, and the data does not provide any insight into the creep behavior of the post-tensioned panels beyond the 90-day monitoring period. Between days 28 and 90, panel D-1 exhibited no shortening due to shrinkage, suggesting that the expansion of approximately 2×10^{-4} in/in due to grouting may be permanent. The reasons for this are unclear, and the extent to which this affects the behavior of the post-tensioned panels cannot be determined. Based on the available data, it is not possible to predict the magnitude of deformation of the masonry due to shrinkage and creep.

CHAPTER 7

CONSTRUCTION

7.1 Reinforced Masonry

Reinforced concrete masonry is a method of construction where hollow concrete block are laid up in mortar so that their alignment forms a series of vertical cavities within the wall. Steel reinforcing bars are placed in these cavities, which are than filled with grout to form a bonded composite structural system. In the construction of reinforced masonry walls containing large amounts of closely spaced reinforcement, the cavities are created by using open-end concrete masonry units. Figure 7.1 shows such a unit, which is essentially a two-core unit without one of the end cross-webs. The steel reinforcement is tied to dowels in the foundation, and the open-end blocks are threaded around the unit as the wall is erected.

Grouting can be accomplished through either the "low-lift" or "high-lift" procedure. In "low-lift" grouting, the masonry is erected in four-feet-high increments. Each increment is filled with grout prior to the erection of the next four-feet-high increment of masonry. The advantage to low lift grouting is that it requires no special equipment, and is relatively simple. In "high-lift" grouting, the wall is constructed to its full height, and a grout pump is used to fill the cores with grout. Grout is pumped into the cavities in four-feet-high lifts and vibrated to insure that all voids in the masonry are filled.

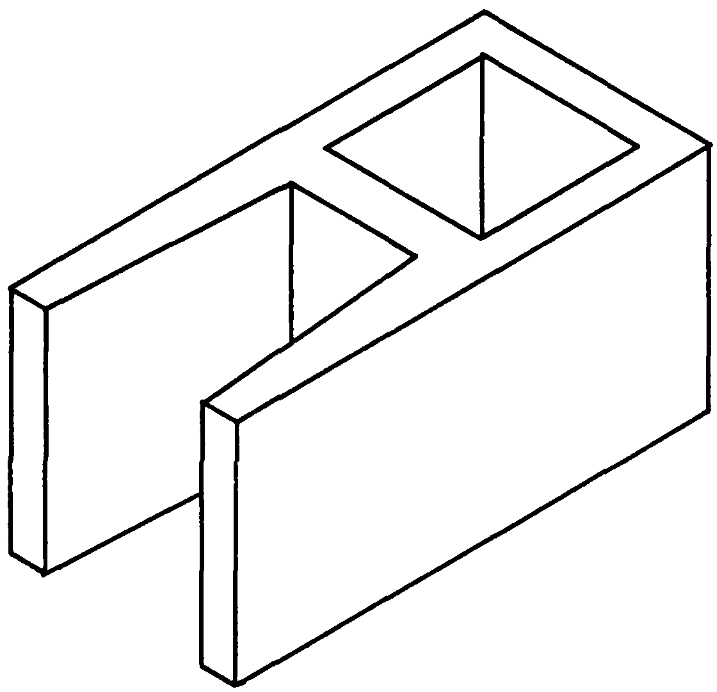


Figure 7.1. Open End Concrete Masonry Unit.

Because water migrates from the grout into the masonry units, the grout shrinks after being placed in the cavities. In order to compensate for this, a 30- to 60-minute delay is required between placement of successive lifts.

7.2 Post-Tensioning Masonry

A post-tensioned masonry wall could be constructed using readily available masonry materials and the high-lift grouting procedure. While reinforced and post-tensioned masonry walls would be constructed of the same masonry materials, provisions must be made for anchorages and proper placement of the post-tensioning rod within the wall.

7.2.1 Foundation Anchorage

Figure 7.2 shows a typical foundation anchorage. The anchorage consists of a steel plate with a hole through which a short length of threaded post-tensioned rod is placed. Nuts are threaded against both sides of the plate in order to hold the rod in position. After the concrete footing is placed, the anchorage is properly positioned and inserted into the footing. Care must be exercised in properly levelling the plate. After the concrete has set, the exposed nut is removed, and erection of masonry can begin.

7.2.2 Bearing Blocks

A typical bearing block is shown in Figure 7.3. The block is made by filling a three-core masonry unit with grout and forming a hole in the middle of the center core. The application of the post-tensioning force causes high

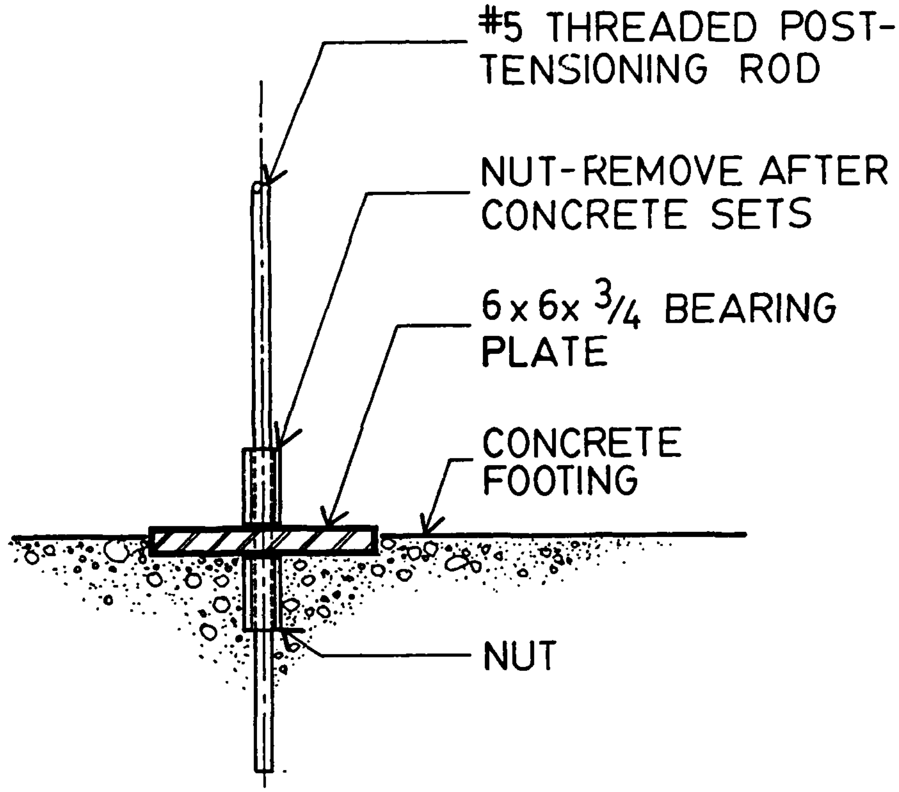


Figure 7.2. Foundation Anchorage.

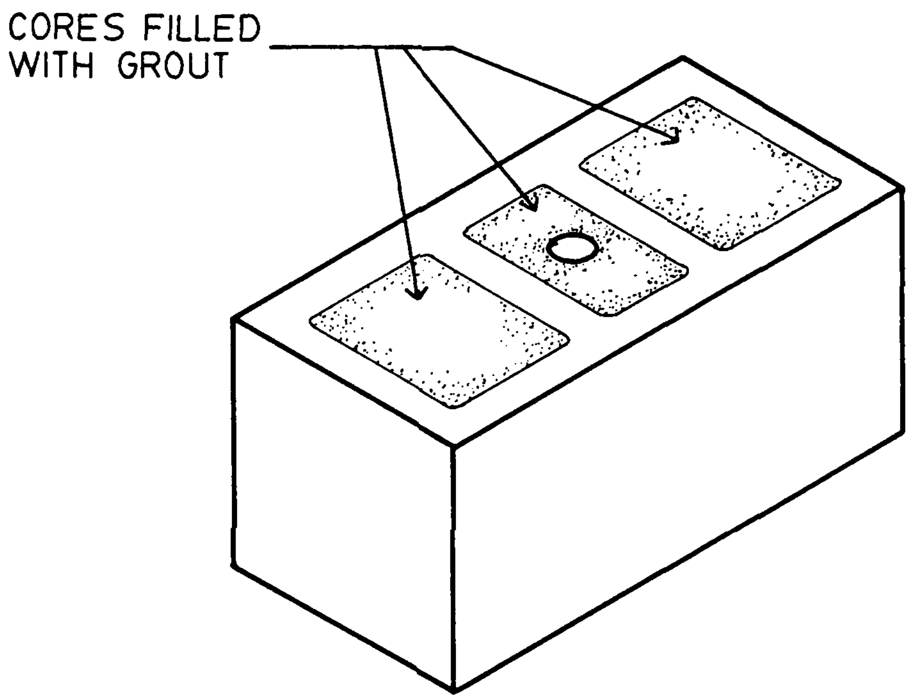


Figure 7.3. Bearing Block.

bearing stresses to develop in the masonry. By using a solid block the masonry area is increased, resulting in lower bearing stresses. The bearing block serves another purpose, which is to properly position the first course of masonry with respect to the anchorages. The bearing blocks are placed on the anchorage and set in a bed of mortar. Masonry units are then set between the bearing blocks until the first course is completed. The cavity which will contain the post-tensioning rod is constructed with open end masonry units, as shown in Figure 7.4

The post-tensioning rods are connected together by means of coupling nuts. The post-tensioning rod will be placed in the cavity after the wall is constructed, and in order to enable the rod to be connected to the anchorage, an access port must be provided. This is accomplished by removing a portion of the face shell from one of the units forming the cavity and replacing it following placement of the post-tensioning steel.

7.2.3 Position Blocks

Figure 7.5 shows a typical position block, which is made from a three-core masonry unit. The position block is necessary to prevent buckling of the wall under application of the post-tensioning force, and does so by bringing the post-tensioning rod into contact with the masonry at points between the top and bottom of the wall. The vertical spacing of position blocks would depend on the height to thickness ratio of the wall. Determination of this vertical

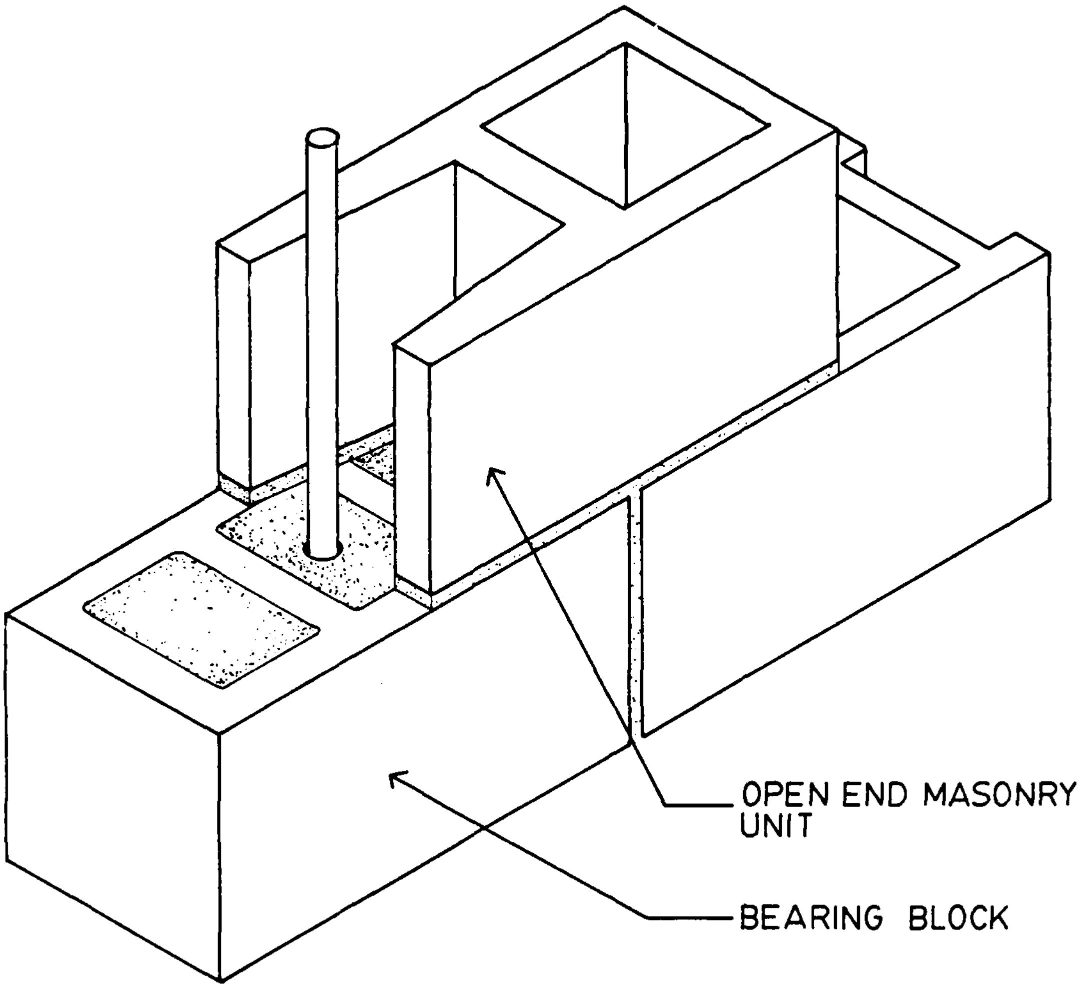


Figure 7.4. Detail of Masonry at Foundation Anchorage.

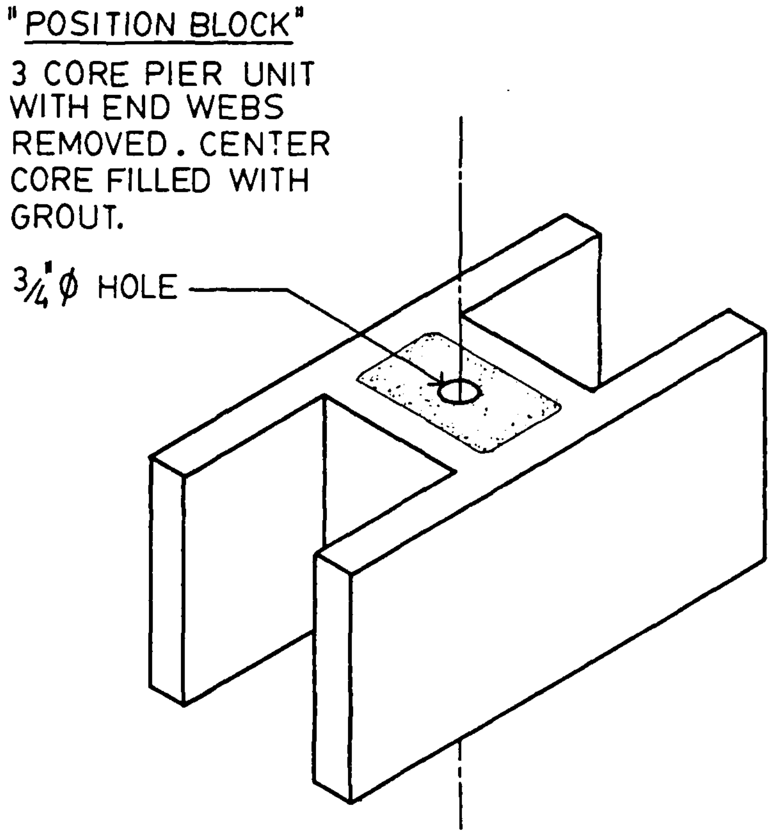


Figure 7.5. Position Block.

spacing is beyond the scope of this study, but it appears that at a minimum one position block is required at mid-height of the wall. Figure 7.6 shows a typical arrangement of open end masonry units and a position block. An access port must be provided above each position block in order to enable the post-tensioning rod to be passed through the hole.

7.2.4 Top of Wall Anchorage

The anchorage at the top of the wall is shown in Figure 7.7. The anchorage consists of a bearing block and steel plate. The masonry units immediately below the bearing block are 2-inch-thick concrete masonry tiles. Because no cross-webs are present, the cavity is accessible, and can be filled with grout.

7.2.5 Post-Tensioning

Following erection of the wall, the post-tensioning rod is placed in the cavity and connected to the foundation anchorage. The access ports are then sealed, and the wall post-tensioned, using the jacking table and hydraulic jack as shown in Figure 7.8. Following post-tensioning, the wall is grouted using the high-lift grouting procedure described earlier. Following grouting, the top course of concrete masonry units is set in place.

7.3 Cost

Reinforced masonry and post-tensioned walls can be constructed of the same masonry materials. The cost of a post-tensioned wall would have to include the cost of the

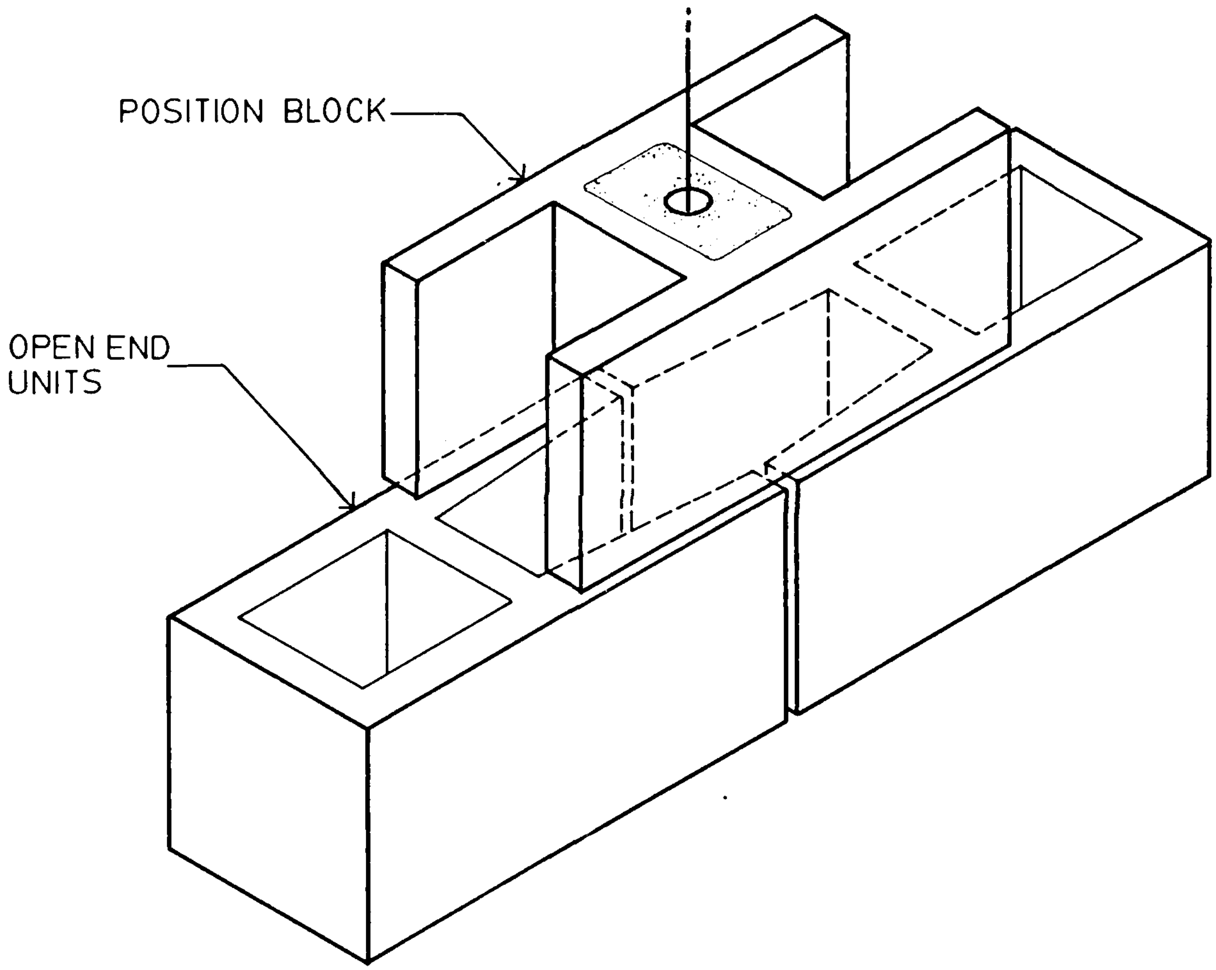


Figure 7.6. Detail of Position Block and Cavity.

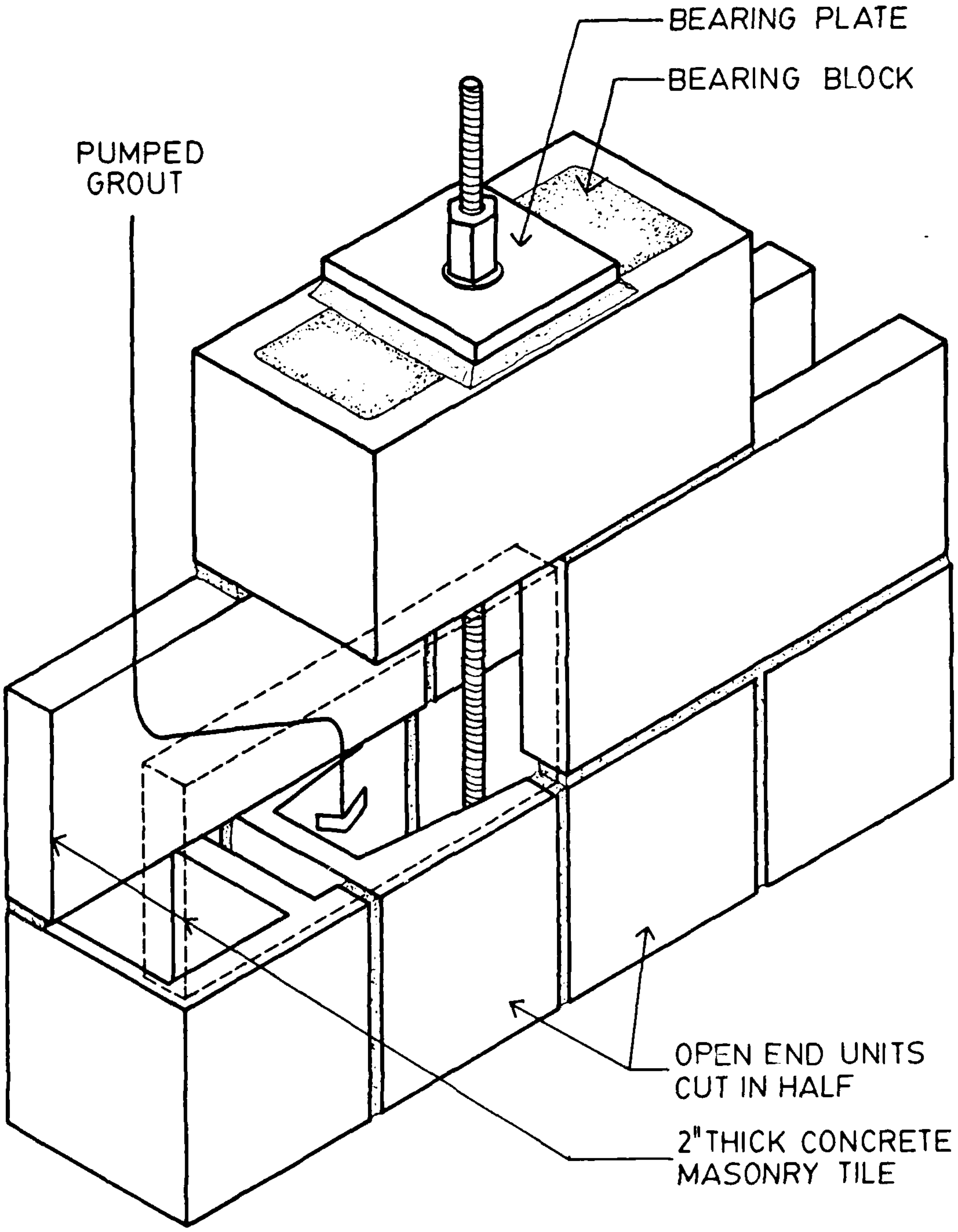


Figure 7.7. Top of Wall Anchorage.

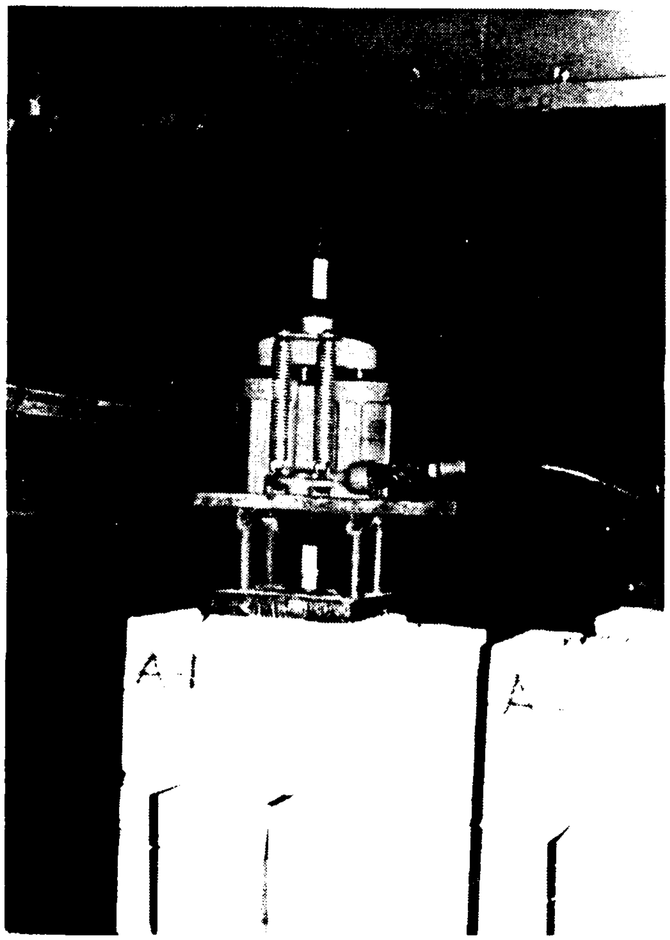


Figure 7.8. Hydraulic Jack and Jacking Table on Wall Panel.

post-tensioning steel, anchorages, and labor required to manufacture the position blocks and bearing blocks. In addition, construction of the post-tensioned wall would require increased supervision in order to insure that the position and bearing blocks are properly positioned within the wall. These additional costs would rule out the use of post-tensioned masonry on most small jobs.

In situations where walls are subjected to large lateral loads caused by seismic or wind forces, post-tensioning may be economically feasible because it permits a reduction in the thickness of the masonry. To illustrate this point, consider a masonry wall which must be designed to resist a moment of 60 in-k. A reinforced masonry wall designed for this moment would have to be 12 inches thick with #6 reinforcing rods at 12 inches on center. A post-tensioned masonry wall would be 8 inches thick, and would use #5 threaded post-tensioning rods spaced at 16 inches on center. The calculations for both walls are shown in Appendix A. It is not possible to perform a detailed cost comparison because the additional costs involved in the construction of the post-tensioned wall would be dependent on the size of the job, the height of the wall, and the method used to manufacture the position and bearing blocks. On a large job, the savings resulting from the use of 8 inch masonry units in lieu of 12-inch masonry units would be significant, and could offset the additional cost of post-tensioning.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

1. The post-tensioning of walls constructed of conventional concrete masonry is feasible.

2. Post-tensioned walls have greater flexural strength than reinforced masonry walls constructed of the same masonry.

3. The equation $f = (P/A) \pm (M/S)$ can be used as the basis for design of post-tensioned masonry walls.

4. Post-tensioned masonry behaves linearly and elastically to the cracking moment. The cracking moment is greater than the allowable moment by an amount approximately equal to the cracking moment of a reinforced masonry wall.

5. A post-tensioned wall can be constructed of commonly available masonry materials. Masonry units which properly position the block within the wall must be fabricated. Care must be used in the construction of a post-tensioned wall to insure that masonry units which position the post-tensioning rods are properly placed. The cavity containing the post-tensioning rod must be constructed so that it can be filled with grout after the wall has been post-tensioned.

6. Post-tensioning can be accomplished with simple tools and by relatively inexperienced personnel.

7. The cost of a post-tensioned masonry wall is higher than the cost of a reinforced masonry wall. Where

reinforced masonry walls contain a large amount of closely spaced reinforcement, post-tensioning may permit a reduction of wall thickness. On a large project, the savings achieved through the use of thinner masonry could offset the additional costs due to post-tensioning.

8.2 Recommendations

1. The wall panels tested in this study were constructed in a stack bond. In actual practice, a running bond would be used. The effects of post-tensioning a wall constructed in a running bond should be investigated.

2. The behavior of masonry walls post-tensioned with large diameter post-tensioning rods spaced relatively far apart should be examined.

3. In actual practice a masonry wall would be post-tensioned shortly after the masonry has been erected. In order to permit this, the mortar would have to reach a high strength at an early age. The use of mortar mixed with Type III Portland in a post-tensioned wall should be investigated.

4. The post-tensioning of masonry constructed of high strength masonry units and various types of high strength mortar should be examined.

5. This study did not provide a clear indication of the deformation of the masonry due to grouting, shrinkage and creep. Deformation of the masonry is an important consideration in the design of post-tensioned masonry, and should be investigated further.

6. The design procedure used in this study was somewhat rudimentary. If post-tensioned masonry is to be used in actual construction practice, the design procedure must be further developed.

APPENDIX A

ALLOWABLE MOMENTS

Allowable Moments for Post-Tensioned Wall Panels

$A = 68.5 \text{ in}^2$ $\qquad f = \frac{P}{A} \pm \frac{M}{S}$

$S = 150 \text{ in}^2$

1. 12,000 lb. post-tension force

$f_t = 0 = \frac{12{,}000}{68.5} \; \frac{M}{150}$; $\quad M = 26{,}277$ in-lbs. say 26.3 m-k

$f_c = \frac{12{,}000}{68.5} + \frac{26{,}277}{150} = 350$ psi

2. 17,000 lb. post-tension force

$f_t = 0 = \frac{17{,}000}{68.5} \; \frac{M}{150}$; $\quad M = 37{,}226$ m-lbs. say 37.2 m-k

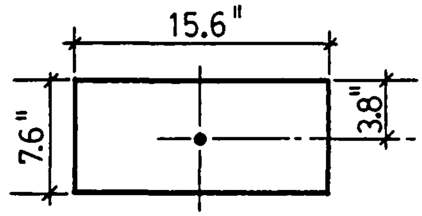
$f_c = \frac{17{,}000}{58.5} + \frac{37{,}226}{150} = 496$ psi

3. 23,000 lbs. post-tension force

$f_t = 0 = \frac{23{,}000}{68.5} - \frac{M}{150}$; $\quad M = 50{,}365$ in-lbs. say 50.4 in-k

$f_c = \frac{23{,}000}{68.5} + \frac{50{,}365}{150} = 672$ psi

Allowable Moment for Reinforced Masonry Wall Panel



f_s=20,000 psi A_s=.31 in.^{2}

f'_m=2829 psi F_b=.33f'_m=934psi

USE 900 psi max per code.

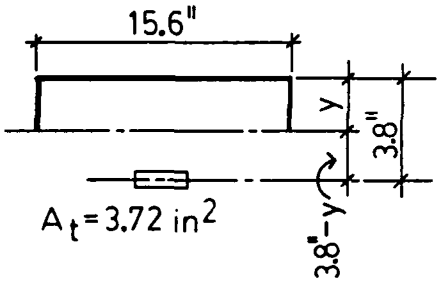
E_s=29,000,000 psi,

E_m=1000 f'_m=2,829,000 psi

USE 2,500,000 psi max per code.

$$n = \frac{29,000,000}{2,500,000} = 11.6, \text{ say } 12$$

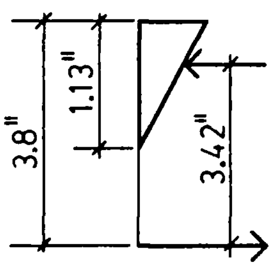
$$A_t = nA_s = 12(.31) = 3.72 \text{ in.}^2$$



$$(15.6y)\left(\frac{y}{2}\right) = (3.8-y)(3.72)$$

$$0 = 7.8y^2 + 3.72y - 14.14$$

$$y = 1.13"$$



$$M_m = \frac{900}{2}(15.6)(1.13)(3.42)$$

$$= 27,129 \text{ in.-lbs.}$$

$$M_s = (.31)(20,000)(3.42)$$

$$= 21,204 \text{ in.-lbs.} \quad \text{(GOVERNS)}$$

APPENDIX B

ULTIMATE MOMENTS

Ultimate Moments for Masonry Wall Panels

$m = q(1-0.59q) \quad q = \frac{\rho fy}{f'_m} \quad \rho = \frac{A_s}{bd}$

$M = mbd^2f'_m$

Post-Tensioned Wall Panels

$b = 15.6$ in. $\quad f'_m = 2829$ psi $\quad A_s = .28$ in^2

$d = 3.8$ in. $\quad f_y = 157{,}000$ psi

1. $\rho = \frac{A_s}{bd} = \frac{.28}{15.6(3.8)} = .00472$

2. $q = \frac{\rho fy}{f'm} = \frac{.00472(157{,}000)}{2829} = .2621$

3. $m = q(1-0.59q) = .2621(1-0.59(.2621)) = .2216$

4. $M = mbd^2f'_m = (.2216)(15.6)(3.8^2)(2829)$

$= 141{,}197$ in.-lbs. (141.1 in-k

Reinforced Masonry Wall Panel

$b = 15.6$ in. $\quad f'_m = 2829$ psi $\quad A_s$ .31 in.^{2}

$d = 3.8$ in. $\quad f_y = 40{,}000$

1. $\rho = \frac{A_s}{bd} = \frac{.31}{15.6(3.8)} = .00523$

2. $q = \frac{\rho fy}{f'm} = \frac{(.00523)(40{,}000)}{2829} = .0739$

3. $m = q(1-0.59q) = .0706$

4. $M = mbd^2\ f'_m = (.0706)(15.6)(3.8^2)(2829) =$

$= 45{,}040$ in.-lbs. (45 in-k)

DESIGN REINFORCED MASONRY WALL FOR MOMENT OF 60 in-k PER FOOT OF WALL

DESIGN BASED ON NCMA TR75-B-1970

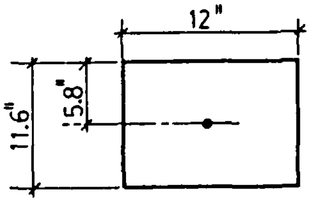
f_s=20,000 psi, f_s=20,000 x 1.33

for wind or earthquake = 26,600 psi

f'_m=2829, F_b=.33(2829) = 933 psi

max per code = 900 psi

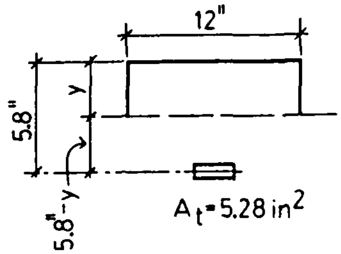
F_b = 900 psi x 1.33 for wind or earthquake = 1200 psi



E_s=29,000,000, E_m = 2,500,000, n=12

TRY 12" CMU grouted solid w/#6@12" OC.

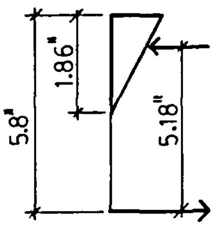
A_s = .44 in^2, A_t = nA_s = 5.28 in^2



$$12_y \left(\frac{y}{2}\right) = 5.28(5.8-y)$$

$$0 = 6y^2 + 5.28y - 30.62$$

$$y = 1.86"$$



$$M_m = \frac{1200}{2}(12)(1.86)(5.18)$$

$$= 69,370 \text{ in.-lbs.}$$

$$M_s = 26,600(.44)(5.18)$$

$$= 60,627 \text{ in.-lbs.} \quad \text{GOVERNS}$$

Design of Post-Tensioned Wall for Moment of 60 in-k per foot of Wall

Assume 8 in. CMU, face shells bedded in mortar, wall grouted solid after post-tensioning.

$$S = \frac{bh^2}{6} = \frac{12(7.6^2)}{6} = 116 \text{ in}^3$$

$$A = 12''(2)(1.25)'') = 30 \text{ in.}^2$$

$$f = \frac{P}{A} \pm \frac{M}{S}; \text{ for } f_t=0 \text{ at } M=60 \text{ in-k, } P= \frac{60,000}{116}(30)=15,517 \text{ lbs.}$$

$$\text{Compressive stress due to post-tensioning} = \frac{15,517}{30} = 517 \text{ psi}$$

$$f_c \text{ at } M = 60 \text{ in-k} = \frac{15,517}{30} + \frac{60,000}{116} = 1014 \text{ psi} < 1200 \text{ psi} \therefore \text{OK}$$

Assume shrinkage and creep losses = 2 x elastic deformation

$E_m = 2,500,000$ psi

$$\varepsilon_m \text{ prestress} = \frac{f}{E} = \frac{517}{2,500,000} = .000206 \text{ in./in.}$$

$E_s = 29,000,000$ psi   Assume #5 post-tension rod, A_s=.28 in.^{2}

$$f_s \text{ prestress} = \frac{15,517}{.28} = 55,418 \text{ psi;}$$

$$\varepsilon_s \text{ prestress} = \frac{55,418}{29,000,000} = .00191 \text{ in./in.}$$

$$\varepsilon_s \text{ req'd} = \varepsilon_s \text{ prestress} + 2\,\varepsilon_m \text{ creep} = .00191 + .000412$$
$$= .002323 \text{ in./in.}$$

$$f_s \text{ req'd} = .002323(29,000,000) = 67,366 \text{ psi}$$

$$P_{req'd} = 67,366\ (.28) = 18,862 \text{ lbs.}$$

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