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Influence of Vertical Compressive Stress on Shear Resistance of Concrete Block Masonry Walls

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NATIONAL BUREAU OF STANDARDS, Ernest Ambler, *Director*

ABSTRACT

The results from tests on eight ungrouted and unreinforced concrete block masonry walls are presented. The emphasis of the research program is on the influence of vertical in-plane compressive stress on the lateral in-plane load resistance of shear-dominated walls. Each wall has nominal dimensions of 64 in. x 64 in. x 8 in. and is fabricated from similar materials by the same experienced mason. The masonry units are hollow concrete block having a nominal compressive strength of 1800 psi based on the gross area. The mortar is proportioned as a Type S. The walls are tested in the NBS Tri-directional Testing Facility using fixed ended boundary conditions at the top and bottom of the wall. Lateral in-plane displacements are applied at the top of the wall while maintaining a constant compressive vertical stress. The vertical compressive stress varies between 120 and 500 psi (based on net cross-sectional area) in the test program. The test data indicate that the maximum in-plane lateral load resistance exhibited by the walls increases with increased vertical compressive stress. The relationship between increasing vertical compressive stress and the resulting increasing maximum lateral load resistance appears to be linear within the range of vertical stresses used in the test program.

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1. INTRODUCTION

1.1 BACKGROUND

This interim report is the first in a series of reports which document an experimental investigation undertaken as part of an overall program of research on masonry walls. The purpose of this and other reports in the series is to present the results to researchers, designers, and code writers in a timely manner. Detailed data analysis and interpretation are omitted from this report. Instead, the analysis and interpretation of the data are presented in summary reports. The summary reports are issued periodically as sufficient data become available to more fully address a particular issue. Generally, summary reports are based on the results presented in several interim reports since several test series are required to fully explore a parameter and its relation to other parameters.

1.2 OVERVIEW OF MASONRY RESEARCH PROGRAM

The principal thrust of the overall program of research is directed towards defining the shear capacity (in-plane lateral load resistance) and behavior of shear-dominated masonry walls. The prediction of shear capacity and behavior of masonry has been identified as an area in which there is a serious deficiency of supporting research. The NBS/BSSC review committee for the ATC3-06 masonry design provisions [1] suggest that research is needed to substantiate and improve the current design recommendations for shear capacity.

The main variables which are to be investigated in the NBS masonry research program are axial compressive stress, aspect ratio (wall length-to-height), masonry type, mortar type, grout, vertical and horizontal reinforcement, out-of-plane loadings, and loading history. Analytical studies are coordinated with the experimental investigations so that a predictive model can be developed for defining the shear capacity and behavior of a masonry wall. The predictive model will lead to improved design standards, but in the interim the experimental test results will aid in substantiating and improving the current design provisions for shear in masonry walls.

1.3 SCOPE

This report presents data from tests of eight masonry walls subjected to in-plane lateral displacements in combination with various vertical (axial) compressive stress levels. All eight walls were square, of the same size, and made from similar concrete block and mortar. The only parameter intentionally varied between tests was the axial compressive stress maintained on the wall during in-plane loading. The wall panels had nominal dimensions of 64 in. x 64 in. x 8 in. The concrete block used in the wall panels was hollow block having a unit compressive strength of approximately 1800 psi based on the gross area. The mortar was proportioned as Type S mortar.

The materials and their properties are described in chapter 2. The wall panels and accompanying prisms are described in chapter 3. The wall panel test setup, instrumentation, and test procedure are discussed in chapter 4. Representative test results are presented in chapter 5 while limited data interpretation is discussed in chapter 6. A summary is presented in chapter 7 along with conclusions based on the data presented in this report.

2. MATERIALS

All materials used in constructing the wall panels and associated prisms were commercially available and were representative of those commonly used in building construction.

2.1 CONCRETE MASONRY UNITS

Two concrete masonry unit shapes were used in the construction of the wall panels:

1. 8 in. x 8 in. x 16 in., 2 core hollow strecher block.

2. 8 in. x 8 in. x 16 in., 2 core hollow kerfed corner block with a steel sash groove in one end.

The dimensions represent nominal sizes. Typical measured dimensions and physical characteristics of the units are presented in table 2.1. The measurements are made in accordance with the procedures set forth in ASTM C140 [2]. The units are illustrated in fig. 2.1. The half blocks at each end of alternating wall courses are made by sawing kerfed corner blocks in half through the kerf. Both halves produced by this procedure are used in the wall panels.

All of the concrete masonry units used in the wall panels and prisms were manufactured on the same day by a commercial block manufacturer. The mixture proportions were set to produce a unit having an ultimate compressive strength of 2000 psi measured over the gross area of the unit. The mixture proportions were:

1950 lbs lightweight expanded shale aggregate
1250 lbs sand
260 lbs portland cement
190 lbs NewCem

NewCem is the proprietary name for a very finely ground water granulated blast furnace slag manufactured by Atlantic Cement Co., Inc. and is a partial replacement for portland cement. It meets the requirements of ASTM C989, grade 120 [3] and when blended within the range of 25 to 65 percent with portland cement, meets the requirements of ASTM C595 [4]. The preceeding description of NewCem is presented only for purposes of information and is not an endorsement of the proprietary product. The mixture used in producing the units makes 115 units with 3.91 lbs of cementitious materials per unit.

2.2 MORTAR

One type of mortar was used in constructing all of the wall panels and prisms. The mortar was a portland cement-lime mortar that was proportioned within the limits of a Type S mortar according to the specifications of ASTM C270 [5]. The materials used in the mortar were:

1. Sand - a natural bank sand that was dug locally with its primary use being for masonry mortar. Sieve analyses were performed on the sand

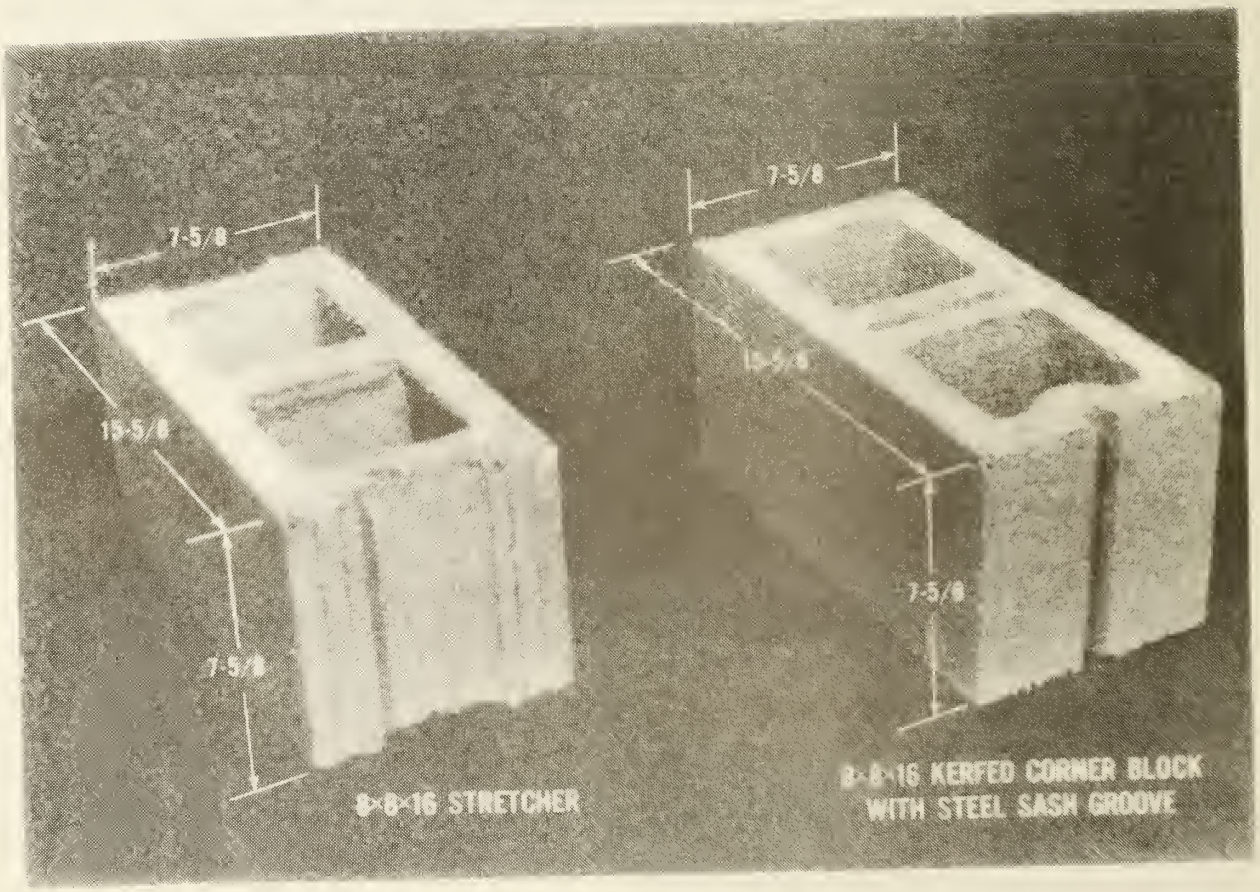

Figure 2.1 Concrete block units

Table 2.1 Dimensions and Properties of Concrete Masonry Units

	Hollow* Stretcher	Hollow* Corner/Sash Groove
Width (in.)	7.63	7.63
Height (in.)	7.59	7.57
Length (in.)	15.62	15.64
Minimum Face Shell Thickness (in.)	1.30	1.30
Gross Area (in.^{2})	119.2	119.3
Net Solid Area (in.^{2})	61.5	67.1
Gross Ultimate Compressive Strength (psi)	1813	1795
Density (lb/ft^3)	102.4	104.5
Absorption (lb/ft^3)	10.8	10.2

*Average of measurements from 6 units.

upon delivery. The analyses were done according to the specifications in ASTM C144 [6] and the results appear in table 2.2. The fineness modulus was 1.57.

2. Portland cement - a commercially available, bagged, 94 lbs per bag, Type I portland cement identified as meeting the specifications of ASTM C150 [7].

3. Lime - a commercially available, bagged, 50 lbs per bag, hydrated lime, Type S, identified as meeting the specifications of ASTM C207 [8].

These materials were proportioned 1:3/8:4 with 1 part by volume of cement, 3/8 part by volume of lime, and 4 parts by volume of sand. The parts were mixed in a typical motorized mortar mixer (fixed horizontal drum with rotating blades) for a period of not less than three minutes after all cement, lime, sand, and most of the water was added. Finally, small amounts of water were added to produce mortar of a consistency acceptable to the mason.

Immediately upon leaving the mixer, the time was recorded and a sample was taken for determining the initial flow rate. The air content of the mortar was measured for selected mortar batches. Six mortar cubes (2 in. x 2 in. x 2 in.) were made during the early part of the wall panel construction. After completing the wall panel, the mason constructed three prisms. Thus, each batch of mortar produced a wall panel, six mortar cubes, and three prisms. Retempering of the mortar, if required, was permitted only once per batch.

Table 2.2 Masonry Sand Sieve Analysis*

Screen Size Number	Cumulative Percent Retained
4	0.1
8	0.6
16	2.0
30	11.0
50	56.9
100	86.6
100+	....

Total 157.2 ÷ 100 = 1.57 Fineness Modulus

* Average of three samples taken upon delivery of sand.

3. WALL PANEL DESCRIPTION

Eight wall panels were fabricated using concrete block taken from the same lot and mortar having the same proportions of cement, lime, and sand. As companions to each wall six mortar cubes (2 in. x 2 in. x 2 in.) were made and three prisms were fabricated. The companion specimens were tested to provide information on mortar compressive strength and wall panel compressive strength.

3.1 WALL PANEL FABRICATION

The wall panels were constructed in running bond with 50 percent overlap of block in alternate wall courses (fig. 3.1). The wall panels had overall nominal dimensions of 64 in. x 64 in. x 8 in. The wall panels (and prisms) were constructed by an experienced mason using techniques representative of good workmanship. The wall panels were fabricated in a controlled environment laboratory from materials stored in the same environment for at least 30 days. The temperature and relative humidity of the laboratory were maintained at approximately 73°F and 50 percent, respectively.

The bottom course of block was laid on a steel beam (channel) section without bedding mortar. The steel beam and first course were then leveled using shims as necessary. The first block laid in the bottom course was a whole kerf unit with no head joint mortar. Head joints were subsequently formed by buttering the end of the next block to be laid with mortar. The head joint mortar was only placed as deep as the face shell thickness. All head joints were "shoved" joints with no closure units or backfilling of head joints. The mortar bed joints were formed by placing mortar along the face shells of the previously laid course of blocks. No mortar was placed on the cross-webs except for the end cross-webs. Each course was laid to maintain a course height of 8 in. The level of each course was fixed by a level string spanning between two plumb posts. The end blocks were plumbed using a 4-foot level to maintain plumb end surfaces of the wall panel. All joints were struck flush with a trowel, but not tooled.

3.2 PRISM FABRICATION

Three prisms were made along with each wall panel using mortar from the same batch as was used for the wall panel. Each prism was made by stack bonding three stretcher units (figure 3.2). The mortar bedding between the blocks was either face shell only or full area bedding. Within each group of three prisms the bedding was the same. The prisms constructed with seven walls had face shell bedding while the other prisms had full area bedding. The mason used a 4-foot level to maintain the level of each block and to plumb the prism.

3.3 WALL PANEL DETAILS

The details of each wall panel are listed in table 3.1. The wall panel identifier is a two part mnemonic with the two parts separated by a hyphen. That part of the identifier preceding the hyphen has the form mHHn where m and n are numbers and HH signifies that a high strength block and a high strength mortar are used. The term high is used only in a relative sense and does not imply

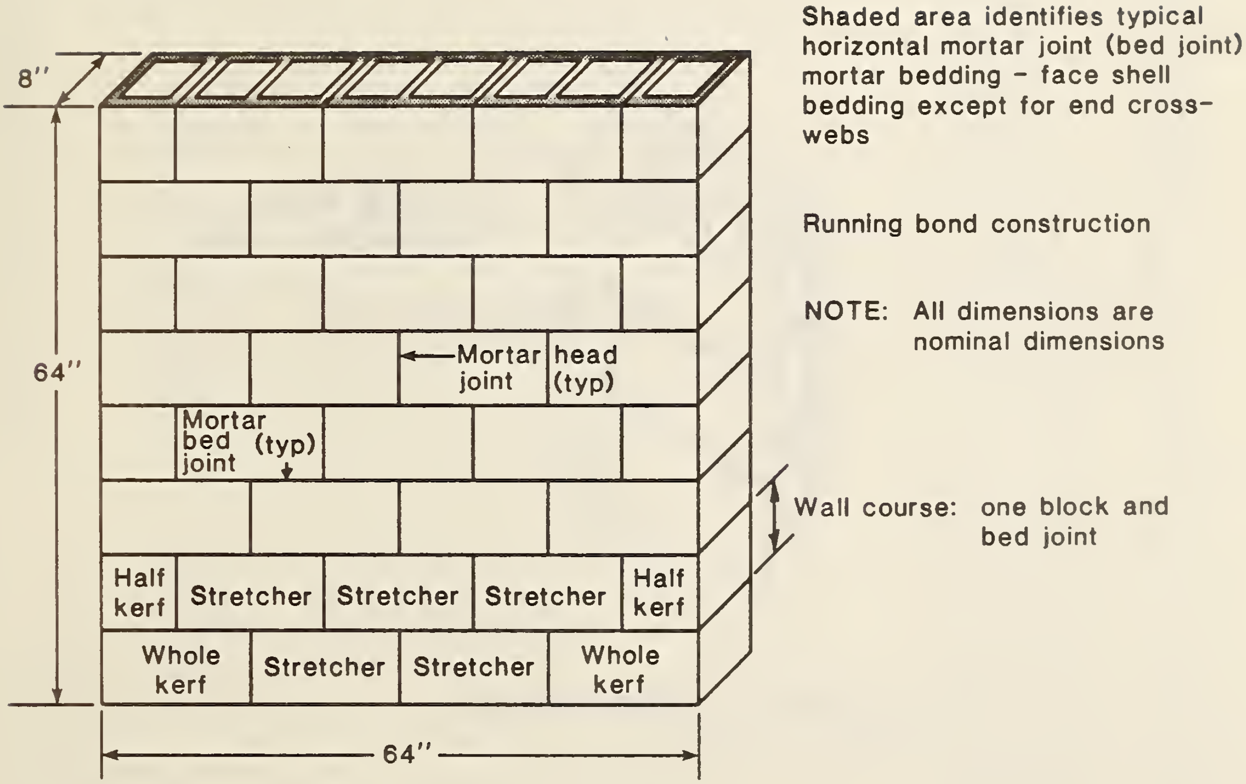

Figure 3.1 Typical wall panel

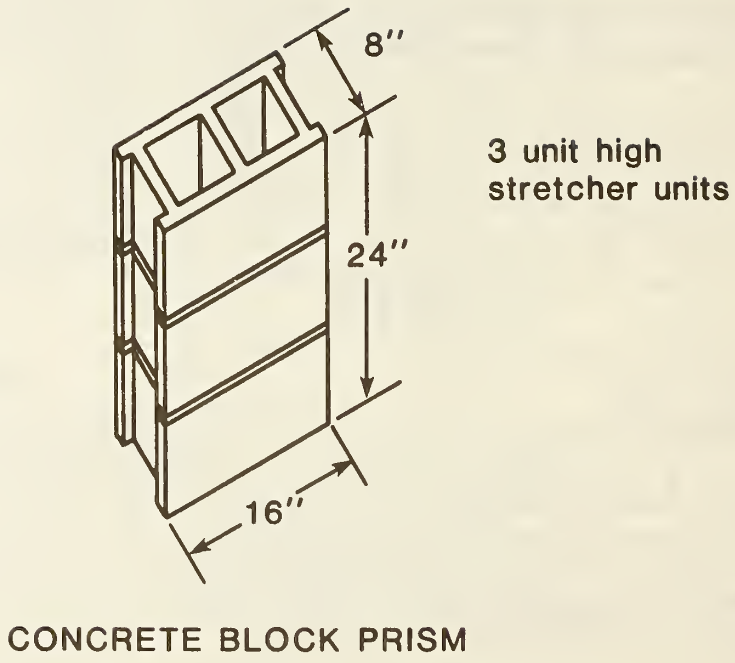

Figure 3.2 Typical prism

Table 3.1 Wall Panel Details

WALL PANEL IDENTIFIER	WALL AGE AT TEST (DAYS)	MORTAR CUBE ULTIMATE COMPRESSIVE STRENGTH* 28 DAY - (PSI) [DAYS]	PRISM BEDDING	PRISM ULTIMATE COMPRESSIVE STRENGTH** (PSI) [DAYS]
64HH120-2L04	210	2437	Face Shell	1839 [28]
64HH160-3L01	126	1825 - 2563 [154]	Face Shell	1820 [89]
64HH240-3L04	131	2237 - 3048 [154]	Face Shell	2132 [88]
64HH300-2L05	211	2160	Face Shell	1870 [28]
64HH320-3L03	128	2095 - 2890 [154]	Face Shell	2091 [85]
64HH400-3L02	127	2139 - 3066 [154]	Full Area	2810 [86]
64HH400-2L03	207	2232	Face Shell	2074 [28]
64HH500-2L06	226	2191	Face Shell	2005 [28]

* The stress is based on an area of 4 sq. in. If a single value is given then it is the average of six cube tests. If two values are given then the first value is the average of three cube tests at 28 days and the second value is the average of three cube tests at the number of days shown in the square brackets. Mortar cubes were removed from the molds after 24 hours and air cured in the laboratory environment until testing.

** Each stress value is the average of three prism tests and the stress is based on a net cross sectional area of 61.5 sq. in. (table 2.1). The number in square brackets is the age of the prisms on the day of their tests.

a specific strength. The value of m is the nominal length of the wall panel in inches and for these wall panels is always 64. The value of n is the approximate axial (vertical) compressive stress (psi-net area) applied to the wall panel in combination with the lateral displacement. That part of the identifier following the hypen is a unique sequence number assigned to the wall panel during construction. The first part of the identifier serves as a guide to the characteristics of the wall test, but may not be unique (e.g., 64HH400). The other entries in table 3.1 are self-explanatory. The ultimate compressive strength of the prisms is determined by testing the prisms in a uniaxial testing machine having a total capacity of 400,000 pounds force. A spherically seated upper bearing block covers the entire bearing surface of the prisms. The load on the prism is applied at any convenient rate for the first 40,000 pounds force while the remaining load is applied at a rate of 40,000 pounds per minute until failure occurrs. The maximum load sustained by the prism is used in computing the ultimate compressive stress.

4. WALL PANEL TESTS: SETUP, INSTRUMENTATION, PROCEDURES

4.1 TEST SETUP

The test setup (fig. 4.1) is the NBS Tri-directional Test Facility (NBS/TTF), a permanent loading apparatus designed to test building components using three-dimensional loading histories. The NBS/TTF is described in a separate report [9], but for purposes of completeness a brief summary is presented in this section.

The NBS/TTF is a computer-controlled loading apparatus which applies forces/displacements in all six degrees of freedom at one end of a test specimen. The other end of the specimen is fixed. The six degrees of freedom are the translations and rotations in and about three orthogonal axes. The application of such actions is accomplished by seven closed-loop, servo-controlled hydraulic actuators which receive their instructions by means of computer generated commands. The major components of the NBS/TTF are illustrated in fig. 4.1. The reaction system is composed of the structural tie-down floor and two vertical buttresses. The load distribution system consists of the two x-shaped steel crossheads, one at the bottom and the other at the top of the test specimen. The load application system is made up of the seven hydraulic actuators. The control system is not visible in the figure, but includes the servo-control electronics, the data acquisition equipment, and a minicomputer.

4.2 INSTRUMENTATION

The instrumentation used to monitor the test of a wall panel can be divided into two groups. The first group consists of load and displacement tranducers mounted on the hydraulic actuators. These devices measure the overall forces applied to the upper crosshead by the hydraulic actuators and the displacements of the actuator pistons as they move the upper crosshead. The second group of instrumentation directly measures the behavior of the wall panel as it is loaded. All of the instrumentation is connected to a computer-based analog-to-digital-converter which has a sample rate of 50,000 data readings per second.

The in-plane displacement of a wall panel is measured by linear varying differential transformers (LVDTs) which are displacement transducers. The LVDTs are placed as shown in fig. 4.2 with four LVDTs located on each end surface of the wall. The actual positions of the LVDTs are listed in table 4.1. The LVDTs are mounted such that they measure the displacement of the wall relative to a fixed reference (tie-down floor).

In addition to the overall measurement of the wall displacement, local measurements of displacement are made on each of the wall panel face shells. These displacement measurements are taken between points on the wall and not referenced to a fixed position. The local displacement measurements are made at the locations shown in fig. 4.3 using specified gage lengths so that strain can be computed. The displacement over the two long gage lengths is measured by LVDTs mounted on swivels between two posts glued to the wall surface (fig. 4.4). The displacement over the short gage lengths is measured by specially

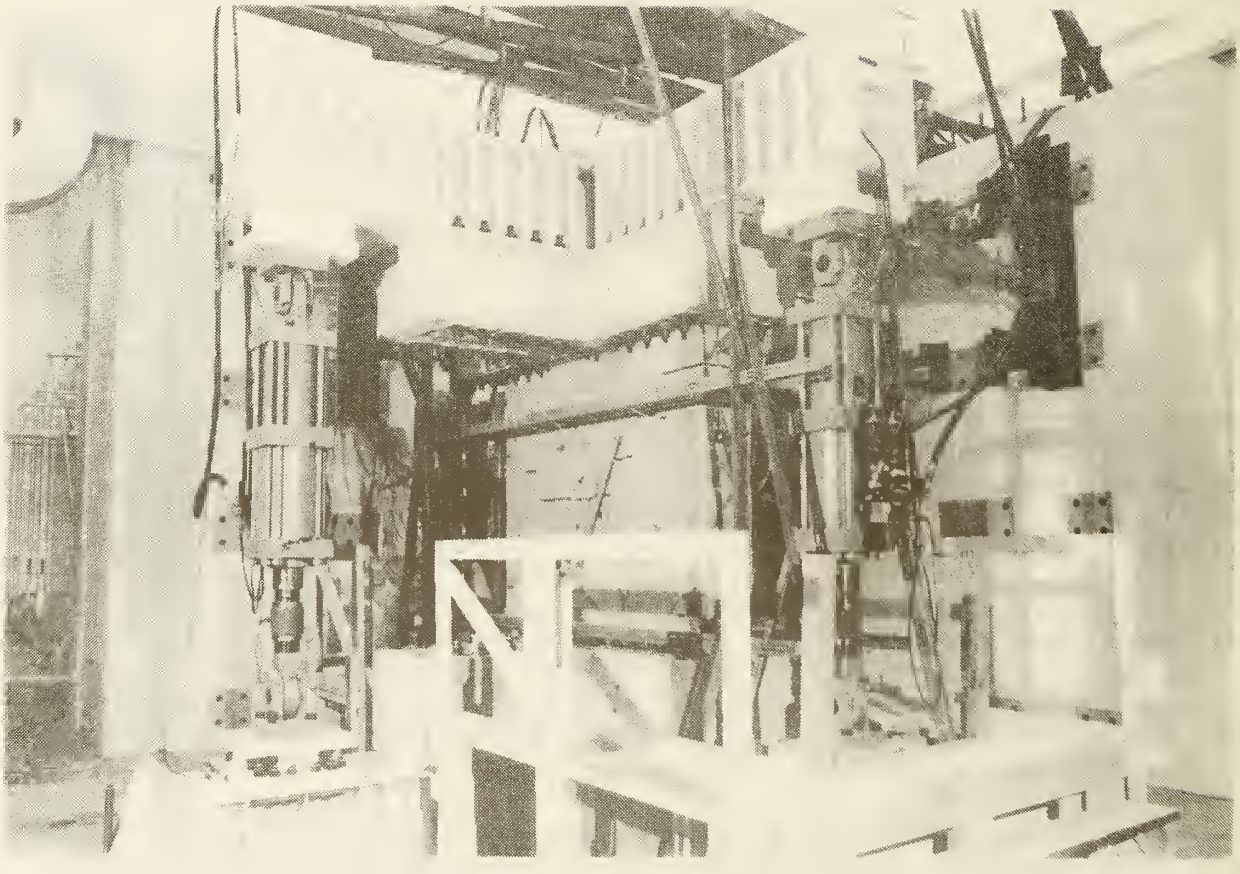

Figure 4.1 Test setup

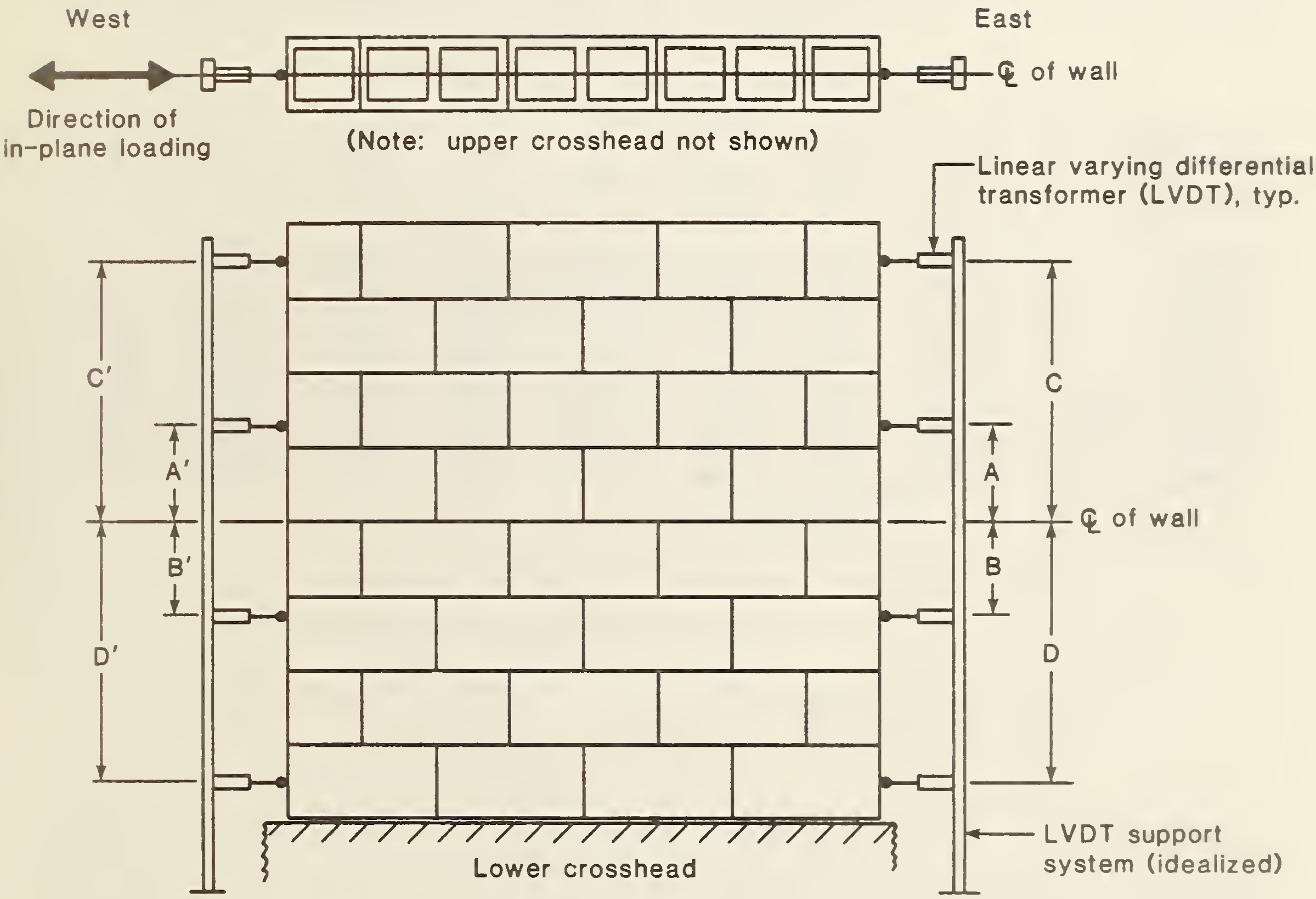

Figure 4.2 Wall panel LVDT layout

Table 4.1 Wall Panel Horizontal LVDT Location Dimensions

Refer to fig. 4.2 for identification of dimension locations A,B,C,D,A',B',C', and D'.

Wall Panel Identifier	A (in.)	B (in.)	C (in.)	D (in.)	A' (in.)	B' (in.)	C' (in.)	D' (in.)
64HH120	11.25	10.50	27.75	27.50	10.75	10.38	27.50	27.00
64HH160	9.75	10.50	28.00	27.50	10.75	10.50	27.50	27.25
64HH240	10.00	10.75	28.00	27.50	10.75	10.63	27.50	27.25
64HH300	11.00	10.50	27.75	27.25	10.88	10.25	27.38	26.75
64HH320	10.13	10.38	28.38	27.50	10.88	10.38	27.50	27.25
64HH400-3L02	10.13	10.63	28.13	27.50	11.13	10.50	27.38	27.25
64HH400-2L03	10.63	10.50	27.50	27.50	11.00	10.25	27.50	27.00
64HH500	10.75	10.50	27.63	27.50	10.88	10.38	27.50	27.00

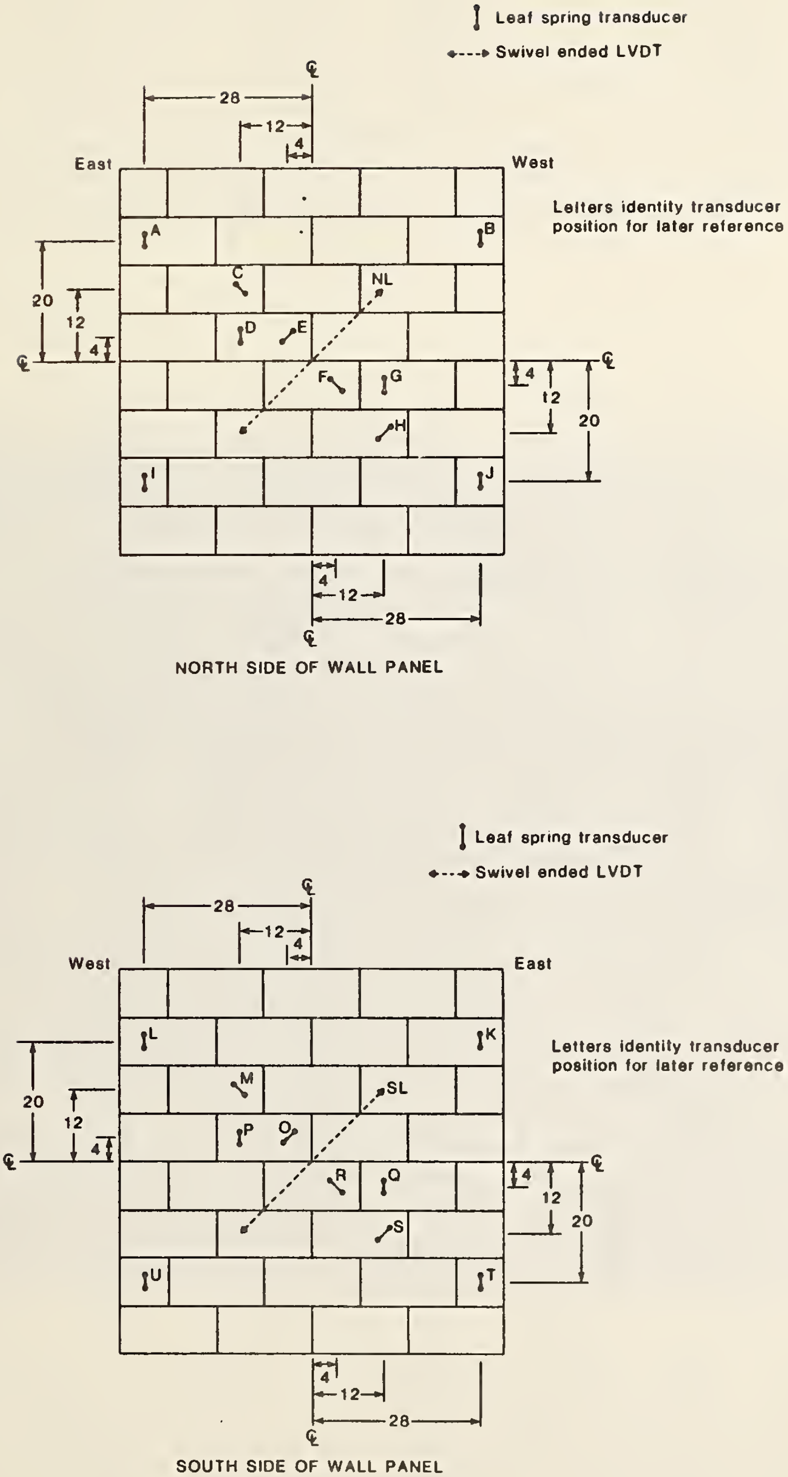

Figure 4.3 Wall panel instrumentation

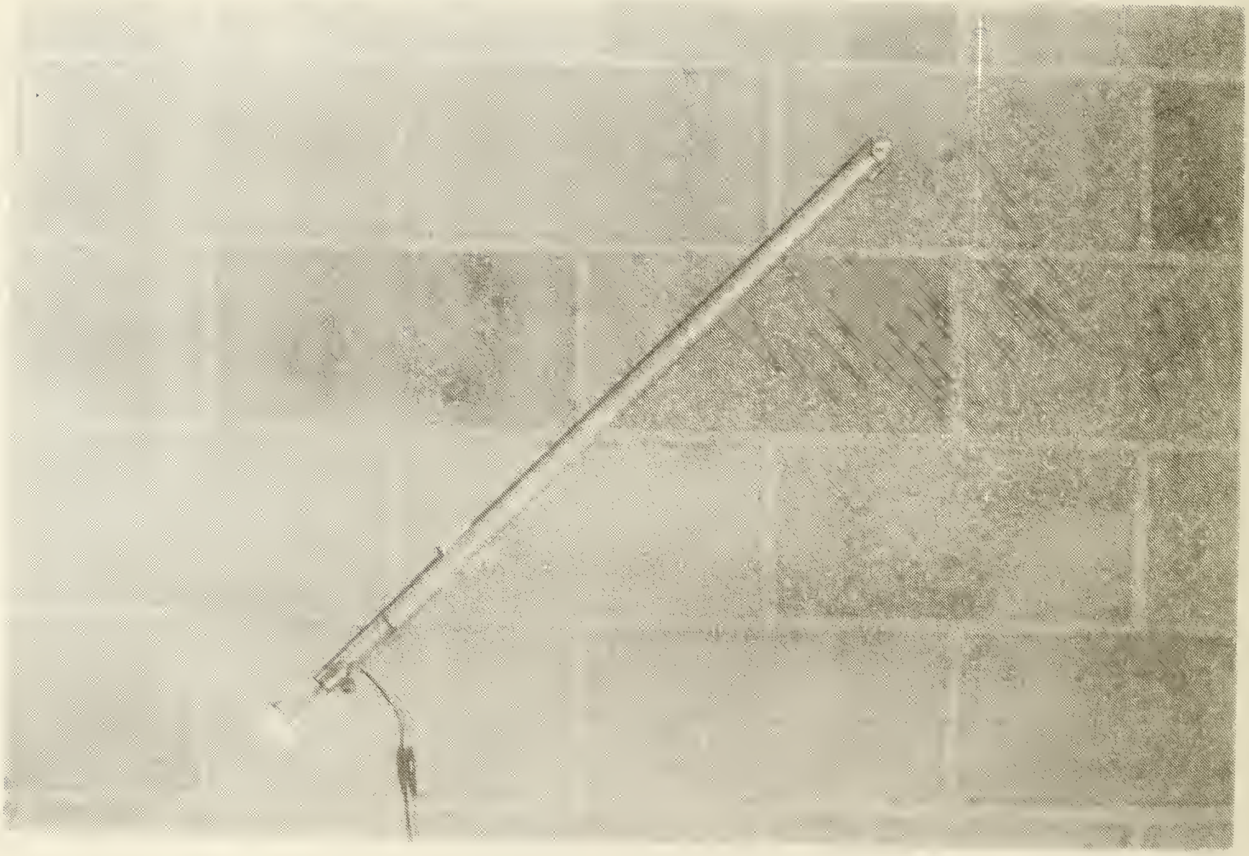

Figure 4.4 Diagonal LVDT

designed leaf spring transducers (LSTs) (fig. 4.5). The LSTs are strips of metal (titanium) with strain gages mounted on each side (temperature compensated) such that the output signal from the strain gages is proportional to the tip displacement of the leaf spring. Each LST is calibrated to determine the relationship between leaf spring tip displacement and the strain gage output. A typical estimate of standard error for the regression analysis of displacement versus LST output is 15 microinches. The LSTs are mounted on posts glued to the wall (fig. 4.5) with a gage length of 1 in. The LSTs are mounted only on block units and their gage lengths do not cross mortar joints.

4.3 TEST PROCEDURES

4.3.1 Placing a Wall Panel

Each of the wall panels was handled by attaching a carrying harness to the panel (fig. 4.6). The harness had attachment points for lifting the wall and a clamping arrangement which held the harness against the ends of the panel. An overhead crane and specially fabricated device was used to place the wall panel in the NBS/TTF. The special device was a large welded assembly in the shape of a "C" (fig. 4.7). The shape permitted the crane hook to be centered above the wall panel without interfering with the upper crosshead during placement (fig. 4.8).

The walls were set in place using mechanical stops which fixed the walls in their horizontal position. The walls were plumbed using small wedges set at four places under the face shells of the walls. The walls were fastened to the lower crosshead first, using an epoxy mortar along the bottom face shells and end cross-webs. The upper crossshead was then lowered onto the wall whose top face shell and end cross-webs were also mortared with the epoxy mortar. A small vertical compressive load (1,000 to 2,000 pounds) was applied to the wall to ensure contact between the wall and epoxy mortar. The upper crosshead was locked in position and the epoxy mortar was allowed to cure at least 16 hours before testing the wall.

4.3.2 Testing a Wall Panel

A typical test proceeded in the following steps. The data channels were checked for unusual variations in output and a measure of the ambient voltage oscillation was obtained. A first set of data was acquired to use as the "zero" condition of the test. The hydraulic actuators were pressurized and another set of data was acquired. The desired vertical compressive load was applied to the specimen with data acquired at regular intervals. After reaching the desired compressive vertical load, in-plane lateral displacements were applied to the wall panel. The lateral displacement was applied with the upper crosshead maintaining a "zero" rotation condition. The vertical displacement of the upper crosshead varied to maintain the desired axial load. The initial direction of lateral in-plane displacement was always to the west (fig. 4.9). The displacement pattern varied slightly between tests but, generally, displacement was increased in the initial direction (west) until a diagonal crack was fully formed. Afterwards, the displacement was either reversed or increased until the wall

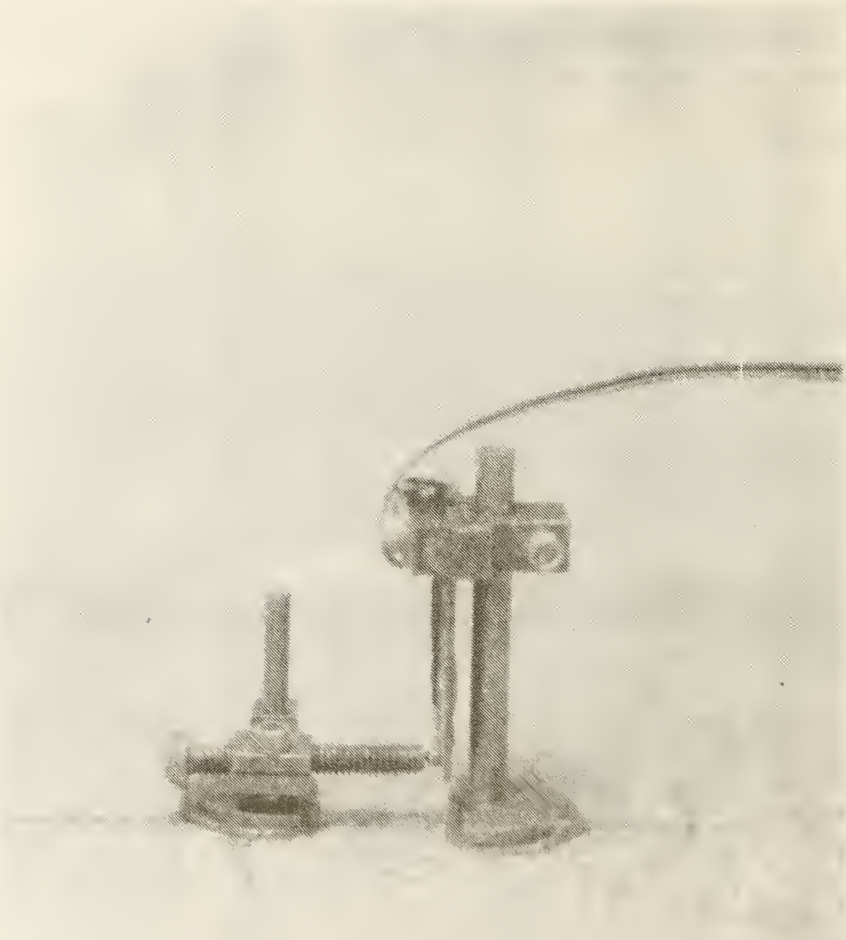

Figure 4.5 Leaf spring tranducer

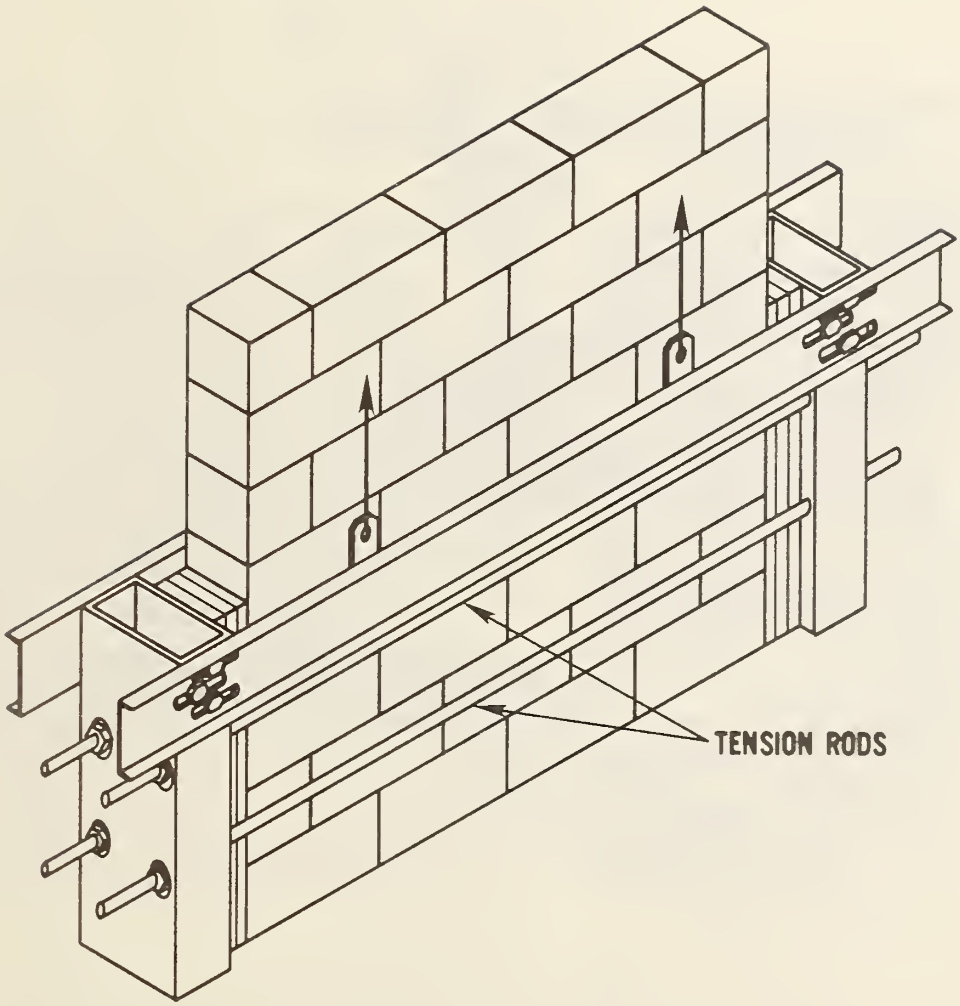

Figure 4.6 Wall panel transport harness

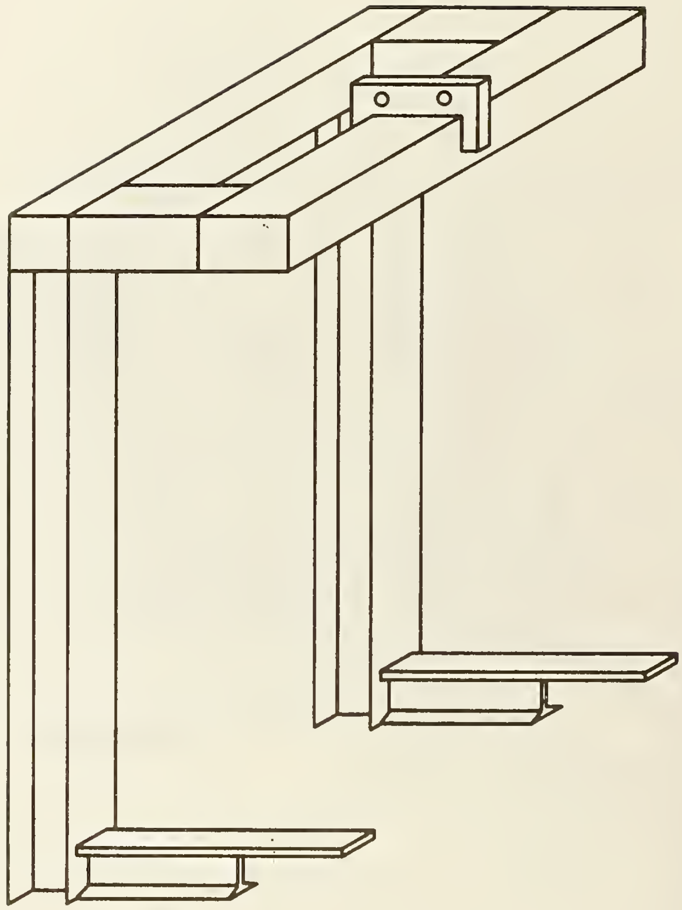

Figure 4.7 Wall panel lifting hook

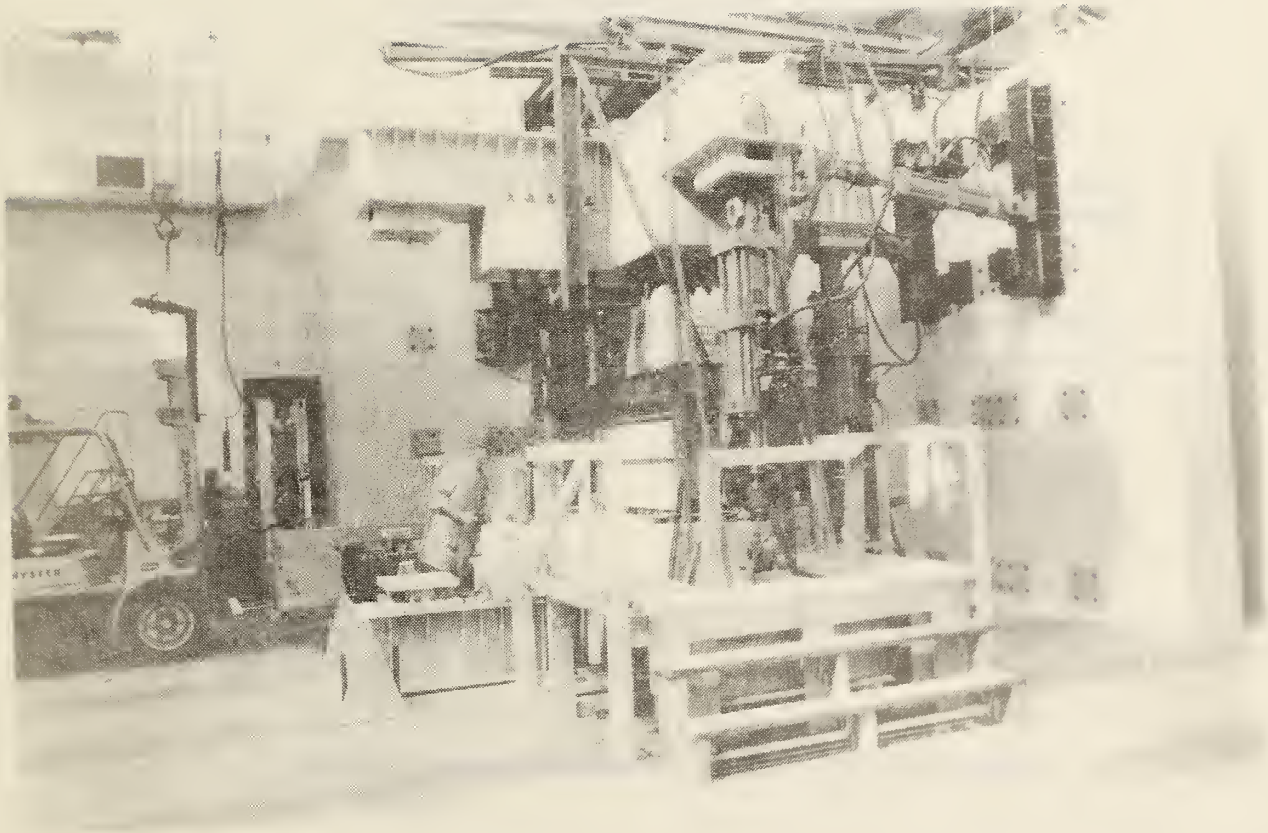

Figure 4.8 Placing a wall panel

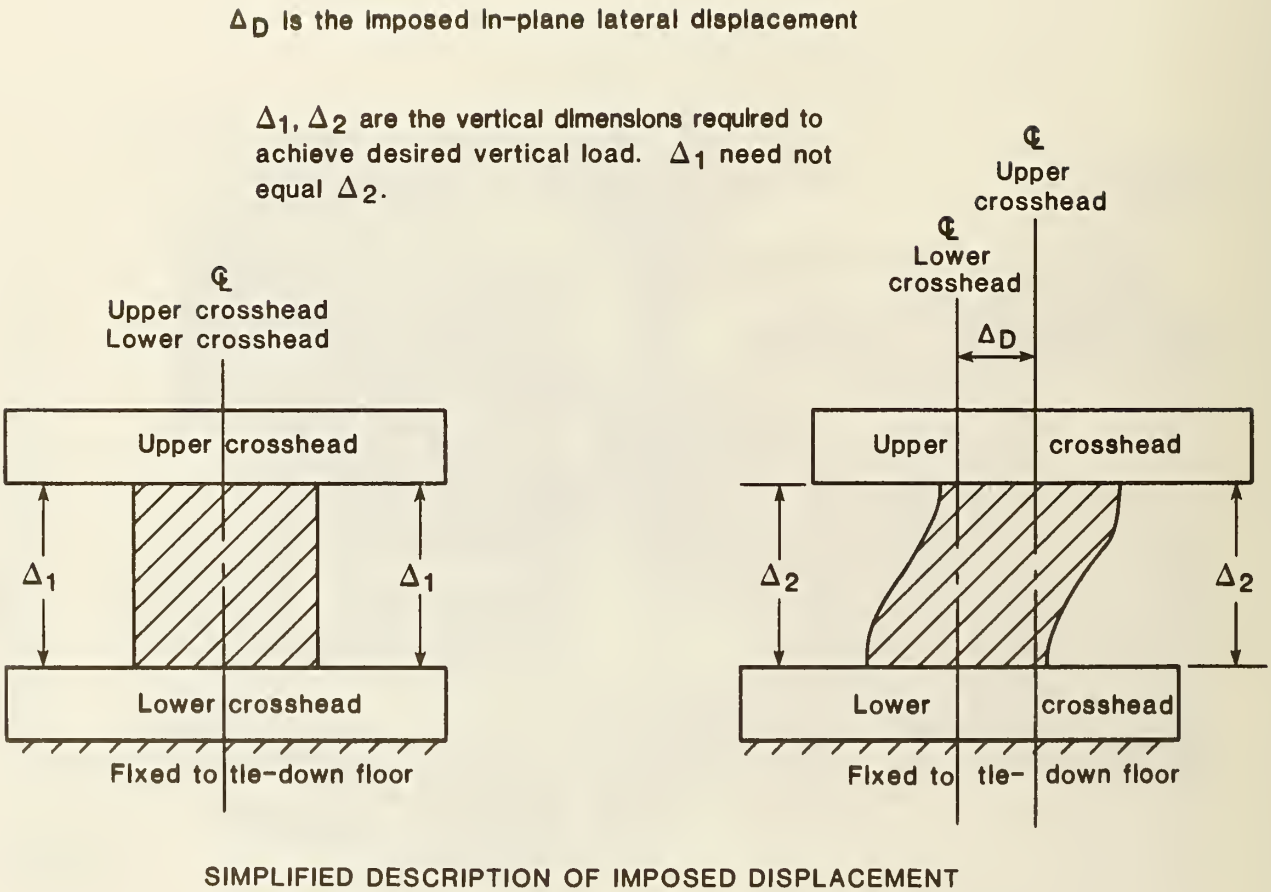

Figure 4.9 In-plane displacement method

could not support the imposed vertical load. Data were acquired at regular intervals during the test. The intervals coincided with lateral displacement increments of approximately 0.005 in.

5. WALL PANEL TEST DESCRIPTIONS

In this chapter, a brief description of each wall test and its observed behavior is presented. All of the descriptions share a common format consisting of a load-displacement curve, a crack pattern map, and a short narrative highlighting the information contained in the two figures. In general, the combination of the load-displacement curve and crack pattern map provides sufficient information to broadly describe the behavior exhibited by a test specimen. Other information such as data from the LSTs is not presented since for the purposes of this report the wall behaviors are adequately described by their load-displacement relationships.

The load-displacement curves describe the complete loading (displacement) history for each wall test and provide a primary indicator of wall behavior. The load used in developing the curves is the horizontal load acting in the plane of the wall as measured by the hydraulic actuator load transducers. The load is referred to as the global in-plane load. The displacement used in the curves is the horizontal displacement of the upper crosshead in the plane of the wall (fig. 4.9). This displacement is referred to as the global in-plane displacement to differentiate it from the in-plane wall displacement measured by the horizontal LVDTs mounted to the wall (fig. 4.2). The global in-plane displacement (GID) is determined by the displacement transducers in the hydraulic actuators. The GID and the wall displacement measured directly by LVDTs are not necessarily the same. The GID is affected by total apparatus displacement while the direct LVDT displacement more nearly measures an absolute in-plane displacement of the wall. However, the direct LVDT displacement can be strongly affected by the breakup of a wall after cracking. Spalling and splitting in the region near a wall LVDT can produce large distortions in the apparent displacement. In general, the GID is a consistent measure of displacement which is unaffected by local wall distortions and, as a result, is best for general comparisons between tests.

The crack pattern maps reflect the observations of wall cracking at selected points during the tests. The crack patterns provide useful information on the physical response of a wall to an imposed loading history. The patterns serve as a guide to identifying regions of high stress, general stress flow, and physical load resisting mechanisms. It is, of course, desirable that the points during a test at which crack patterns are recorded be identified on the load-displacement curves. This is accomplished by using an identifying symbol which marks a location on the load-displacement curve corresponding to an associated crack pattern on the map. The symbols are capital letters starting with the letter A. Thus, in the narrative of each description reference will be made to specific points on the load-displacement curve identified by the symbols and by implication the crack pattern also associated with that symbol. The symbols are unique within a description, but not between tests. Symbol A does not refer to the same point on every load-displacement curve.

5.1 64HH120 - 2L04

The loading history of this wall included displacements with several different compressive axial stress levels (fig. 5.1). The purpose of this test was to find the minimum level of compressive axial stress required to produce a "shear" failure (diagonal cracking) rather than a "flexure" failure (horizontal cracking). The initial axial stress of 60 psi did not suppress the flexural mode as shown by the crack pattern (fig. 5.2) which exhibited a pronounced horizontal flexure crack at the first course mortar joint. The lateral displacement was returned to zero and the axial stress was increased to 120 psi. The lateral displacement was then increased until the bottom west block developed a diagonal crack extending from the previous flexure crack in the mortar joint to the bottom corner of the block. The displacement was denoted by the symbol A in fig. 5.1. The decrease in lateral load resistance coinciding with the formation of the block diagonal crack was very pronounced. At this point in the test it was decided to increase the axial stress and repeat the lateral movement until a complete wall diagonal crack could be formed. The lateral displacement was decreased until the lateral force was approximately zero. The axial stress was increased to 200 psi and the lateral displacement was then increased. It appeared that the cracking was confined to the existing cracks so the axial stress was increased again. The previous steps were repeated except that the axial stress was increased to 300 psi. A complete wall diagonal crack occurred at this axial stress level with an accompanying reduction in lateral load resistance. The wall diagonal crack passed mostly through the blocks rather than the mortar joints. The displacement was denoted by symbol B in fig. 5.1. The displacement was then reversed until a diagonal crack formed along the other diagonal. The displacement at which this occured was denoted by symbol C in fig. 5.1.

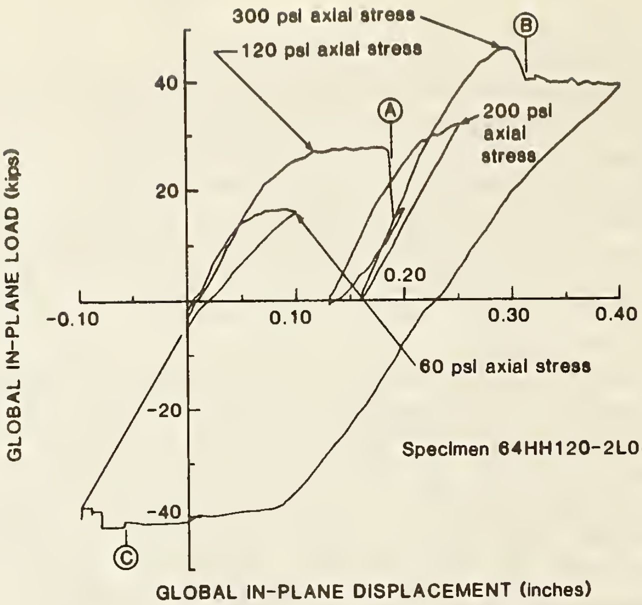

Figure 5.1 Specimen 64HH120-2L04 load-displacement curve

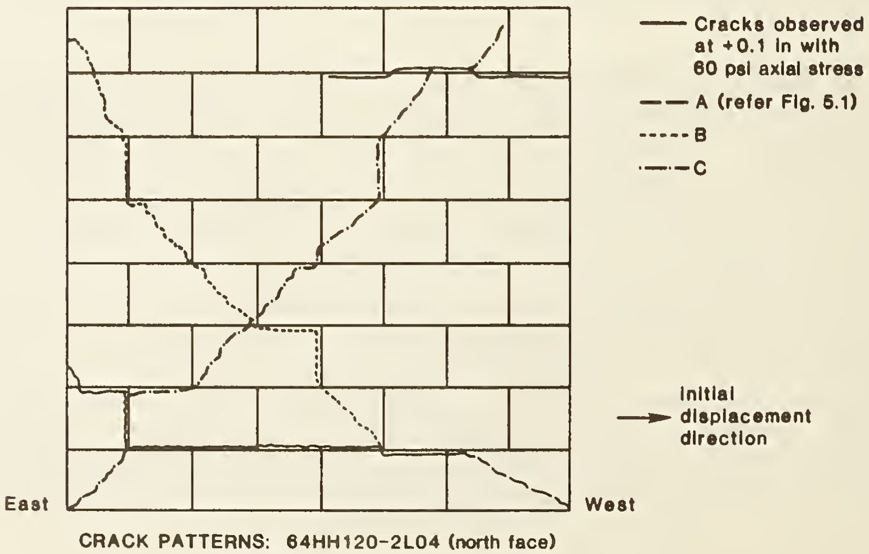

Figure 5.2 Specimen 64HH120-2L04 crack pattern

5.2 64HH160 - 3L01

The compressive axial stress was initially set at 160 psi. The lateral displacement was eventually increased to a displacement equal to about three times that at which diagonal cracking actually occurred in the wall (fig. 5.3). The diagonal cracking which occurred with the 160 psi axial stress was primarily confined to the mortar joints in a stair-step fashion (fig. 5.4). The axial stress was subsequently increased to observe the effect, if any, of increasing the axial stress after cracking had occurred. As seen in the load-displacement curve the maximum lateral load resistance for higher axial stress was significantly greater than the resistance observed at the same displacement for lower axial stresses, but the overall maximums were not significantly different. For example, the maximum lateral load resistance observed for the case of 240 psi axial stress was approximately 30 kips and occurred at a displacement of about 0.31 in. The resistance observed at about the same displacement for the case of 160 psi axial stress was 21 kips. However, the maximum lateral load resistance for the case of 160 psi axial stress was 30 kips. Thus, the maximum load resistance was about the same regardless of axial stress when the wall was already cracked before increasing axial stress. The limiting resistance seemed to involve that associated with slippage along the crack since relatively few new cracks formed, but crushing zones were observed along certain portions of the primary diagonal crack. Lateral load resistance was stable within the lateral displacement imposed on the wall at the higher compressive axial stress.

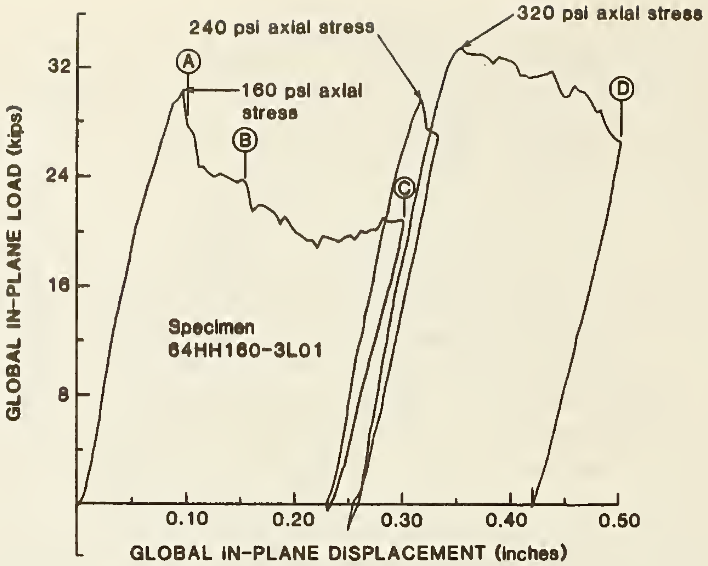

Figure 5.3 Specimen 64HH160-3L01 load-displacement curve

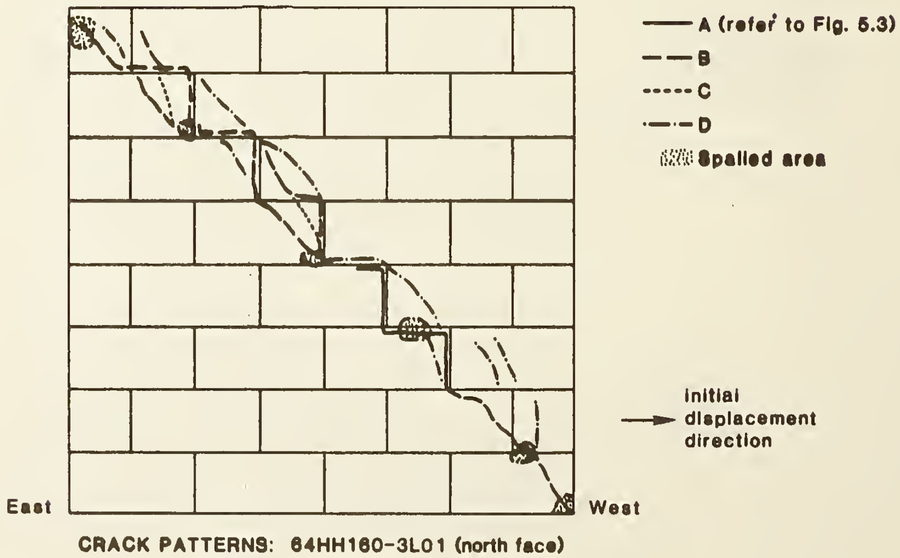

Figure 5.4 Specimen 64HH160-3L01 crack pattern

5.3 64HH240 - 3L04

The compressive axial stress was maintained at 240 psi throughout the test. The diagonal cracking began in the center portion of the wall diagonal and extended to each corner. A sharp drop in lateral load resistance (fig. 5.5) occurred as the diagonal crack reached its full length. The diagonal crack passed through both block units and mortar joints (fig. 5.6). As the displacement was increased past the displacement at which diagonal cracking first occurred the lateral resistance decreased slowly. However, a point in the loading history was reached (denoted by the symbol B in fig. 5.5) where two loud bangs were heard from the wall and the resistance decreased at a higher rate. No new cracks were observed to form with the noises and based on later examination of the wall it was suggested that the noises were caused by the splitting of cross-webs in the compression zones of the wall. The displacement was cycled several times with a small displacement amplitude near the extreme displacement limit to observe the rate of degradation of resistance with non-reversed cycling. The displacement was then fully reversed until a diagonal crack formed along the other diagonal of the wall panel. The final failure was primarily crushing along the diagonal crack in the lower west corner compression zone.

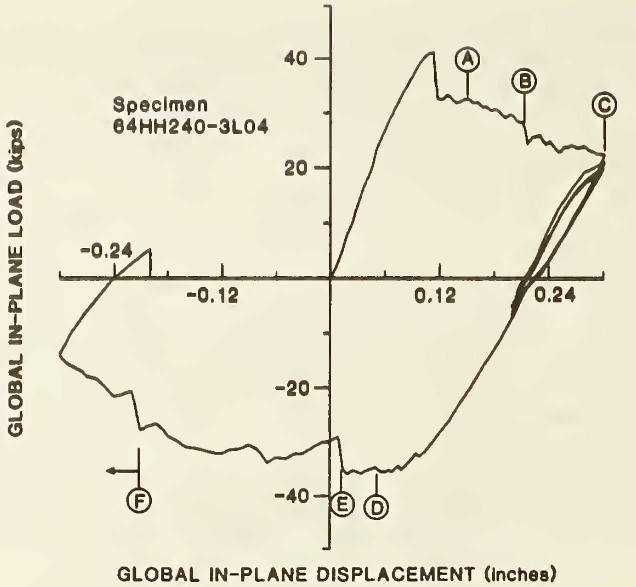

Figure 5.5 Specimen 64HH240-3L04 load-displacement curve

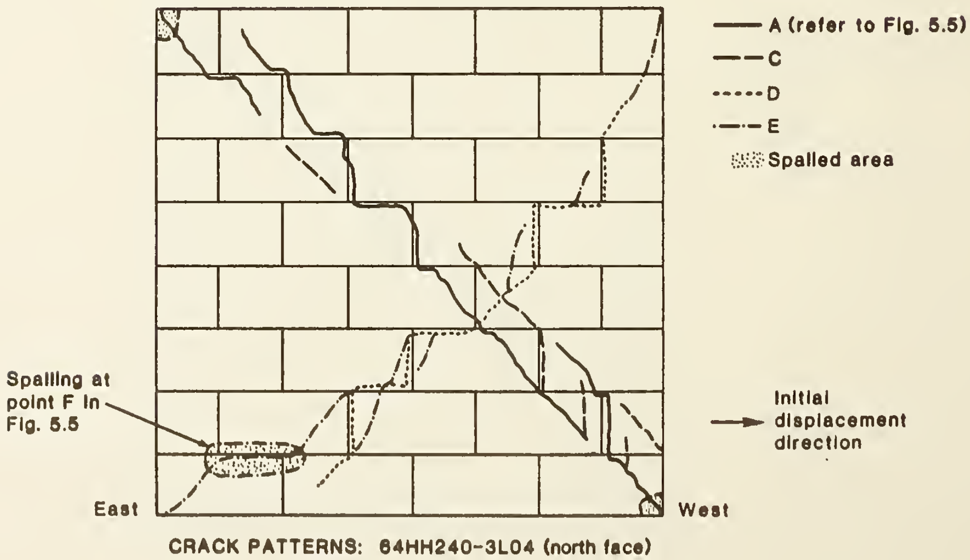

Figure 5.6 Specimen 64HH240-3L04 crack pattern

5.4 64HH300 - 2L05

The compressive axial stress was maintained at 300 psi throughout the test. The loading history was basically one cycle with fully reversed displacements (fig. 5.7). The diagonal crack which formed during the initial lateral displacement of the wall panel initiated in the central region of the wall and extended along the diagonal to the opposite corners (fig. 5.8). The cracking occurred primarily through the block units and not the mortar joints. The small flexure crack which was observed in the lower east bed joint occurred at a displacement very near the occurrence of diagonal cracking. As the displacement was increased past the displacement at which the first full diagonal crack formed the lateral resistance was initially stable (remained constant) but, with increasing displacement, a parallel diagonal crack formed and the resistance decreased. The displacement was increased until noticeable crushing occurred in the lower west corner of the wall. As the crushing occurred the lateral resistance decreased sharply. The displacement was then fully reversed. Diagonal cracks formed along the other diagonal, but this time the diagonal cracking passed through both block units and mortar joints. The angle of the crack was also different.

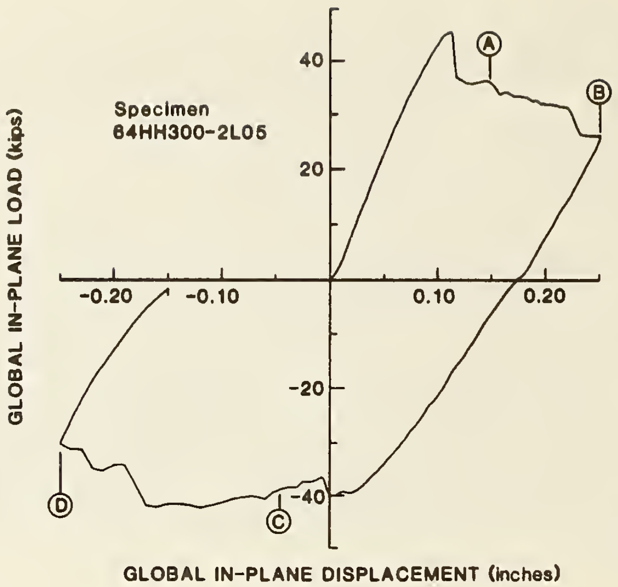

Figure 5.7 Specimen 64HH300-2L05 load-displacement curve

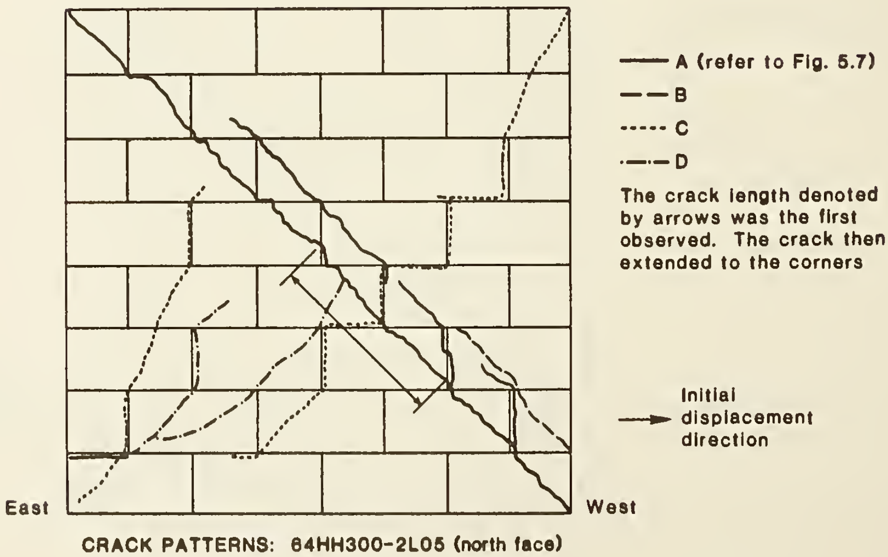

Figure 5.8 Specimen 64HH300-2L05 crack pattern

5.5 64HH320 - 3L03

A compressive axial stress of 320 psi was maintained on the wall panel throughout the lateral displacement. The wall was first displaced 0.2 in. to the west (fig. 5.9). The diagonal crack which formed (fig. 5.10) during this displacement was accompanied by a loud bang and a sudden drop in lateral load resistance. Displacement of the wall between the displacement at first cracking and 0.2 in. produced several more loud bangs, but no new visible cracking. The load resistance decreased with increasing displacements. The displacement was fully reversed and then cycled three times between the two reversed displacement limits. The first reversal to 0.2 in. (denoted by symbol C in fig. 5.9) in the east direction caused two diagonal cracks to form, one after the other. The first crack did not extend to the two corners (lower east and upper west), but the second crack included extensions to the corners. The second cycle to 0.2 in. to the west (denoted by symbol D in fig. 5.9) produced a vertical crack along the face shells of the west end with an accompanying large reduction in lateral load resistance. The second cycle to 0.2 in. to the east (denoted by symbol E in fig. 5.9) produced vertical cracking on the east side, but no drop in resistance was observed. The third cycle produced some reduction in resistance and stiffness, but no new cracks were visible. The attempt at a fourth cycle was terminated when the wall panel could not sustain the 320 psi axial stress and crushing occurred.

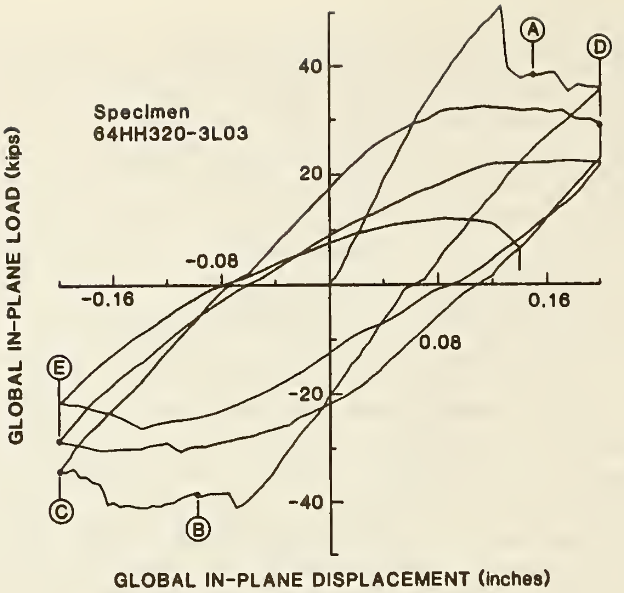

Figure 5.9 Specimen 64HH320-3L03 load-displacement curve

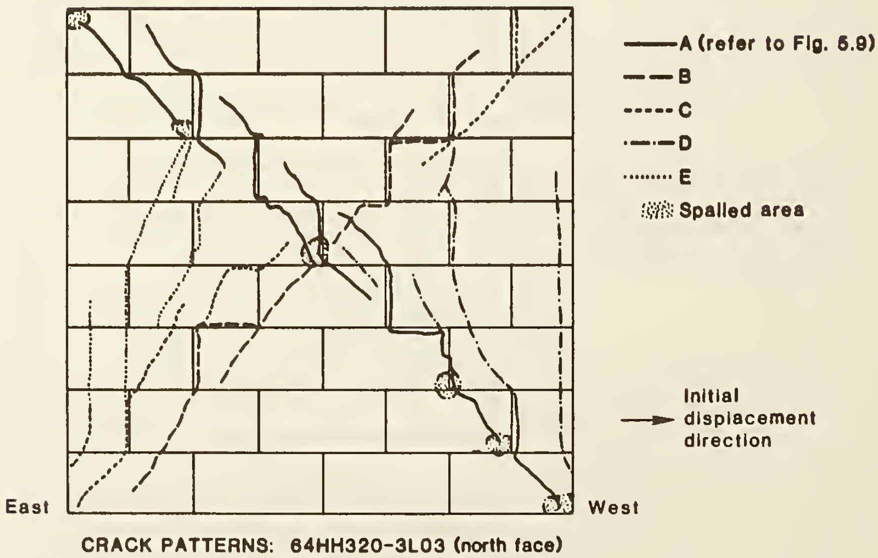

Figure 5.10 Specimen 64HH320-3L03 crack pattern

5.6 64HH400 - 3L02

The compressive axial stress was maintained at 400 psi. The loading history was basically one cycle between reversed displacement limits (fig. 5.11), but the displacement limits were unequal. The first diagonal crack due to displacement in the west direction occurred at 0.125 in (fig. 5.12). The cracking initiated in the central region of the wall and then propogated along a line below the true wall diagonal. A second diagonal crack formed at 0.175 in. which propagated along and above the true wall diagonal. The lateral load resistance was essentially constant during the formations of both cracks. The lateral load resistance remained constant until a displacement of about 0.3 in. was reached at which point a loud bang was heard followed by extensions of the previous cracks and a sudden decrease in lateral load resistance. During the reversal of displacement diagonal cracks formed along the other diagonal, but the test was terminated when the wall could no longer sustain the 400 kip axial stress.

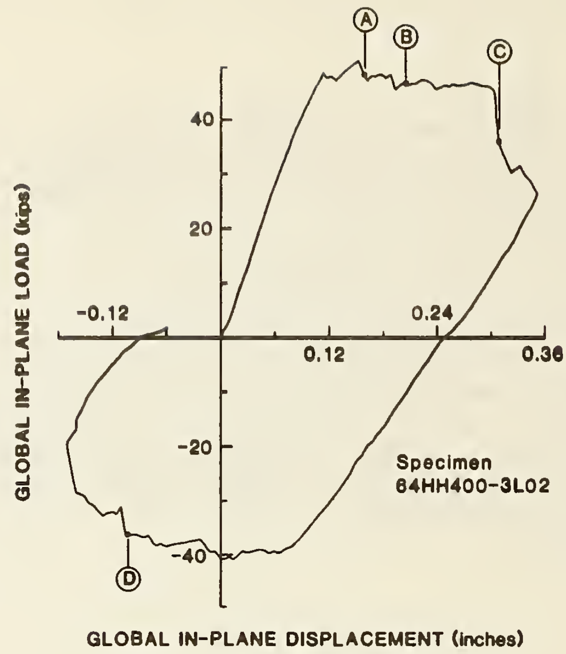

Figure 5.11 Specimen 64HH400-3L02 load-displacement curve

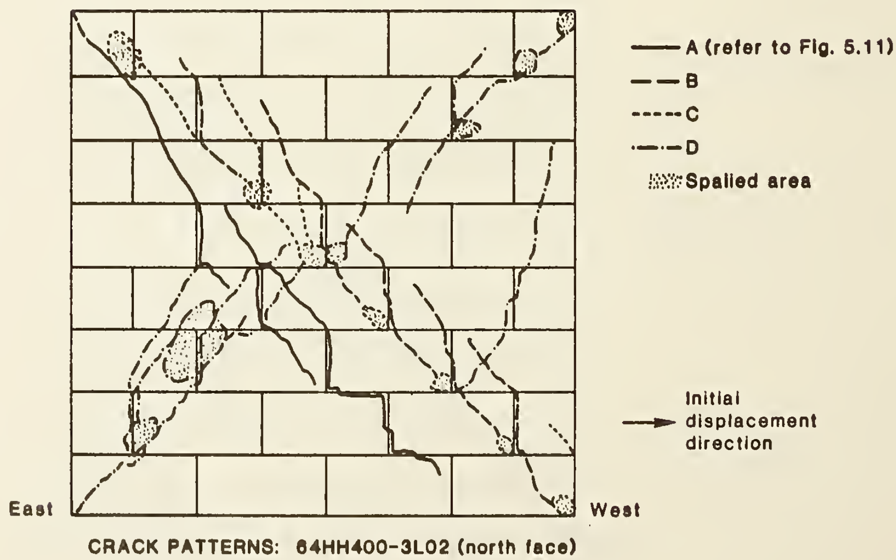

Figure 5.12 Specimen 64HH400-3L02 crack pattern

5.7 64HH400 - 2L03

A compressive axial stress of 400 psi was maintained on the wall. This test was a monotonic test to failure (fig. 5.13). The final failure was an inability to sustain the 400 psi axial stress. The first diagonal crack to form included the upper east corner, but did not extend to the lower west corner (fig. 5.14). The crack was visible only after the lateral load resistance dropped. Just prior to the drop in resistance a loud bang was heard. The second, roughly parallel, crack included the lower west corner. The second crack occurred only after a significant increase in displacement past the displacement coinciding with the formation of the first crack. The lateral load resistance increased as the displacement was increased until a displacement was reached where a complete diagonal crack formed. Increasing the displacement further led to a zone of crushing in the lower west corner followed by a sudden loss in vertical load (stress) resistance.

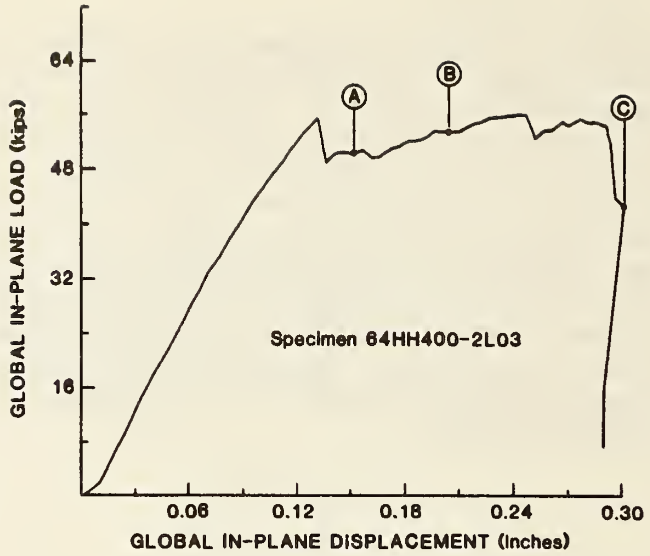

Figure 5.13 Specimen 64HH400-2L03 load-displacement curve

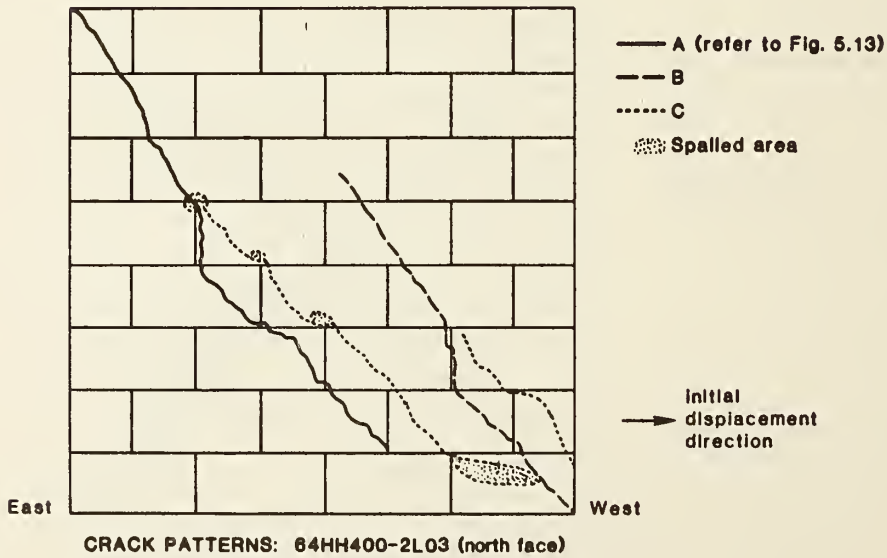

Figure 5.14 Specimen 64HH400-2L03 crack pattern

5.8 64HH500 - 2L06

The axial stress was maintained at 500 psi during most of the test. The loading history was basically a single cycle, reversed displacement pattern (fig. 5.15), but an unexpected problem modified the loading history. The first attempt at applying a lateral displacement was terminated by the control software because preset load limits were exceeded. Preset load limits restrict the range of loads which can be applied to the specimen as a precaution against unintentional overloading of the specimen. The axial stress was reduced and the lateral displacement returned to zero. The preset load limits were increased so that the error indication would not occur and the axial stress was again increased to 500 psi. The lateral displacement was then applied with no further difficulties. The initial diagonal cracking occurred in the center region of the wall (fig. 5.16) accompanied by a reduction in lateral load resistance. With increasing displacement, the crack propagated to the corners and a second crack formed roughly parallel to the first crack. A displacement level was reached, however, which resulted in crushing of certain regions of the wall followed by a sharp drop in lateral load resistance and the inability of the wall to sustain the 500 psi axial stress (denoted by symbol C in fig. 5.15). The axial stress was reduced and the displacement reversed until zero lateral load was achieved. The axial stress was then increased to 500 psi and the displacement increased in the opposite or east direction. Diagonal cracking occurred in the opposite direction, but the cracks were not continuous. The test was terminated because the vertical displacement required to maintain the vertical 500 psi stress was considered excessive.

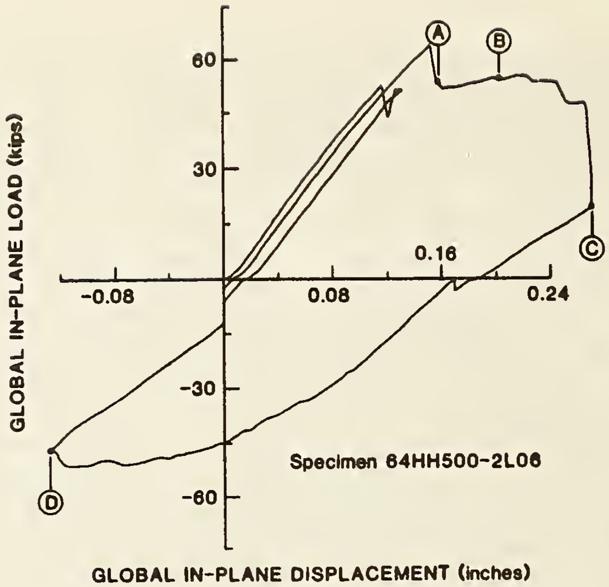

Figure 5.15 Specimen 64HH500-2L06 load-displacement curve

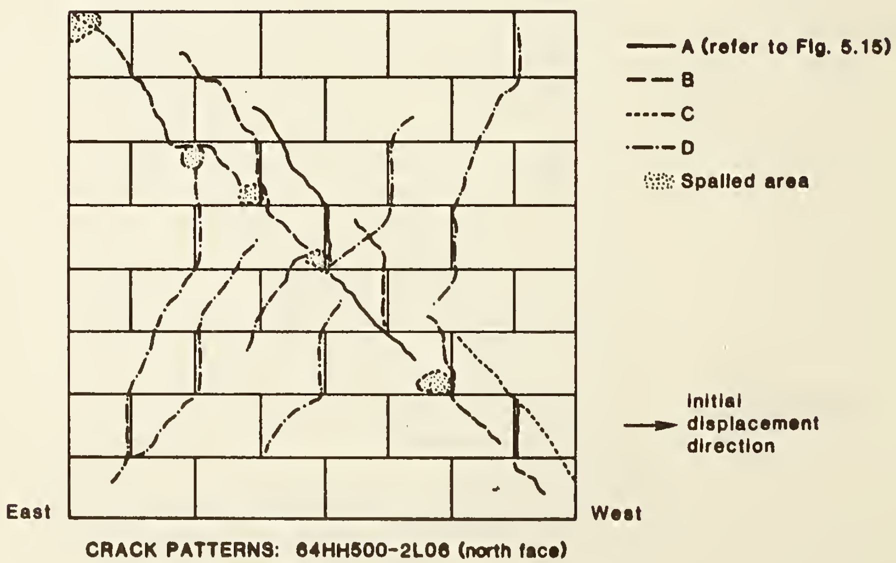

Figure 5.16 Specimen 64HH500-2L06 crack pattern

6. DISCUSSION OF RESULTS

The discussion which follows is limited to the wall panels described in this report. Detailed data analysis is not presented, but it is left to a future report when other test results will be available. A general overview of the apparent behavior is presented with trends noted where evident.

6.1 GENERAL BEHAVIOR

The crack patterns exhibited by each wall panel indicate a diagonal-tension type of failure when a flexural mode of failure is suppressed by the presence of sufficient axial (vertical) compression. There are variations in the crack patterns which point to a change in the actual critical region with increasing axial compression. The wall panel with 120 psi axial compression (64HH120) fails with a localized diagonal-tension failure in the compression zone of the panel (fig. 5.2). However, the same panel fails with a complete diagonal crack when the axial compression is increased to 300 psi with a resulting higher lateral load resistance. Such behavior indicates that the failure criterion is dependent on the complete state of stress and not just one particular component of the stress. Just as importantly, however, is the recognition that state of stress computations can not be based solely on the overall panel average stresses, but must also include the local conditions which may be radically different. An example is the compression zone of a cracked section. The compression stresses and shear stresses may be quite high in this zone with the result being a local diagonal-tension failure.

At the other end of the spectrum, relatively high axial compression alters the diagonal crack pattern such that instead of a single complete diagonal crack which splits the panel, multiple diagonal cracks form. The cracks radiate from opposite corners at angles which cause the cracks to bound a region of the wall along the true diagonal. An example is the crack pattern of 64HH400 shown in fig. 5.14. The final failure seems to be a crushing type of failure in the bounded region. It should be pointed out that the axial stress level required to produce banded diagonal cracking is high in comparison to typical design levels of axial stress.

Between the two extreme behaviors the crack pattern is a relatively straightforward diagonal crack which generally follows the wall panel diagonal. In the wall panels tested, the cracks generally passed through the block units or at least included portions which passed through block units. It would appear reasonable that since the cracking passed primarily through the block units the general susceptibility to cracking for these wall panels is determined by material properties of the block units and not the mortar.

6.2 LOAD-DISPLACEMENT RELATIONSHIPS

The load-displacement curves for all eight wall panels are shown in fig. 6.1. The load is the in-plane global lateral load and the displacement is the movement of the wall at the top course of the wall as measured by the top west LVDT (fig. 4.2). The curves are only shown to a point slightly beyond achieving the peak lateral load resistance. The curves for 64HH120 and 64HH500 have been adjusted to eliminate their initial loading cycles. It is clear that

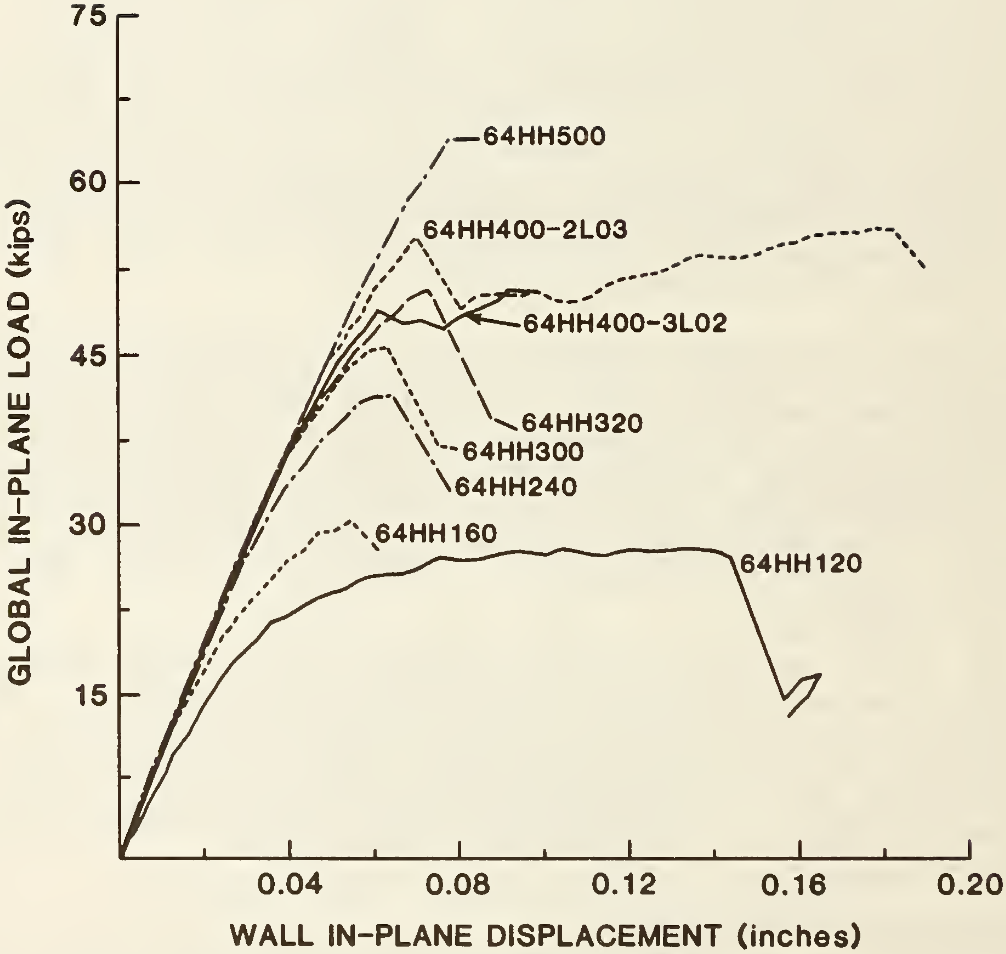

Figure 6.1. Combined load-displacement curves

wall 64HH120 is heavily influenced by flexure which accounts for the general rounding of the curve and the large displacement required to achieve diagonal-tension failure. The wall 64HH160 demonstrates much less influence of flexure, with only a small increase in maximum lateral load resistance. The remaining walls show the same general trend, less rounding of the curve and a sudden drop in lateral load resistance. The wall 64HH400 - 3L02 is an interesting anomaly in that its behavior is not like its companion 64HH400 - 2L03. At present, it is not known why 64HH400 - 3L02 exhibited such behavior.

The walls 64HH400 - 3L02, 64HH400 - 2L03, and 64HH500 exhibited a markedly more stable maximum lateral load resistance as compared to the other five walls. The complete load-displacement curves are not shown in fig. 6.1, but the curve for wall 64HH400 - 2L03 in fig. 6.1 can be used as a guide to their general behavior. The lateral load resistance is maintained at a high level for a much larger displacement than was possible for the other five walls. The relationship for 64HH400 - 2L03 suggests that the load resisting mechanism changed during the test. The first peak resistance coincides with the formation of the initial diagonal cracks while the gradual increase in resistance after that is associated with the development of the banded crack pattern. Final failure results from the crushing of the material within the banded region.

The effect of displacement reversals is observed in walls 64HH240 and 64HH320. The load-displacement curve for wall 64HH240 shown in fig. 5.5 suggests that for partial reversals of displacements such as a half-cycle pattern the effect of reversals on stiffness and lateral load resistance is small. However, the load-deflection curve for wall 64HH320 (fig. 5.9) indicates that the effect of complete displacement reversals is significant with large reductions in stiffness and lateral load resistance.

6.3 DIAGONAL TENSION STRAIN

Numerous strain measurements are available for each wall panel test. The strain measurements are from a variety of locations on each wall panel (fig. 4.3). For this report the data from only one of the locations is presented. The strain measured by an LVDT mounted parallel to the diagonals provide an excellent general indication of overall wall diagonal strain. Since the cracking appears to form as a result of diagonal tension the displacement measured by the north side LVDT (denoted as NL in fig. 4.3) provides a measure of the diagonal tensile strain in the wall. The local displacements measured by the diagonally mounted wall LVDTs are divided by the initial gage length to arrive at a strain value. The gage length used in all computations for the diagonally mounted LVDT strain is 33.941 in. which is the diagonal length of a 24 in. square (fig. 4.3).

In fig. 6.2, the computed strains from the north LVDT measurements are plotted versus the same wall displacement used in fig. 6.1. The strain-displacement curves exhibit a definite breakpoint at which the curves become asymptotic with large increases in strain for small increases in displacement. This is, of course, indicative of the formation of physical cracks in the wall. The curve for 64HH120 is an exception to the general trend, but it may be discounted since the wall is strongly influenced by flexure rather than by shear as is the case

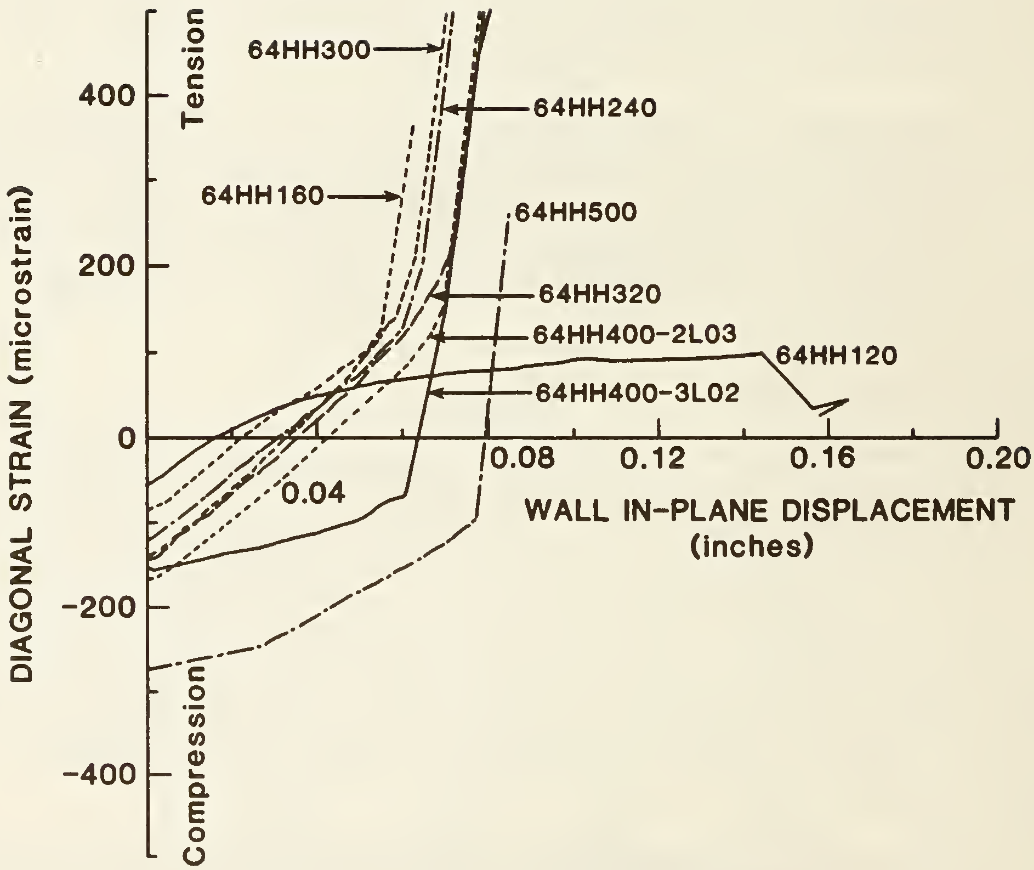

Figure 6.2. Combined diagonal strain-displacement curves

in the other walls. More interestingly, however, is that for the walls having moderate amounts of axial stress (160-320 psi) the strain at which the strain-displacement relation changes is approximately the same for all the walls and has a value of about 150 microstrain. This common value certainly suggests that the limiting criterion for the formation of diagonal cracking and consequently maximum lateral load resistance is determined by a critical tensile strain. The critical or threshold value of strain is more clearly shown in fig. 6.3 which is a plot of global in-plane load versus the same strain as in fig. 6.2. The wall panels 64HH400-3L02 and 64HH500 are exceptions to the common threshold tensile strain of 150 microstrain, but the two walls do exhibit a similar strain threshold. That fact may eventually provide a clue as to why they behave differently, but the information currently available does not provide an answer.

6.4 MAXIMUM LATERAL LOAD RESISTANCE

Typically, the single most important aspect of behavior to a designer is the maximum resistance. As a result, the factors which influence the maximum resistance need to be accounted for in design. Based on eight wall panel tests the influence of axial compressive stress on the diagonal-tension capacity of a wall follows a definite trend. A list of the maximum lateral load resistances for the wall panels is presented in table 6.1. Clearly, for changes in axial compressive stress between 120 and 500 psi, increasing the axial stress increases the maximum lateral load resistance. The trend of the relationship is shown in fig. 6.4 which is a plot of maximum shear stress versus the axial compressive stress. The maximum shear stress is computed by dividing the maximum lateral load resistance by the net cross sectional area of the wall (246 sq. in.). The axial stress is the total axial load divided the same net cross sectional area of the wall. The relationship is strongly linear as evidenced by the "best-fit" line developed by a linear regression analysis and its resulting correlation coefficient (0.98).

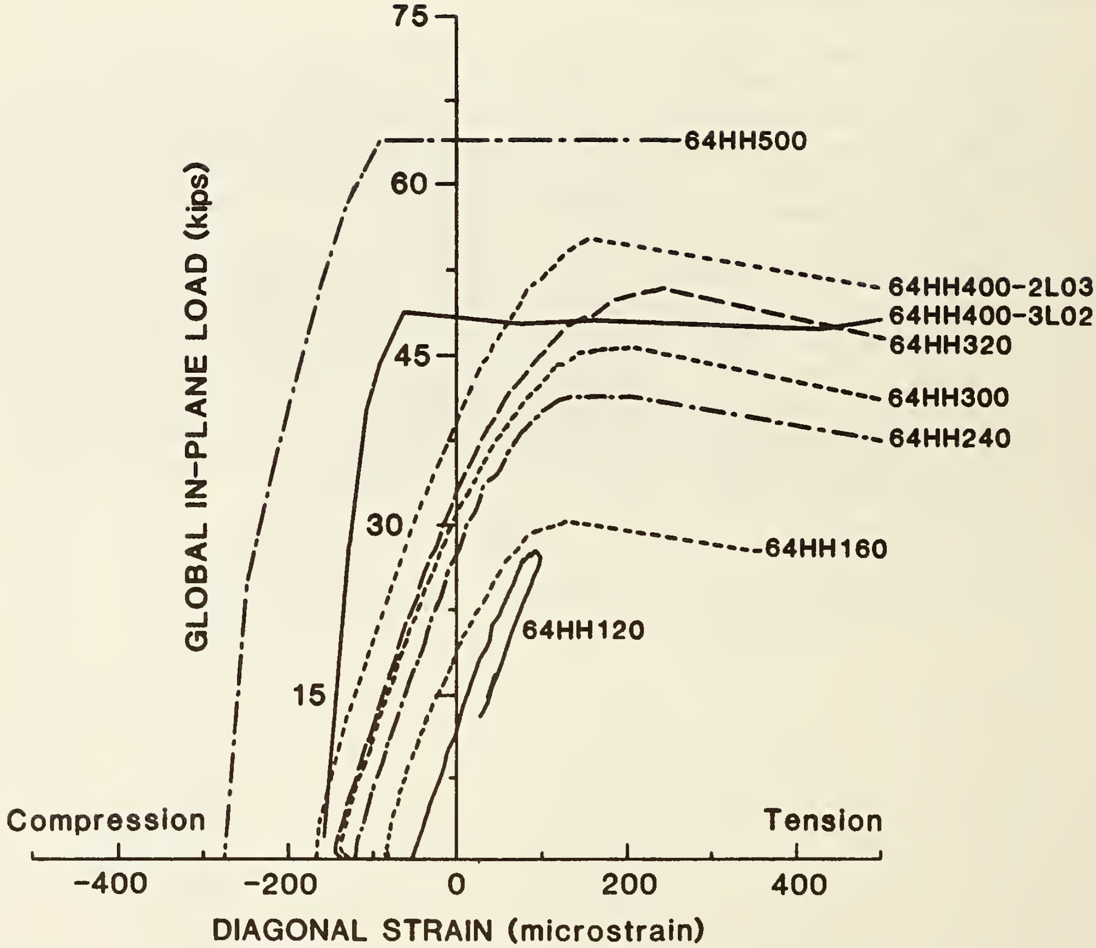

Figure 6.3. Combined load-diagonal strain curves

Table 6.1 Wall Panel Maximum Lateral Loads

Wall Panel Identifier	Axial Compressive Stress (psi)*	Maximum Lateral Load Resistance (kips-psi)*	Wall Displacement at Maximum Resistance (in.)**
64HH120-2L04	122	27.8 - 113	0.171
64HH160-3L01	162	30.3 - 123	0.054
64HH240-3L04	243	41.2 - 167	0.057
64HH300-2L05	305	45.7 - 186	0.061
64HH320-3L03	325	50.7 - 206	0.069
64HH400-3L02	406	50.5 - 205	0.089
64HH400-2L03	406	55.9 - 227	0.174/0.067***
64HH500-2L06	507	63.9 - 260	0.0081

* Stress is based on a net cross sectional area of 4×61.5 in.^{2} = 246 in.^{2}.

** Displacement of the top west horizontal LVDT.

*** A first peak lateral load resistance occurred at 0.067 in. with a value of 55.2 kips.

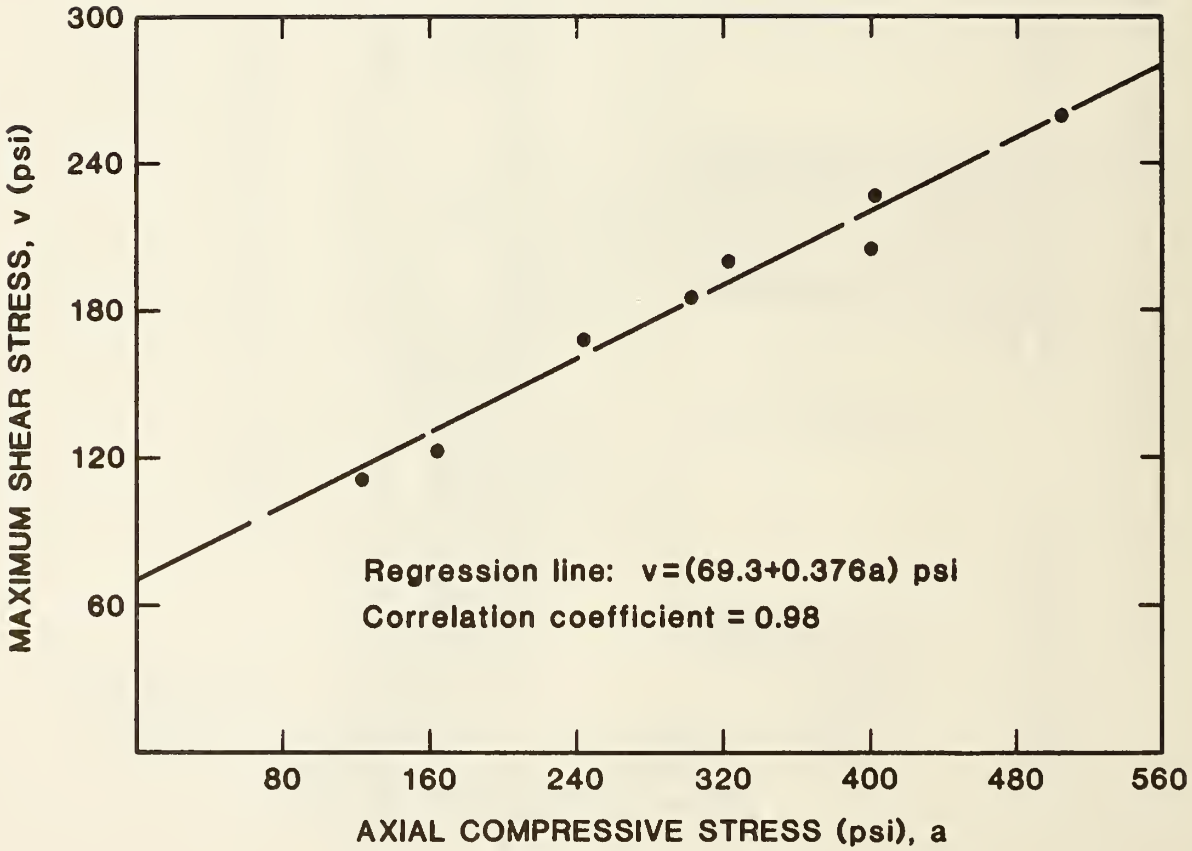

Figure 6.4. Maximum shear resistance versus axial stress

7. SUMMARY AND CONCLUSIONS

7.1 SUMMARY

Experimental data were presented from eight wall panel tests. Each wall panel was approximately 64 in. long by 64 in. high by 8 in. thick and constructed of similar hollow concrete block masonry units. Each wall panel was subjected to a lateral in-plane displacement at its top surface while the bottom surface was fixed. The primary variable was the magnitude of vertical in-plane compressive stress maintained on the wall during the lateral displacement. The acquired data included forces and displacements imposed on the wall panels and local strain measurements on each of the two face shell surfaces of the walls. A limited presentation of data interpretation demonstrated the strong, nearly linear relationship between increasing levels of vertical compression and the maximum lateral force resistance achieved by the wall panels.

7.2 CONCLUSIONS

The conclusions presented below are based on eight wall panel tests. The wall panels have similar geometry and are built using similar materials by the same mason.

- A nearly linear relationship existed between increased amounts of axial compression and the resulting maximum lateral (in-plane) load resistance of the wall panel.

- The lateral displacement at which maximum lateral load resistance was achieved was not significantly influenced by the amount of vertical compression for failures of a similar type.

- Tensile strain across the wall panel diagonal was the critical determinant of diagonal cracking and there appeared to exist a threshold strain of about 150 microstrain above which diagonal cracking occurs.

- The effect of fully reversed displacement cycling on load resistance after diagonal cracking was significant with large amounts of degradation of both load resistance and stiffness. The effect of partial reversals such as half-cycle repetitions was apparently not significant to either the load resistance or stiffness for a small number of repetitions.

8. ACKNOWLEDGEMENTS

The concrete block units and the mason who fabricated the wall panels and prisms were provided by the National Concrete Masonry Association.

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The results from tests on eight ungrouted and unreinforced concrete block masonry walls are presented. The emphasis of the research program is the influence of vertical in-plane compressive stress on the lateral in-plane load resistance of the walls. Each wall has nominal dimensions of 64 in. x 64 in. x 8 in. and is fabricated from similar materials by the same experienced mason. The masonry units are hollow concrete block having a nominal compressive strength of 1800 psi based on the gross area. The mortar was proportioned as a Type S. The walls are tested in the NBS Tri-directional Testing Facility using fixed ended boundary conditions at the top and bottom of the wall. Lateral in-plane displacements were applied at the top of the wall while maintaining a constant compressive axial load. The vertical compressive stress varies between 120 and 500 psi (based on net cross-sectional area) in the test program. The test results indicate that there is a linear relationship between increasing amounts of vertical compressive stress and the resulting increased in-plane maximum lateral load resistance.

12. KEY WORDS (Six to twelve entries; alphabetical order; capitalize only proper names; and separate key words by semicolons)

Axial load; concrete block; masonry; shear; structural testing; walls.

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