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INTERFACE BONDING OF SHOTCRETE REINFORCED BRICK MASONRY ASSEMBLAGES (VOLUME 1)

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ABSTRACT

Nine 3 ft. by 3 ft. shotcrete reinforced brick masonry assemblages and one 3 ft. by 3 ft. brick masonry control specimen were tested under a single reversed cycle diagonal compression load similar to the ASTM E519-74 testing procedures. The interface surface conditions, between the brick and shotcrete were varied. The surfaces of the single wythe of old brick were either dry, wet, or epoxy coated before application of the 3-inch reinforced shotcrete layer. Ultimate load capacities of the specimens were similar; however, specimens with epoxy-enhanced interfaces were the most ductile; the dry brick specimens showed interface bond failure immediately after the ultimate in-plane load was attained.

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1. INTRODUCTION

1.1 Purpose

The purpose of this research was to investigate experimentally the interface bonding of shotcrete reinforced brick masonry assemblages. The objectives were to determine, 1) the adequacy of the connection between the brick masonry and shotcrete, and 2) the extent of the composite action due to the bonding conditions. To satisfy the objectives, three surface conditions are investigated. The variables, i.e., the surface conditions, were 1) a natural bond between a dry brick masonry surface and the shotcrete, 2) a natural bond between a wet brick masonry surface and the shotcrete, and 3) an enhanced bond using an epoxy coating on the surface of the brick masonry at the interface between the brick and shotcrete.

Current repair and retrofit of brick masony incorporates the use of a shotcrete skin with a saturated brick masonry surface. The potential composite action between the brick masonry and shotcrete skin is disregarded during the analysis of the repaired, or retrofitted, masonry elements. Ideally, some degree of composite action would occur between the brick masonry and shotcrete, thereby justifying the incorporation of composite action in analysis.

1.2 Scope

The scope of this project was limited to the testing of nine shotcrete treated specimens: three with dry interfaces, three with wet interfaces, three specimens with an epoxy enhanced interface and one control specimen. The control specimen was a single wythe brick masonry assemblage without a shotcrete treatment. Initially, three control specimens were to be used; however, two of these were broken and replacement was not possible. The variables, the interface condition of each specimen prior to the shotcreting, were used to investigate the bond characteristics and the influence on composite action.

1.3 Background

1.3.1 Brick Masonry Buildings and Brickwork

Many brick masonry buildings of the late nineteenth and early twentieth centuries are still in use today. These buildings are unreinforced brick buildings utilizing weak brick and masonry components relative to today's standards. The brick used in these unreinforced masonry buildings are solid units of clay, either burned or sun dried, about 8 x 3.75 x 2.25 inches in size. The primary function of this brick was to carry the vertical loads; the thickness of the walls being mainly governed by the vertical loads. anticipated.

A great variety of brickwork patterns existed to give the exterior of the masonry buildings a particular look. However, behind this facade, there would generally be a multiple of wythes of brick masonry making up the load carrying part of the structure. By placing the brick endwise or crosswise, a wall could be built of any thickness, that is, any multiple of the width of the brick. Thus the nominal thickness of a wall might be 4, 8, 12, 16, 20, 24, . . . inches. Figure 1.3.1 shows the various arrangements of brick for laying walls of various thicknesses. Figures 1.3.2 to 1.3.4 show walls of various thicknesses and various bonds.

The four inch wall was seldom used for bearing purposes, but was often used for interior non-bearing partitions and as a fire wall. The four inch wall, too, was used as a non-load bearing veneer connected to the structural frame via a metal tie (16).

In the early twentieth century, many cities allowed eight inch thick walls as being sufficient for the usual home; however, other cities required a minimum of twelve inch thicknesses. An objection against the twelve inch walls, for ordinary dwellings, was the loss of floor space. On the other hand, the twelve inch wall had the benefit of insulation properties (16, 20, 23).

The 16 to 24 inch walls were generally used in the heavy duty factory areas where the walls had to carry heavy loads of machinery and were subject to excessive vibrations (16). The arrangement of the brick for these thick walls, as seen in Figs. 1.3.2 to 1.3.4, is more compliated.

It has been noted that poor craftmanship abounded in the construction of the interior wythes of the multiwythed

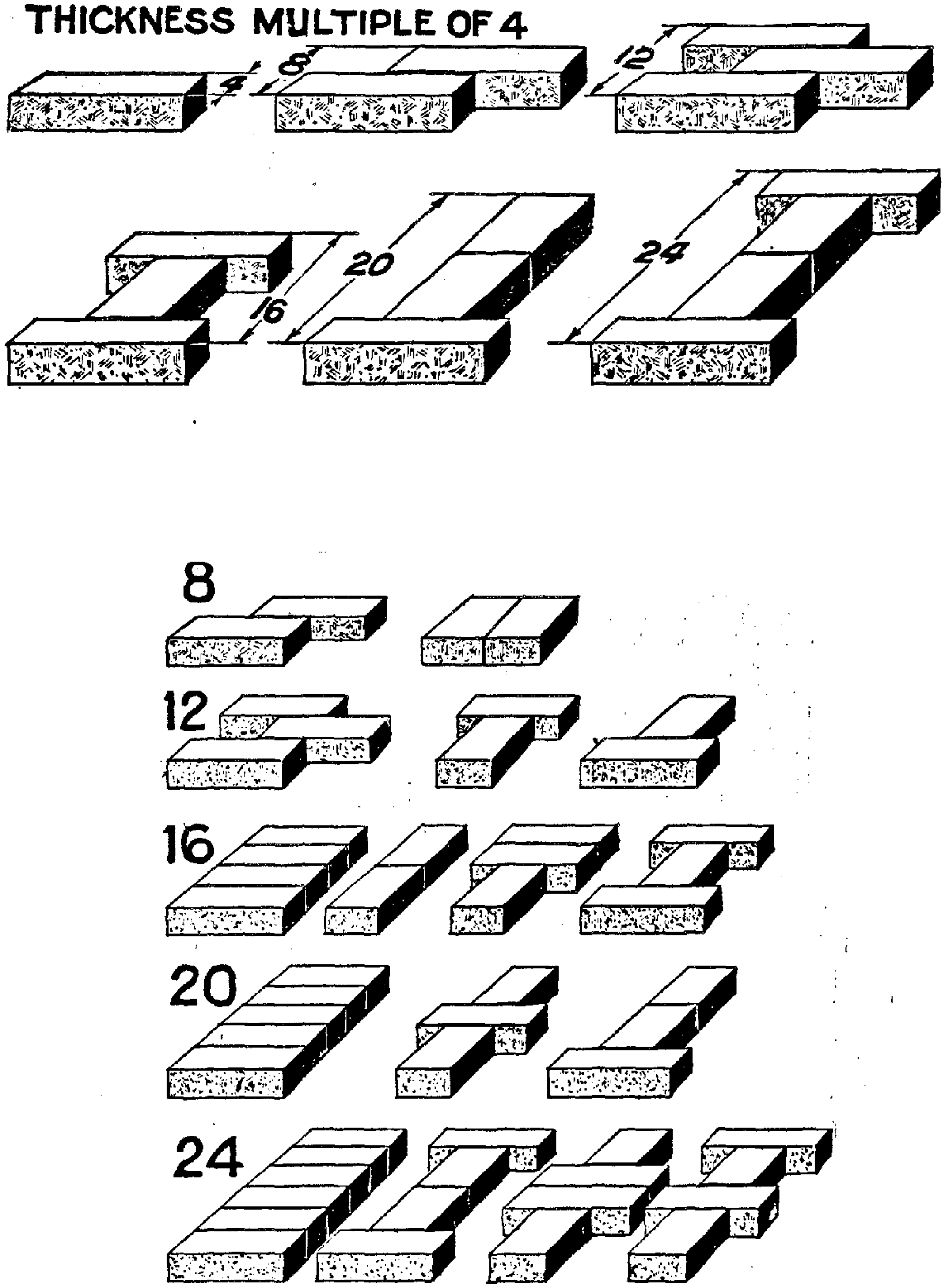


Figure 1.3.1 Thickness is the Multiple of the Width of the Brick, Various Arrangements (16).

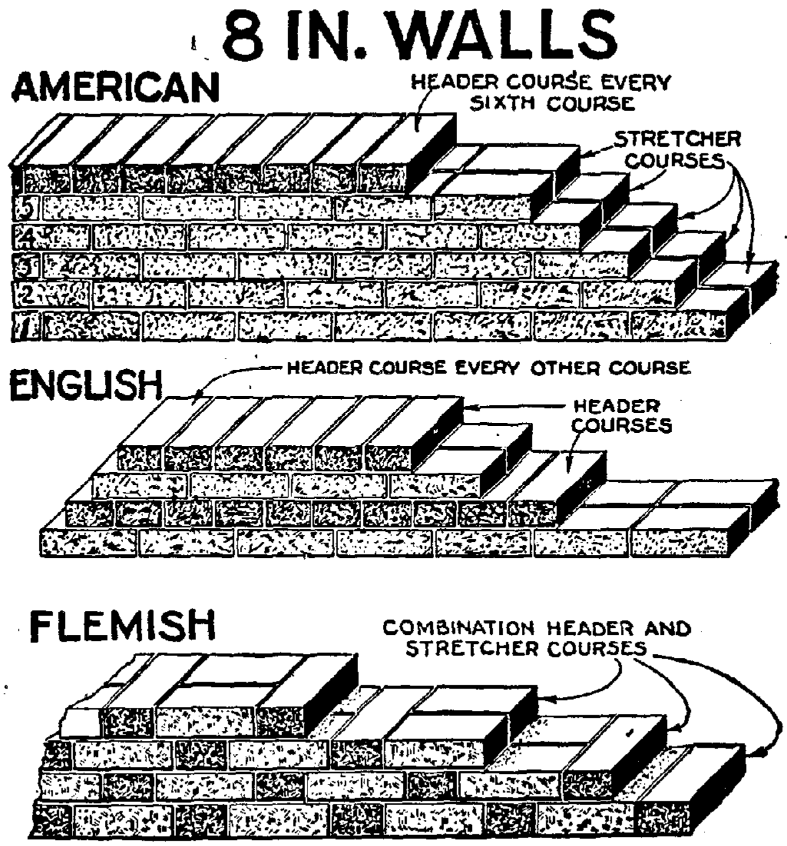


Figure 1.3.2 Eight Inch Walls Laid in Various Boards (16).

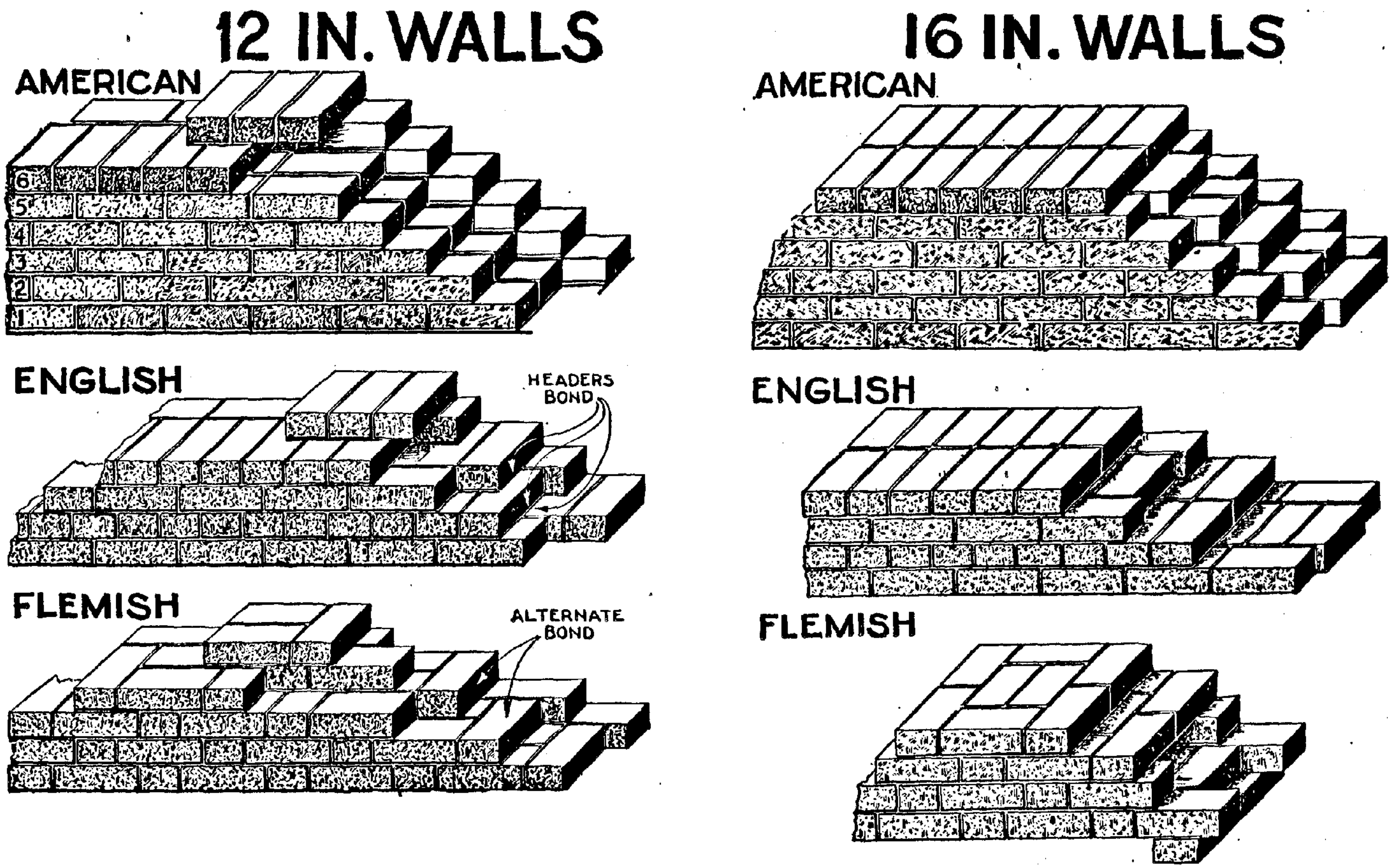


Figure 1.3.3 Twelve and Sixteen inch Walls Laid in Various Bonds (16)

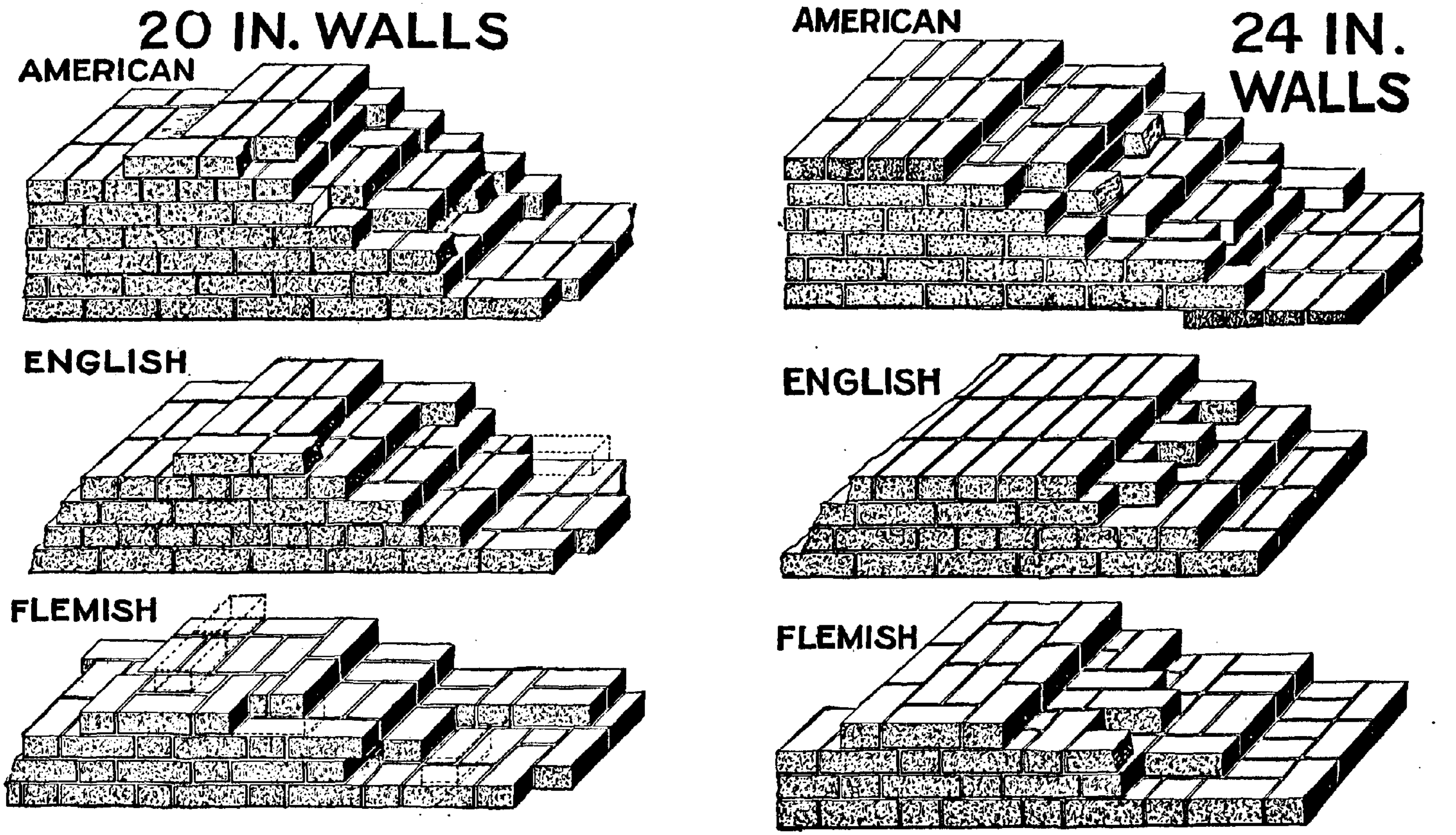


Figure 1.3.4 Twenty and Twenty-Four inch Walls Laid in Various Bonds (16).

brick walls. The mortar for these interior wythes was generally weaker and even sparcely placed. The worst bricks were used for the inner wythes, saving the quality bricks for the face. Occasionally even chips and chunks of brick were used. However, these practices were not always followed as can be seen in Fig. 1.3.5, a photo of a building in Atlanta being torn down. It is common practice to wet bricks prior to use; however, the bricks of the inner wythes were often not saturated with water. The dry bricks would soak up water from the mortar which was needed in the Portland cement mortars for hydration. For walls with Portland cement mortar, this created a much weaker structure. However, if a lime mortar was used, this would not cause a big problem because lime mortar gains strength when the mortar dries out.

In brickwork, the term "bond" applies to the overlapping of the brick, one upon the other, either along the length of the wall or through its thickness in order to tie brick together. The use of overlapping increases the strength of the structure.

The strength and rigidity of a wall with bonding is much greater than that of a wall without it. In practice the lap is made of either 0.25, 0.50, or 0.75 of the bricks length. Figures 1.3.6 and 1.3.7 show the lap positions. By patterning these ratios, an attractive facia can be made as in the American, English and Flemish patterns in Figs. 1.3.2 and 1.3.4.

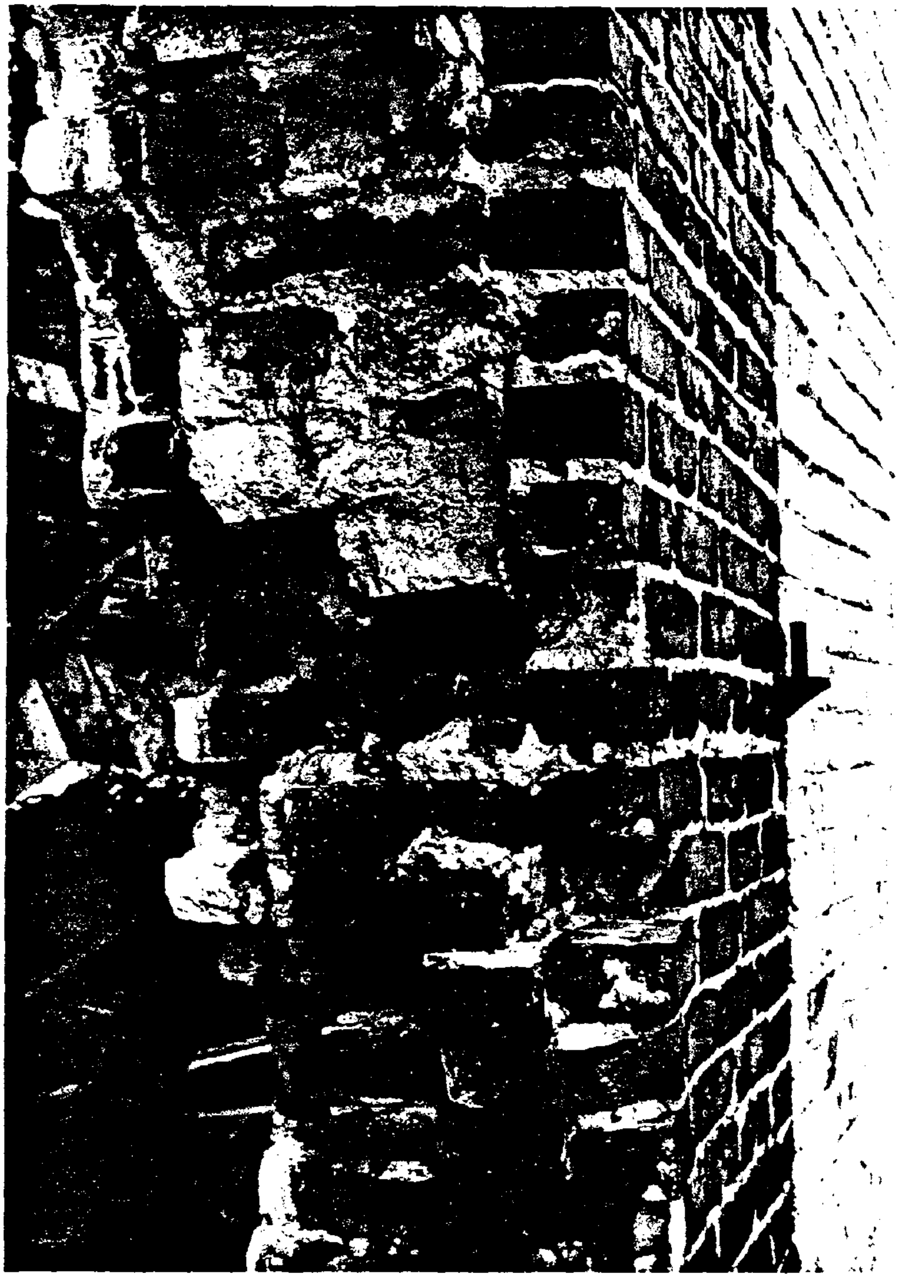


Figure 1.3.5 View of Multiple Wythe Brick Wall of a Building Being Torn Down in Atlanta.

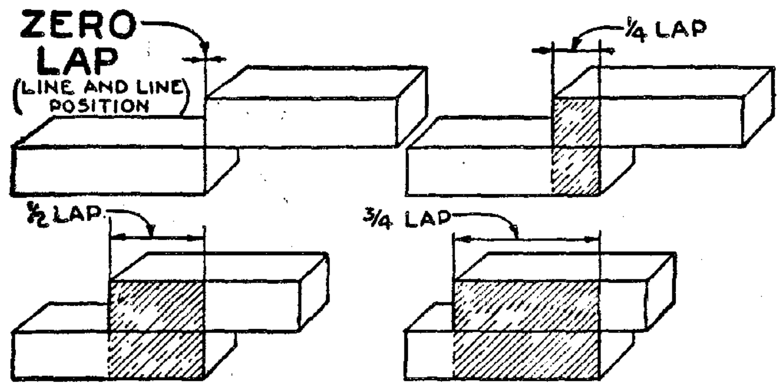


Figure 1.3.6 Lap Positions (16).

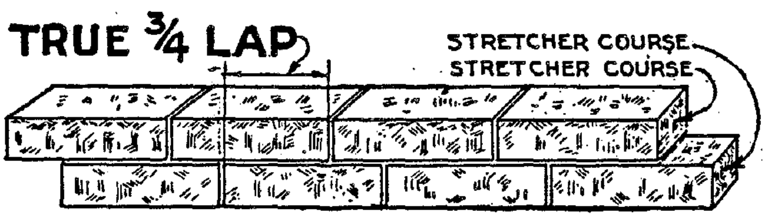


Figure 1.3.7 Example of a True 3/4 Lap (16).

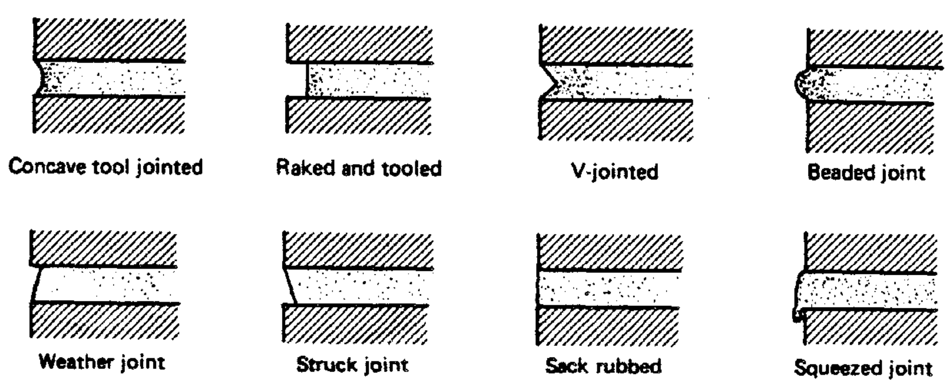


Figure 1.3.8 Types of Mortar Joints (22).

The "stretcher" and the "header" are the two most commonly used terms to describe how brick are laid When the brick is laid lengthwise in the wall, thus showing its long narrow dimension or "face" on the surface, it is called a "stretcher." If its length extends back into the wall, so that its short dimension shows on the surface, it is called a "header." The stretcher secures strength in the length of the wall, and the header, strength across the wall. Figures 1.3.9 and 1.3.10 show the various brick positions and various parts of a brick wall.

As is illustrated in Figs. 1.3.2 to 1.3.4, the numerous types of bonds consist of stretchers and headers arranged in various patterns. The common or American bond is the same as a true stretcher bond except that every sixth course is a row of headers. The English and Flemish bonds are also variations similar to the American bond. These three bond patterns are not the only existing bond patterns, but they are prominent examples of the numerous variety of bond patterns used in history.

A number of mortar joints are utilized in the bond patterns. Figure 1.3.8 shows eight types of mortar joints that are commonly used.

1.3.2 Brick

By definition, a brick is a solid unit of clay, either burned or sun dried, about 8 x 3.75 x 2.25 inches in size. The great variety of bricks may be classed with respect to many variables. The brick may be classifed with respect

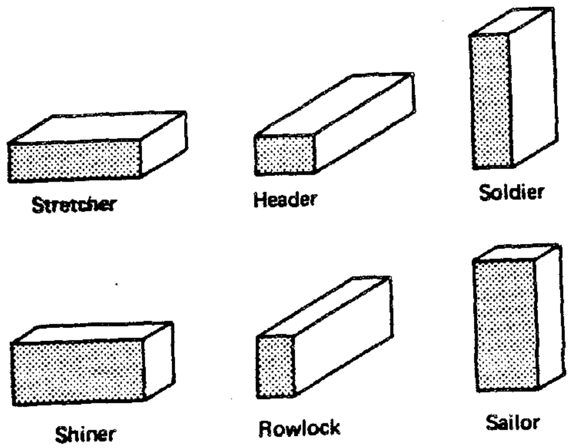


Figure 1.3.9 Terms Applied to Various Brick Positions (22).

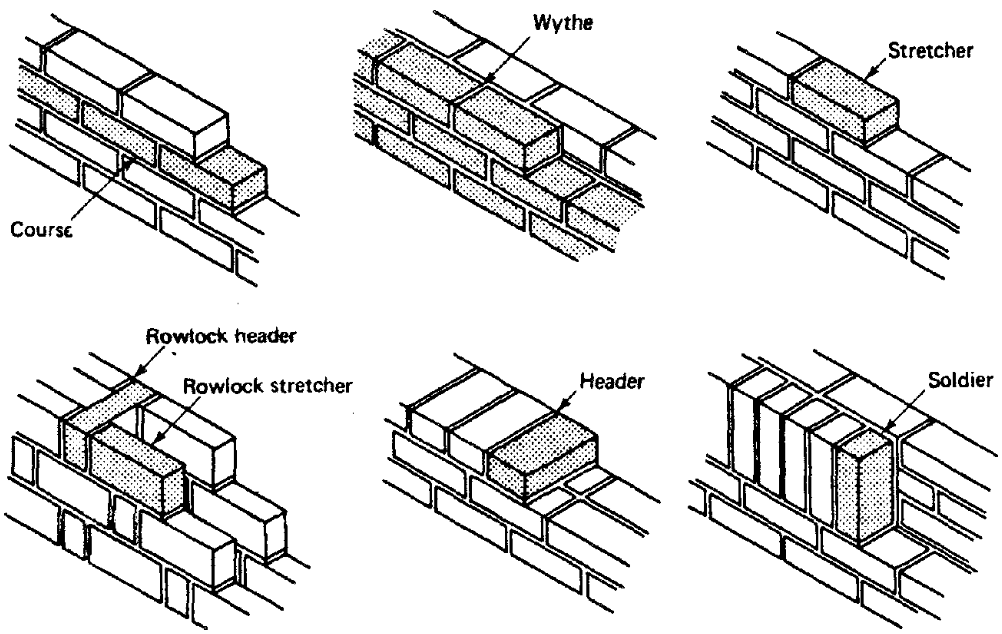


Figure 1.3.10 Terms Applied to Various Parts of a Brick Wall (22).

to the degree of heating, hardness, color, method of making, finish, dispositon of the material, service, material, and the shape of the brick.

The material used for the manufacture of brick is a hydrated silicate of alumina. The silicate of alumina may contain various impurities, such as oxides of iron, calcium, magnesium, sodium, titanium, and potassium. The presence of these impurities in varying amounts will cause deviations to occur in both the chemical composition and physical properties of the clay brick.

Natural clay, which is very common in the earth's surface, occurs in three principal forms, which are 1) surface clays which are found near the surface of the earth, 2) sedimentary clays such as shales which are subjected to high pressures causing a hardened form, and 3) fire clays which occur at greater depths and have a high resistance to fluxing at high temperatures (vitrification).

The most significant properties that give rise to clays suitability for the manufacture of brick are its plasticity, tensile strength, fusibility, and shrinkage. The strongest brick clays, or the brick possessing the greatest plasticity and tensile strength, are those brick which contain the highest percentage of the hydrated aluminum silicates.

On heating, all clays lose their plasticity and cannot regain it. Therefore, on burning, clays are converted into rigid bodies.

There are four distinct methods employed in the manufacture of brick in which the equipment required depends largely upon the nature of the raw material and to some extent upon the desires of the manufacturer. The manufacturing processes are as follows: 1) hand method, 2) soft mud method (obsolete), 3) dry press method, 4) stiff mud method.

The hand method for making brick is used in isolated or exceptional cases, usually at a very small scale and where labor is cheap. In this process the clay is taken directly from the clay bank and thrown into a pit with the proper amount of water. A large wheel fixed to a shaft and drawn by a horse or several laborers passes through this pit and thoroughly mixes the clay and water. The tempered clay is then pressed by hand into a wooden or metal mold or four sided case. Figure 1.3.11 shows a home made brick mold and strike. The mold being made of the desired shape and size, allowing for the shrinkage of the brick in drying and firing. As illustrated in Fig. 1.3.12, the molder dips the mold in water, places the mold in sand to prevent the clay from sticking, packs the clay into the mold, scrapes off the excess clay, and then strikes off the top smoothly. The brick is then turned out and pre-dried. These bricks can later be sun baked or kiln baked in small kilns. The hand made bricks are easily identified by their characteristic scar marks due to the wood cases,

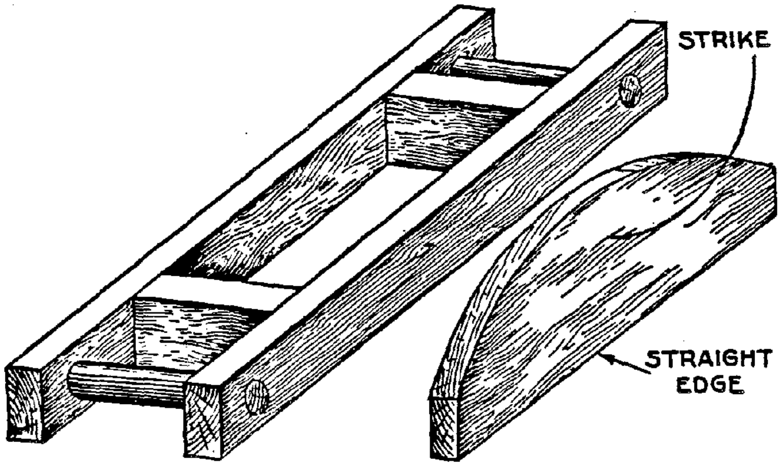
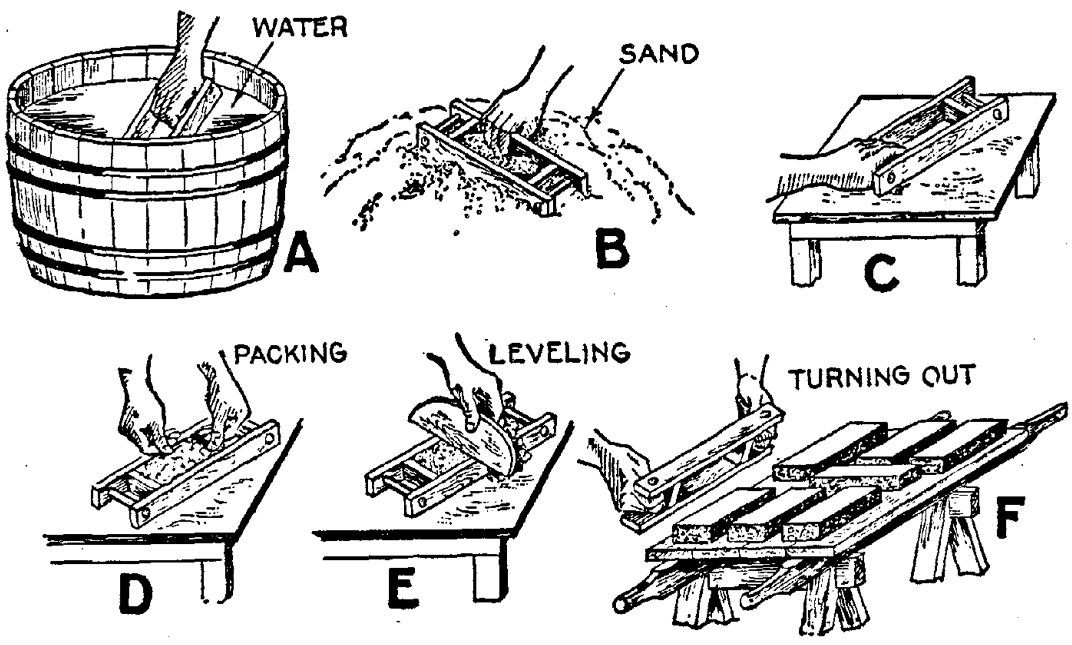


Figure 1.3.11 Homemade Brick Mold for Single Brick, and Strike (16).



Making a brick by hand. A) wetting the mold; B) sanding the mold; C) placing the mold on table; D) packing clay in mold; E) levelling with strike; F) turning out brick on board.

Figure 1.3.12 Making a Brick by Hand (16).

their deformed corners, and their soft surfaces (16, Appendix A-4).

The soft mud process is used for the production of brick made out of clays that contain too much water in their natural state (20 to 30%). This process is simply a method of forming the brick in a mold under pressure. The soft mud machine can mold four to six bricks in one operation. The brick formed by this process are uniform in size and resemble the bricks made by the hand process.

The dry press method is generally used in areas where the clay cracks easily on drying if much water is added to it. In this process, the clay is ground and taken directly to the machine without any water added to it. This machine is a heavy press, and using an enormous amount of pressure, it compresses the clay into a brick. Brick made by this process is characterized by being very dense and having very smooth surfaces and sharp corners. No predrying is needed in this process, i.e., the bricks are taken directly to the kilns.

The stiff mud method is a process whereby the ground clay is mixed thoroughly in a pugmill or in a wet pan. From the mill the clay is taken and put directly into an auger machine, where it receives further mixing and tempering. By means of a large auger, the clay mixture is extruded through a die. The clay comes out as a continuous bar and is ready to be cut into the desired lengths. The cutting is generally done on a cutting table equipped with high tensioned cutting wires. The wires leave a rough side where they cut through the clay. This side of the brick is generally used for the mortar joint. As soon as the clay product has been formed, it is ready for drying. Older plants resort to free air drying, whereas modern plants use drying floors or tunnel dryers.

The burning or firing of bricks is the most important factor in the production of bricks. Their strength and durability depends on the character and degree of firing to which they have been subjected. The action of the heat brings about certain chemical decompositions and recombinations which alter the physical characteristics of the dry clay, thus producing desirable structural qualities.

Various types of kilns have evolved in the history of brick making; although they all perform the same task. Whether they be updraft, downdraft, or continuous kilns, their results are generally the same.

1.3.3 Mortar

One of the most essential elements in brick masonry is the mortar that binds the brick elements together to form a structure. There are several types of mortar which are composed of different materials, differing ratios of materials, and different strengths.

The principal mortars are lime mortar, lime cement mortar and cement mortar. There are two types of cement mortars, natural and portland cement. Lime mortar is composed of sand, and either slaked lump lime or hydrated lime. The sand must be clean and consist of sharp angular particles free from vegetable matter, loam, large stones, and dust. Slaked lump lime is formed by adding water to lump lime. The lump lime is the product of calcining limestone in kilns. Hydrated lime is produced by adding the water during the grinding and manufacturing process where it is converted into hydrated lime, without necessarily saturating it with water.

In the 1920's lime mortar was recommended for ordinary house construction because it was felt that a strong bond between brick and mortar was not necessary (16). However, according to Edwards (Appendix A-7), lime mortar was used for more than the construction of ordinary houses. An example of a lime mortar mix design for the 1920's was:

> 1) five bushels of fresh burned lime or its hydrated equivalent
>
> 2) one cubic yard of sharp river sand
>
> The supply of fresh lime mortar to be kept well in advance of the work so that none less than two weeks old be used. Only enough water to make a workable mix should be added. (16)

Evolving from the lime mortar was the lime cement mortar. The lime, as before, gives the mortar mix workable properties and some strength, while the cement provides the basic strength. The lime cement mortar was quite prevalent in the structures of the early 1900's and was especially used in the Middle West of the U.S. (16). An example of a mix design for lime cement mortar is:

1 part lime

1 part cement

4 to 6 parts sand

The ingredients are to be mixed well and only enough water to provide a workable mix should be added. This mortar should be used directly after mixing (16).

The lime cement mortar is slightly stronger than the lime mortar; however, the increase in strength is small when compared to the strength of the third mortar type, the Portland cement mortar. The Portland cement mortar sets faster and is harder to work with than the two previous mixes. One reason for the difficulty in working with the Portland cement mortar is that it contains no lime. Lime gives plasticity to and delays the setting time of mixes that incorporate it. However, Portland cement mortar is much stronger than the lime or lime cement mortars, and was thus recommended in uses where extra strength was required or in heavy bearing situations. Edwards (Appendix A-7) noted that the excessive strength acquired from Portland cement mortars is so large that it at times caused cracking to occur through the brick rather than through the mortar joints. A typical mix for Portland cement mortar in the 1920's was:

1) one part American Portland cement showing a tensile strength of 500 lbs. per sq. in. on seven days.

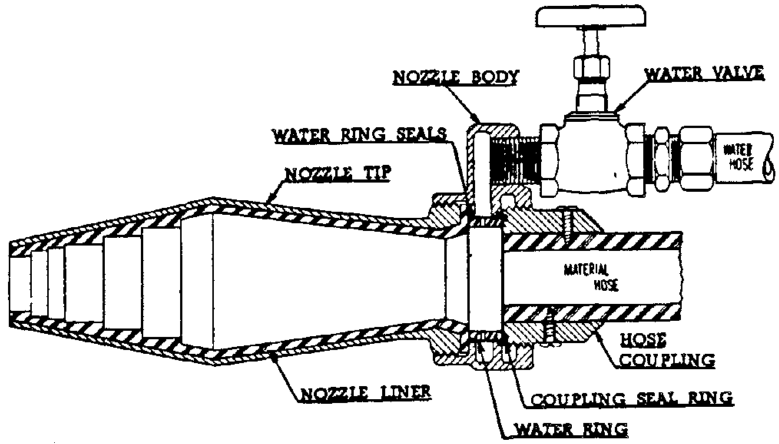
2) three parts of clean sand

Enough water to be added to create a somewhat workable mix. The cement mortar is to be mixed fresh each day just before being used and in such quantities that none shall be left over at the end of the day's work (16).

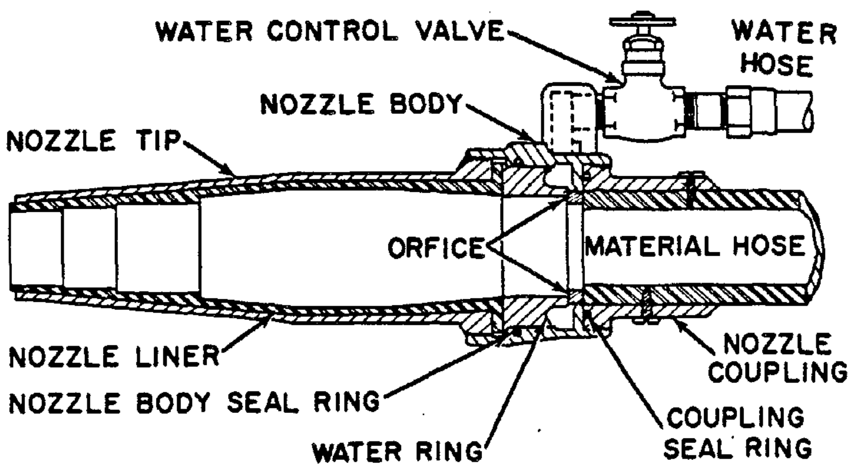
1.3.4 Shotcreting

Shotcrete, or gunite to which it is sometimes referred, is a pneumatically applied mortar or concrete. Shotcrete is a mixture of portland cement, aggregate, and water, shot into place through a hand-held nozzle at high velocity by compressed air.

The two basic shotcreting processes are the wet and dry mix processes. The wet process requires that the ingredients, cement, aggregate and water, be mixed prior to pumping to the nozzle. The dry mix process requires that the cement and damp aggregate be premixed and pneumatically conveyed through the delivery hose to the nozzle where the operator adds the remainder of the water. Figures 1.3.13(a) and (b) show two typical nozzles used in the dry mix shotcreting process. The water enters at the beginning of the nozzle and combines with the cement and aggregate in the mixing chamber. Hydration of the cement begins at the moment the



(a) Balloon-type nozzle tip



(b) Modified nozzle tip

Figure 1.3.13 Dry-Mix Nozzles (4).

water is added at the nozzle. The dry process generally is superior to the wet mix process because a competent nozzleman, by controlling the water added, can produce an excellent finished mix. (The wet mix often is less expensive than the dry.)

A typical arrangement (plan view) for the equipment used in the dry mix shotcreting process is shown in Fig. 1.3.14(b). Figure 1.3.14(a) is a vertical section of a typical double chamber gun used for the dry mix process. The cement aggregate mixture is added to the upper chamber. The material is transferred to the lower chamber where it is then pneumatically conveyed through the material hose to the nozzle.

Shotcrete, produced by the wet and dry processes, is designed to be sufficiently stiff that it supports itself without sagging or sloughing from a vertical surface. Because of the stiffness of shotcrete at impaction, shotcrete is well adapted for thin applications, even less than one quarter of an inch. However, thick layers can be obtained by the application of numerous thin coatings.

Shotcrete is being used for construction of thin lightly reinforced structures, canal and tunnel linings, swimming pools, and prestressed tanks (3, 4, 12, 21). Shotcrete is used in the repair, restoration, strengthening and waterproofing of existing concrete and masonry structures (1, 3, 4, 12, 15, 18, 19, 24).

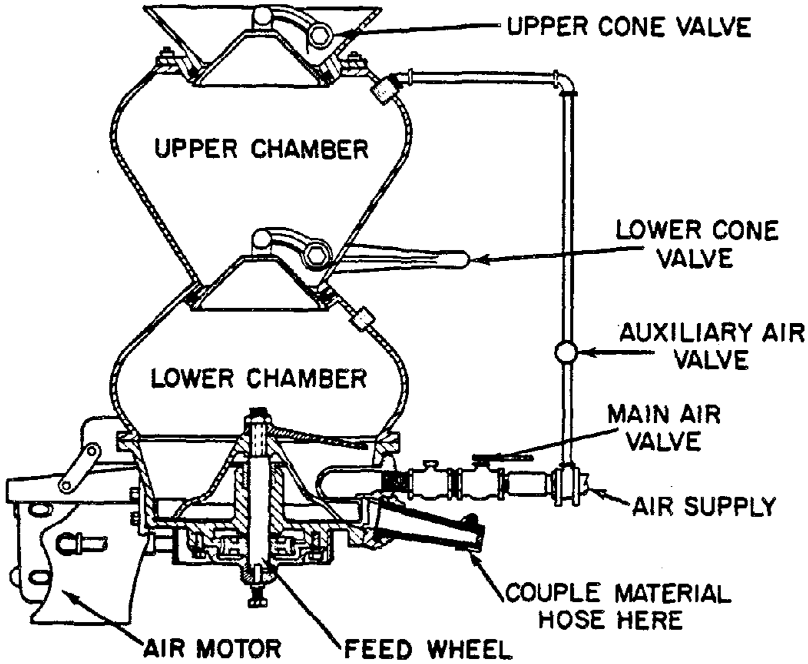


Figure 1.3.14 (a) Vertical Section of a Typical Double Chamber Gun (4).

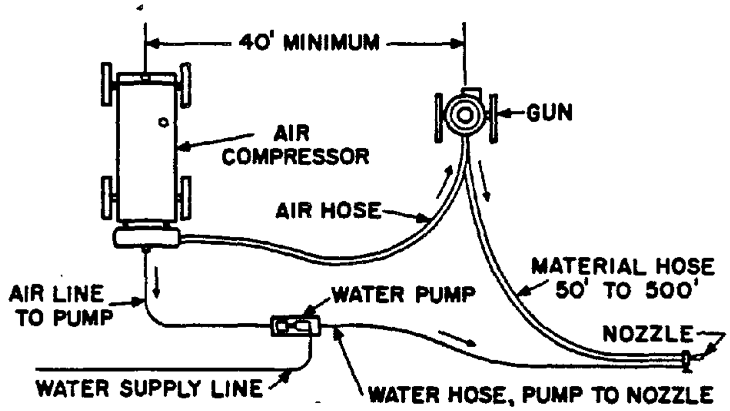


Figure 1.3.14 (b) Typical Arrangement (Plan View) of Equipment for Shotcreting (4).

When the shotcrete mixture, produced by both wet and dry processes, leaves the nozzle at high velocity and strikes the hard surface which is to be covered, the coarser particle ricochet (rebound) from the surface. The rebound of material may alarm some users, however, initial rebound of the coarser particles leads to a thin bond coat of fine grout on the surface being treated. After a thin layer of the bond coat has built up, it acts as a cushion to reduce the amount of rebound and to ensure build-up of the shotcrete layer. Excessive rebound should be limited to only the initial passes, otherwise an undesirable rich mixture will be produced.

The optimum distance that the nozzle should be held from the point of application varies depending on the nozzle tip size, hose size, production, air compresssor size, wind velocity, skill of nozzleman, and type of surface being coated. On large, flat, unobstructed surfaces with a 2-in. hose and a 600 cu. ft. per min. or larger air compressor, the nozzle can safely be held back 6, 8, or 10 feet to produce satisfactory work. For reinforcing bar or wire encasement, the nozzle should be held as close as practical. When shooting a finish flash coat over work which has been troweled, screeded, or otherwise worked, the further back the nozzle, the smoother and more attractive will be the finished surface. The nozzleman should hold the nozzle at 90 degrees to the surface and at 45 degrees relative to the intersecting planes of a corner.

For dry-mix shotcrete, the normal ingredients are Type I Portland cement, damp angular sand, water, and possibly admixtures. Different projects require different mixes of sand and cement. The basic mix, however, is considered to be 3.5 parts sand to 1 part Portland cement, with the sand having a fineness modulus of 2.42 and a moisture content of 3 to 5 percent. Mixing damp sand with cement causes the cement to adhere to the larger sand particles, thus preventing segregation in the gun chamber. The grading for fine aggregate should comply with the ASTM C33 "Specifications for Concrete Aggregates" (8). This requires the folowing grading for fine aggregates:

Sieve Size, U.S. Standard Square Mesh	Percent Passing by Weight
3/8 in.	100
No. 4	95 - 100
No. 8	80 - 90
No. 16	50 - 85
No. 30	25 - 60
No. 50	10 - 30
No. 100	2 - 10

For jobs requiring large aggregate, another gradation specification should be followed.

For the use of shotcrete in engineered applications, the strength of the finished product is needed for design. The most reliable determination of the quality of shotcrete is to obtain core samples from a typical gunned section (4, 12, 20). The test cores taken from a gunned area should have a minimum diameter of 3 in. and a L/D ratio of at least 1 if possible. Core strength should be converted for L/D as described in ASTM C42 "Obtaining an Testing Drilled Cores and Sawed Beams of Concrete" (10).

If taking cores from the structure is not feasible or desirable, small unreinforced test panels, at least 1 ft. square and 3 inches thick, should be periodically shot. Cores or cubes can be extracted for compressive tests and visual examination as described in ACI Publication SP-14, "Shotcreting" (4). Another method that is not generally recommended is the fabrication of cylinders by shooting shotcrete into hardware cloth cylinder molds (4). To test these cylinders, the hardware cloth is removed after 24 hours and the cylinders are tested in compression. If cubes are taken as test samples, the cube strength can be presented directly or converted to a cylinder strength by multiplying the cube strength by 0.85.

Additional information on shotcreting can be obtained frm the ACI Publication SP-14, "Shotcreting" (4).

1.3.5 Reinforcing for the Shotcrete

Shotcrete, a specialized concrete, has low tensile strength and requires reinforcing steel to carry tensile forces. It is emphasized that the soundest shotcrete will be obtained when reinforcing steel is designed and placed to cause the least interference with placement of the shotcrete (4). Depending on the thickness and nature of the work, reinforcement may consist of either round bars, welded wire mesh, or any of the great variety of metal lath used in the process of plastering. Small bar sizes should be used, with a #5 bar being the normal maximum size (4).

Sufficient clearance should be provided around the reinforcement to permit complete encasement with sound shotcrete. The clearance needed depends on the maximum size of aggregate in the mix and size of reinforcement. The minimum clearance between the reinforcement and the form or other back up material may vary between 0.5 inch for the case of a mortar mix and wire mesh reinforcement to 2 inches for the case of a concrete mix (having large aggregate) and #5 reinforcing bars (4). Minimum cover should also comply with the specification governing the work or applicable building code.

1.3.6 Surface Conditions

Shotcrete, being a portland cement product, acquires strength through a hydration process. A low slump shotcrete mix uses the minimal water content required for the hydration of the portland cement. Because the water content is initially the bare minimum, any loss of water at the interface between shotcrete and brick could cause a weak bond between the shotcrete and brick and a layer of low strength shotcrete at the interface. If a shotcrete treatment were applied to a totally dry brick wall, the brick would soak up all available water causing an extremely weak bond between the shotcrete and brick, and a weak layer of shotcrete at the interface.

A means of reducing the loss of water from the shotcrete at the interface is to saturate the brick masonry that is to be treated prior to the application of the shotcrete. The water content at the interface would be unaltered, and the shotcrete could develop its full bond and strength characteristics.

Epoxy glues and pastes are used to enhance the interface bonding for materials using hydraulic cement (18). The epoxy is coated on the existing surface prior to the application of the shotcrete. Some epoxies are insoluble in water, curing in a water environment. The epoxy is supposed to increase the adhesive force of the shotcrete. Epoxies could also create a vapor barrier at the interface, reducing any loss of water from the shotcrete into the brick masonry.

Epoxy coatings are expensive, enough epoxy to coat 100 square feet could easily cost 55 dollars. The net increase of adhesive bond at the interface between the shotcrete and brick may not outweigh the epoxy costs.

1.3.7 Strength of Brickwork in the 1920's

During the 1920's, the Common Brick Manufacturer's Association, working with the U.S. Bureau of Standards, conducted investigations into the properties of brickwork (15). The intent of these investigations was to arrive at exact engineering data upon which design formula could be used.

Gram of the 1927 edition of "Audel's Masons and Builders Guide" (16), presents investigations on three factors that can have an influence on the strength of brickwork. The investigations deal with the strength of individual brick, mortar and the bonding of brickwork.

Table 1.3.1 shows the results of compression tests on individual brick from three geographical areas. The ultimate strengths between geographical areas and types of brick vary between 5.2 ksi to 20.0 ksi. The extreme differences of load capacity between bricks indicates the need for builders to explicitly specify the type of brick to be used in construction.

"Audels Guide" (16) also presents the findings of tests at Columbia University by Prof. Macgregor (16). Macgregor's tests indicated that the brickwork laid in cement lime mortar (using a 1-1-6 mixture) was stronger than the straight 1-3 cement mortar, although the 1-3 cement mortar cubes were considerably stronger than the cement lime mortar cube. Macgregor (16) indicated that the lime cement mortar's favorable

Table 1.3.1 Compressive Strength of Individual Brick (16).

(tested flat)

BRICK	Lbs. per sq. in.	Tons per sq. ft.
Arkansas		
Red grade 1	12253.	953.
Red grade 2	11966.	860.
Red grade 3	5620.	406.
Illinois		
Shale building brick	10690.	770.
Underburned common	3920.	280.
Kentucky		
Dark gray	20030.	1442.
Gray	16793.	1210.
Dark green	7243.	521.
Red	5290.	380.

Ultimate load capacities

from "Audels Masons and Builders Guide"

response could be attributed to the increase in plasticity given to it by the lime. This increase in plasticity further resulted in a more thorough bedding of the brick, and a more complete filling of the joints.

In Table 1.3.2 are the results presented in "Audels" (16) by Rudolph P. Miller on tests of old brickwork from the Raquet and Tennis Club in New York City. The test samples varied in header brick orientation. The average ultimate strength of the valid specimens was 1,781 lbs/sq. in. It was concluded at that time, that the number of header courses used did not have a positive effect on the compressive strength of the pier.

Gram, in "Audels" (16), also presented some tests on the influence of the strength of mortar and piers. In Fig. 1.3.3, it shows that the lime cement mortar is comparatively equal in strength to the plain cement mortar.

1.3.8 Review of Previous Research

To the best of the author's knowledge, no laboratory study has been published on the investigation of the interface bonding of shotcrete reinforced brick masonry.

Numerous unreinforced brick masonry buildings and buildings with masonry walls have been strengthened to resist anticipated earthquakes (18, 19) and many buildings of unreinforced masonry or with masonry walls have been repaired and strengthed after a damaging earthquake (1, 15, 18, 24). To strengthen the masonry, the use of a reinforced shotcrete

Table 1.3.2 Tests on Old Brickwork from Raquet and Tennis Club, New York City (16).

by Rudolph P. Miller

	Height		Area in Compression	Ultimate Strength		First crack at
Specimens	Inches	Courses	sq. in.	Total lbs.	PSI.	PSI.
1	23.5	9	193.60	268,970	1389	516
A	27.2	10	206.25	181,000	877	640
B	24.5	9	186.34	390,000	2093	1588
C	21.7	8	196.00	365,000	1862	1275

Average Ultimate Strength of specimens 1, B and C 1781 psi.

The bearing of pier A was uneven, and not used.

from "Audels Masons and Builders Guide"

Table 1.3.3 Influence of the Strength of Mortar on Piers (16).

Pier No.	Compressive Strength of Bricks psi	Compressive Strength of Mortar psi	Mortar Mixture ratio	Compressive Strength of Piers psi
1	4040	0	Dry sand	740
2	4040	38	1 lime; 3 sand	740
3	4040	355	2 lime; 1 cement; 9 sand	1420
4	4040	695	1 lime; 1 cement; 6 sand	1840
5	4040	1280	1 lime; 2 cement; 9 sand	1700
6	4040	1640	2 lime; 1 cement; 7 sand	1930
7	4040	2620	1 cement; 3 sand	1980

28 day tests

from "Audels Masons and Builders Guide"

skin has been popular. The shotcrete treatment involves removing inner wythes of brick from the masonry wall, drilling holes into the wall, grouting of reinforcing dowels into these holes, placement of vertical and horizontal reinforcement, wetting of the brick, and the application of shotcrete in place of the masonry wythes that were removed.

The shotcrete reinforced wall is analyzed as a reinforced concrete wall, typically vertical loads are carried by the masonry while all lateral loads are carried by the shotcrete. However, at times, the structural capacities of the masonry are disregarded, relying on the shotcrete skin to carry all loads. Bending characteristics and resulting composite action between the shotcrete and brick masonry is not considered. The dowel connection is provided to assure the bricks from falling during subsequent earthquakes.

An example of the use of shotcrete in structural rehabilitation of brick masonry is the retrofitting of the California State Capitol (19). The strengthening scheme used for the retrofit of the Capitol was as follows: 1) the removal of two interior wythes of brick; 2) drilling holes into the remaining wall and epoxying rebar dowels in the holes; 3) placing inplane rebars in both directs; and 4) shooting 12 inches of shotcrete in place of the removed wythes of brick.

1.3.9 Utilization of ASTM E519-74 Testing Procedures

The ASTM E519-74 test, "Diagonal Tension (Shear) in Masonry Assemblages" (11) is illustrated in Fig. 1.3.15. The E519 test has been used as an economical replacement for racking shear tests on masonry walls. The test method avoids the need for a hold down force to prevent rotation of the specimen as required in the racking load test prescribed in ASTM Method E72 (6). The E519 method approximates the inplane loading of a masonry wall and was intended for evaluating the effects of variables such as type of masonry unit, mortar, workmanship, etc. However, the use of this test on shotcrete treated masonry specimens will indicate relative increases in load capacities, bonding at the interface of the two materials, and the composite action of the brick and shotcrete under conditions similar to a plain masonry assemblage under inplane loads.

Because the strengthening technique involves the use of materials with different modulus of elasticity, flexure of the specimens was considered. Other than the typical horizontal and vertical gauges, as recommended in the testing procedures (Fig. 1.3.16b), lateral deflection gauges used to measure lateral deflection and delamination of the specimen during testing (Fig. 1.3.16a).

The method calls for 4 ft. by 4 ft. masonry assemblages, but allows the use of a smaller size specimen if the testing equipment will not accommodate the 4 by 4's. Due to minimal

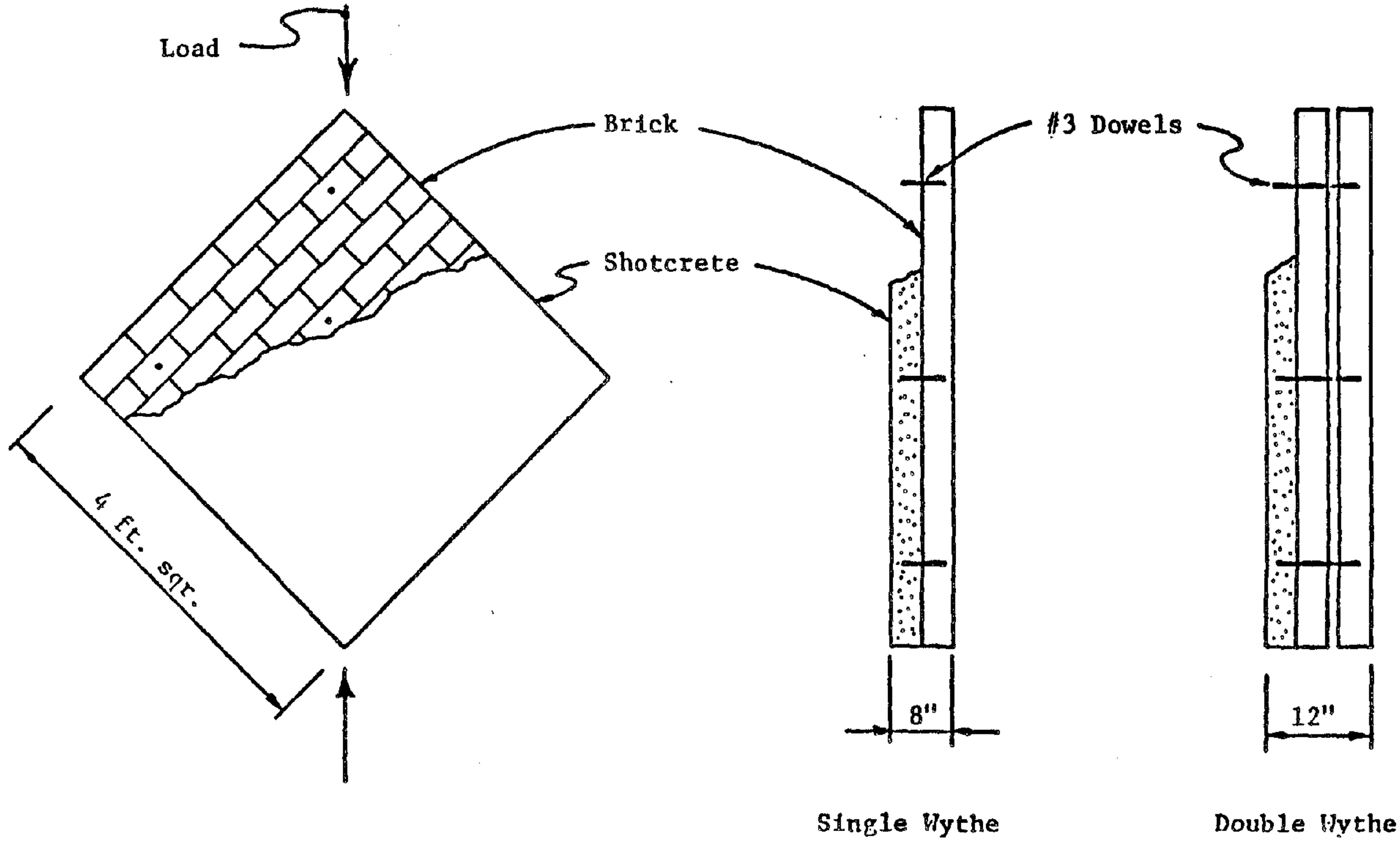


Figure 1.3.15 Diagonal Tension (Shear Test of Composite Brick-Shotcrete Specimens (18).

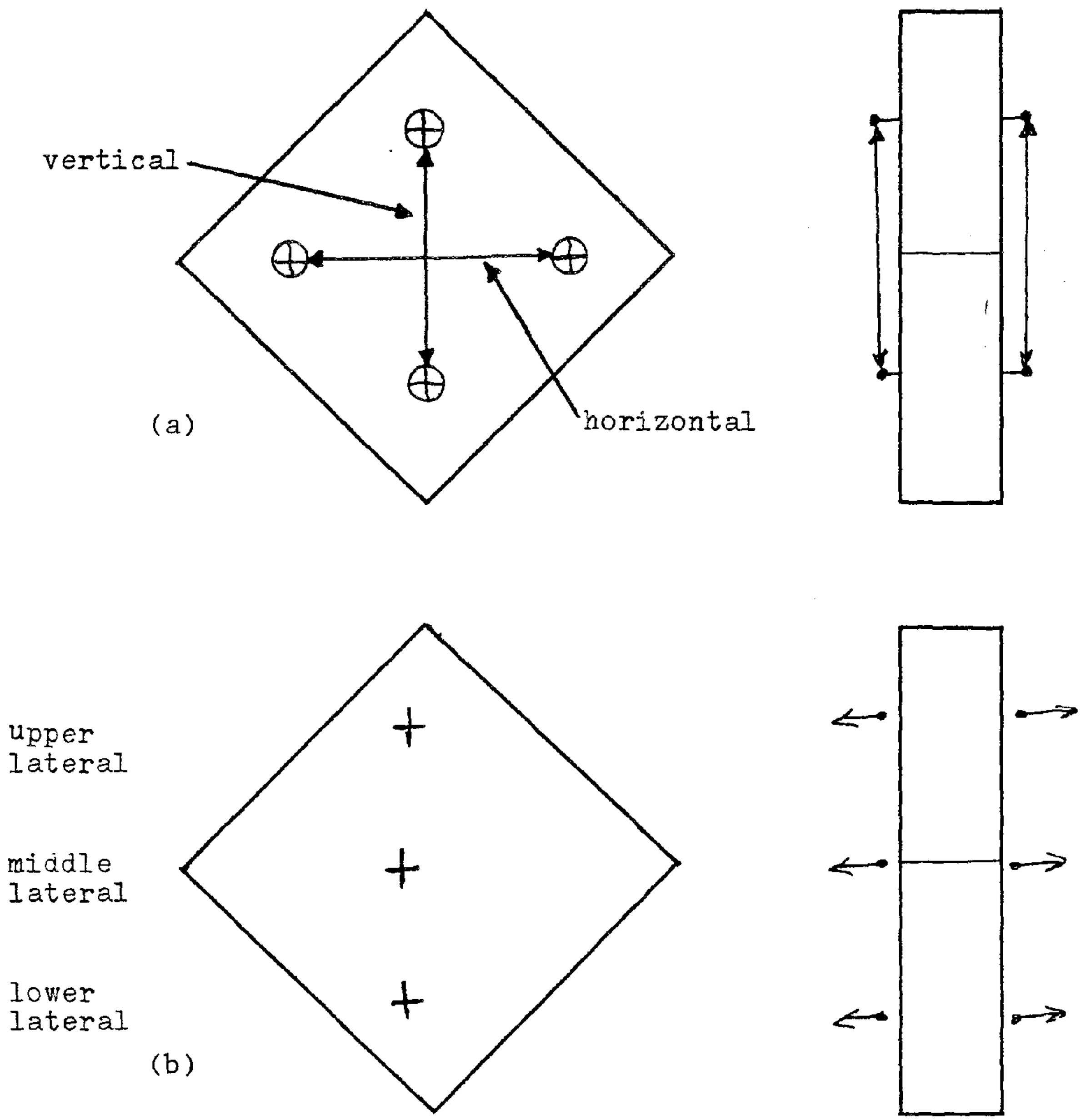


Figure 1.3.16 Locations Where Deflections are Measured During Testing.

research in size effects, there is no correlation between small-scale tests and the recommended 4 by 4 specimens (11, 15). This research project uses 3 ft. by 3 ft. specimens for the investigation. An accompanying project uses 4 ft. by 4 ft. specimens to provide a correlation.

The method calls for mortar cubes to be tested at the same time as the specimens; however, this procedure was not followed due to time restrictions. Loading shoes, like that in the specifications, were made. A cement grout was used to assure adequate bearing.

The specimens were built like those for the E519-74 test and rotated by the 45° with a large "C" clamp. The 450,000 lb. Riehler test machine was used to provide the compressives loads required.

2. EXPERIMENTAL PROGRAM

2.1 Specimen Design

To investigate the interface bonding of shotcrete reinforced brick masonry, 3 ft. by 3 ft. nominal size brick masonry panels were used to model an inner wythe of an unreinforced brick masonry wall. The brick masonry assemblages were modeled after an inner wythe of an unreinforced brick masonry wall because the shotcrete reinforcing treatment typically has been used to replace the interior wythe of brick such that the shotcrete interfaces with an inner wythe. As presented in Section 1.3.1 of this report, many unreinforced brick masonry buildings of the late nineteenth and early twentieth centuries are still in use today. Many of these masonry buildings are in need of repair and strengthening. Because of the number of old masonry buildings that are in distress, the structural properties of the brick masonry assemblages were modeled after the old unreinforced brick masonry of the late nineteenth and early twentieth centuries.

Old solid soft bricks were used in the masonry assemblages to model the predominant type of brick used in the late nineteenth and early twentieth centuries (Appendix A). The old solid soft brick were used also because the results from tests incorporating these brick would be conservative with respect to the quality and strength of bricks that were produced in the 1970's and 1980's. The use of core bricks was considered; however, upon subsequent interviews (Appendix A), it was revealed that very few core bricks were produced, if any, during the time period being modeled. Old solid bricks for the construction of the masonry panels were obtained from the Atlanta Civic Center which was built between 1928 to 1932 and demolished by Atlanta Wrecking and Salvage Company in 1981.

As is revealed in a number of personal interviews (Appendix A) and research references (16, 20, 22), the mortar used in construction of masonry buildings in the early twentieth century was extremely weak. The typical mortar used in the early twentieth century contained a mix of slag cement, lime, and sand. The mix used, typically, was 1, 1 and 4 to 6, respectively. To model the weak mortar, Magnolia Mortar mix was used. Magnolia Mortar mix was first used in Atlanta in the 1920's. The mortar mix could be compared with the standard type 0 mortar, which is extremely weak by today's codes.

As described in Section 1.3.4, two types of shotcreting processes were reviewed as possible methods for treating the specimens. The dry mix process was chosen and used to treat the masonry assemblages because of the dry mix's superior finished properties as well as the techniques low waste percentage, finishability, and good quality control.

A 3" thickness of shotcrete was used on the assemblages. John Kariotis, a structural engineer in Los Angeles, California, recommended the use of a 3" to 4" shotcrete thickness because the 3" to 4" treatment would approximate the weight of the interior wythe which would be removed so the total mass of the building is unchanged. A 3" shotcrete treatment was used also because the 3" thickness would give the final specimens symmetrical cross-sections.

6 x 6 - W3.5 x W3.5 welded wire fabric was used to reinforce and help hold the shotcrete during placement. The welded wire was chosen because it was easy to cut into appropriate sizes which fit the specimens and because it was easy to position in the form work requiring no additional supports to hold individual bars in place.

To satisfy the objectives of this research, three shotcrete-brick surface conditions were investigated. The variables, i.e., the surface conditions, were 1) a natural bond between a dry brick masonry surface and the shotcrete, 2) a natural bond between a wet brick masonry surface and shotcrete, and 3) an enhanced bond using an epoxy coating on the surface of the brick masonry at the interface between the brick and shotcrete. Three specimens for each of the three variables were constructed, along with three control specimens that were not treated with shotcrete. Two of the control specimens were broken before testing, and replacement was not possible. A total of ten specimens were tested, nine shotcrete treated and one control. Sikadur 370, an insoluble epoxy was used for the epoxy enhanced bond.

In place of the racking load test prescribed in ASTM Method E72(6), the ASTM E519-74 test, "Diagonal Tension (Shear) in Masonry Assemblages" (11) was used to test the shotcrete treated assemblages and the control specimen. The E519 test approximated the inplane loading of a masonry wall and was used to evaluate the increase in load capacity of panels with a shotcrete treatment, interface bonding between brick and shotcrete, and composite action of the two-material assemblages. The E519 test suggests the use of 4 ft. by ft. assemblages; however, 3 ft. by 3 ft. panels were used for ease of handling.

2.2 Specimen Construction

Nine shotcrete reinforced brick masonry assemblages and three plain brick masonry assemblages were constructed to investigate experimentally the interface bonding of shotcrete to unreinforced brick masonry walls. To satisfy the objectives of this research, three shotcrete-brick surface conditions were investigated. The three bond conditions were: 1) a natural bond between dry brick masonry and the shotcrete, 2) a natural bond between water saturated brick masonry and shotcrete, and 3) an enhanced bond using an epoxy coating on the surface of the brick masonry at the interface between the brick and shotcrete. Three specimens for each of the conditions were constructed along with three control specimens, masonry panels with no shotcrete.

Appendix B contains information on the construction of 46 brick masonry assemblages which were used for the complete project funded by the National Science Foundation. Of the 46 specimens presented in Appendix B, 9 shotcrete reinforced and 3 plain brick assemblages are presented in this report. The three dry interface specimens are referred to as specimens D-1, D-2, and D-3. The three wet interface specimens and epoxy interface specimens are referred to as specimens W-1, W-2, W-3, and as E-1, E-2, and E-3, respectively. The control specimens are referred to as specimens C-1, C-2, and C-3. Within Appendix A specimens W-1, W-2, and W-3 are specimens 4, 5, and 6; specimens E-1, E-2, and E-3 are specimens 7, 8, and 9; specimens D-1, D-2, and D-3 are specimens 10, 11, and 12; and specimens C-1, C-2, and D-3 are specimens 1, 2, and 3, respectively.

Apprentice masons built the brick specimens. Their time was donated by the Bricklayers and Allied Craftsman Local 8 Labor Union. Four to six apprentices worked at building the assemblages each day. The masons were supplied with mortar and bricks by student employees. The mortar used was a weak mortar incorporating Magnolia Mortar mix, sand, and water. Magnolia Mortar mix is lime and portland cement mixture that was first used in Atlanta in the 1920's. The mortar was mixed in a concrete mixer and wheeled to mud boards where it was used (Fig. 2.2.1). The mix was

Figure 2.2.1 Mixing Mortar.

a basic 1:3 ratio, 1 unit of mortar mix to 3 units of sand on a volume basis. Enough water was added to create an acceptable workable mix as judged by the apprentice masons.

The specimens were built on wood platforms that were level and which were supported by bricks at two points. This support system allowed a fork lift to pick the specimens up at a future date. Figure 2.2.2 shows the wood platforms and the support system on which the specimens were built. The specimens were built next to each other with plywood spacers vertically between them. The spacer was later used as a form for the shotcreting process. The specimens were placed next to each other so that the shotcrete applications could be spread evenly and easily down a row of specimens.

Upon leveling a support, the masons constructed the 3 ft. wide by 3 ft. high, single wythe specimens using a full bed and head joint. The joints were not raked or struck because the masonry assemblages were modeling the inner wythes of unreinforced brick masonry walls. The inner wythe's joints typically were not raked or struck. The bricks were not soaked prior to use because typically the inner wythe bricks of masonry building of the late ninteenth and early twentieth centuries were not soaked.

The author noted that the masons would add extra water to the mortar to retain a workable mix. At times, a mason would mix small scraps back into the mortar on his mud

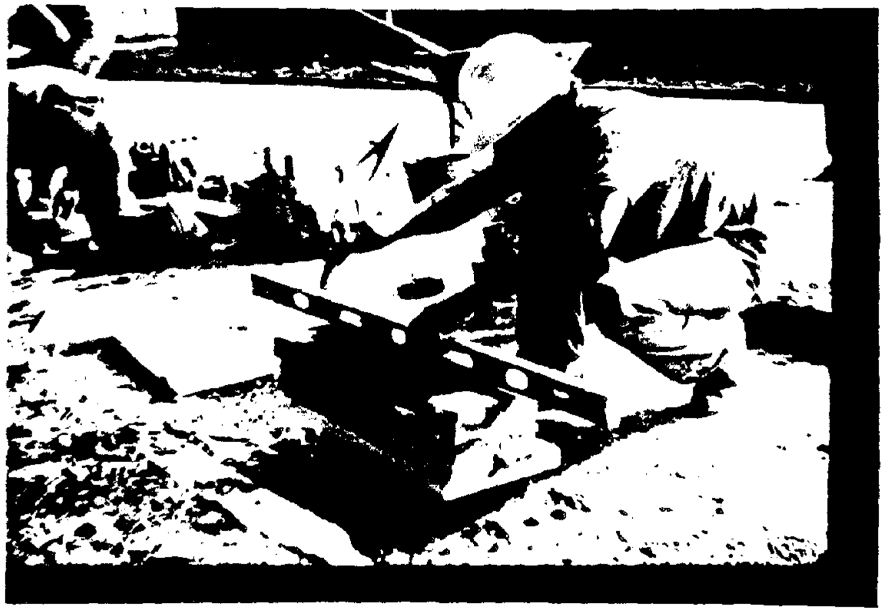


Figure 2.2.2 Leveling Up a Specimen.

Figure 2.2.3 Brick Panels on Cherry Street.

board; however, this practice was limited.

Masonry prisms made of two bricks with a full bed joint between them were collected as often as time permitted. The prisms were allowed to set uncovered overnight; on the next day they were placed in the moisture room for testing at a future date. Mortar cubes were cast and were kept covered in the steel cube mold for one day, then placed in a moisture room for testing at a later date.

Figure 2.2.3 shows construction of all 46 specimens as reported in Appendix B. The exact specimen sizes after construction are illustrated in Figs. B.2 to B.13 of Appendix B.

Sections of 6 x 6 - W3.5 x W3.5 welded wire fabric were cut to the size of each 3 ft. x 3 ft. specimen. The welded wire fabric was used to reinforce the shotcrete and to help hold the shotcrete from sluffing during placement.

Formwork was placed around the specimens to create plane edges, a uniform 3-inch thickness, and a barrier between specimens. The formwork consisted of oiled plywood framed into place such that the leading edges protruded 3 inches out from the plane of the brick.

Shotcreting started on September 30, 1981 and finished on October 1, 1981. Western Waterproofing Company, Inc. of Norcross, Georgia was contracted to preformed the dry-mix shotcreting using a standard 4000 psi shotcrete mix. The pneumatic gun used was an Allentown Pneumatic Gun, Model

N-O with steel wheels and a capacity of three-quarters to one and one-half cubic yards of shotcrete per hour. Figure 1.3.14 shows a vertical section of a gun like that used by Western Waterproofing.

The shotcreting crew consisted of three men, one nozzleman and two others who attended to the hoses, pumper, and air compressor. The attendants, on a plywood board, would mix 27 shovels of damp sand with one bag of Type I Portland cement (Fig. 2.2.4). After mixing, they would load parts of the mix into the hopper of the pumper (Fig. 2.2.5). The water content of the sand was about 5.9 percent which was acceptable. The mix ratio of cement-to-mortar by volume was 1:3 which was considered rich according to ACI Committee 506 (3, 4).

The nozzleman, by adjusting the water added at the nozzle, controlled the resulting mix. He held the nozzle at about 90 degrees to the plane of treatment, and 8 to 10 feet away from the specimen. Figure 2.2.6 shows the nozzleman shooting high on the plane of treatment while Fig. 2.2.7 shows the nozzleman shooting a low position. As can be seen in Figs. 2.2.6 and 2.2.7, the nozzleman made every effort to assure a high quality product.

Prior to the treatment of the specimens, the masonry surfaces were prepared according to the three interface conditions. The surfaces of specimens D-1, D-2, and D-3, dry interfaces, were left alone. The surfaces of

Figure 2.2.4 Mixing the Dry Portions of the Shotcrete.

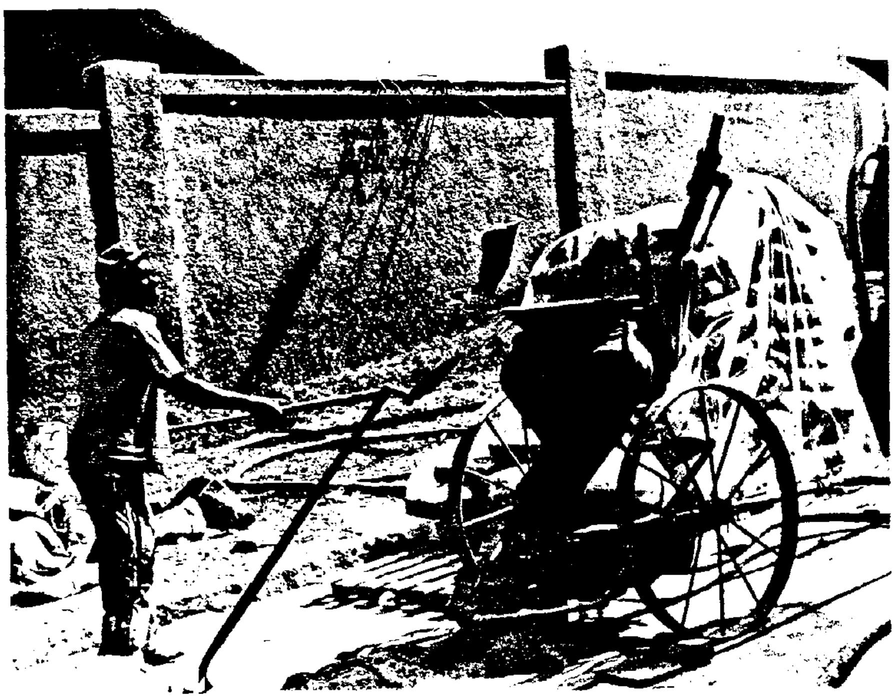


Figure 2.2.5 Filling the Hopper of the Gun with the Dry Portions of the Shotcrete.

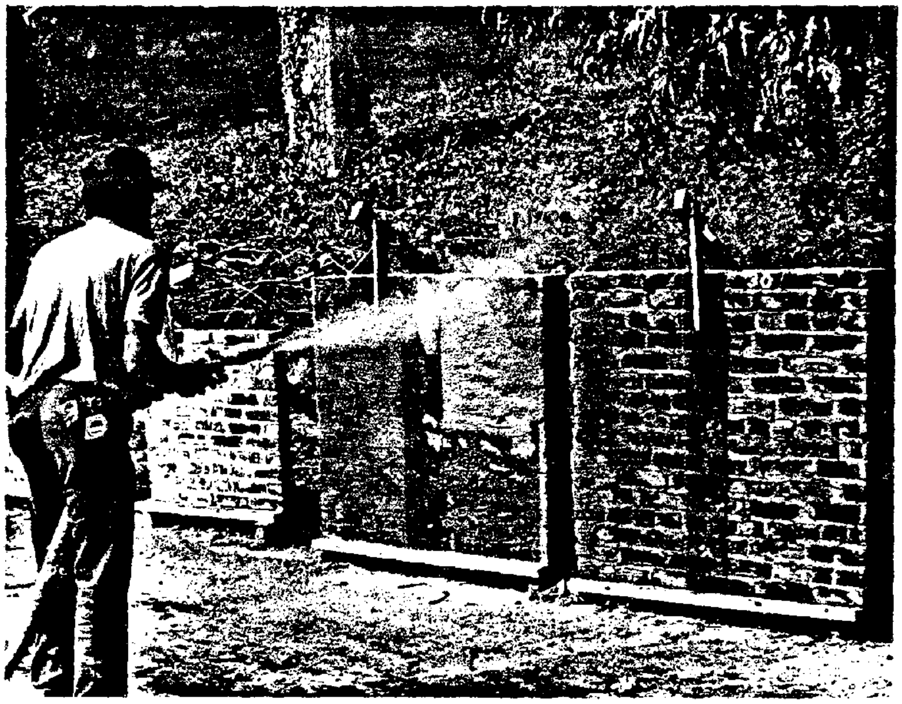


Figure 2.2.6 Nozzleman Shooting High.

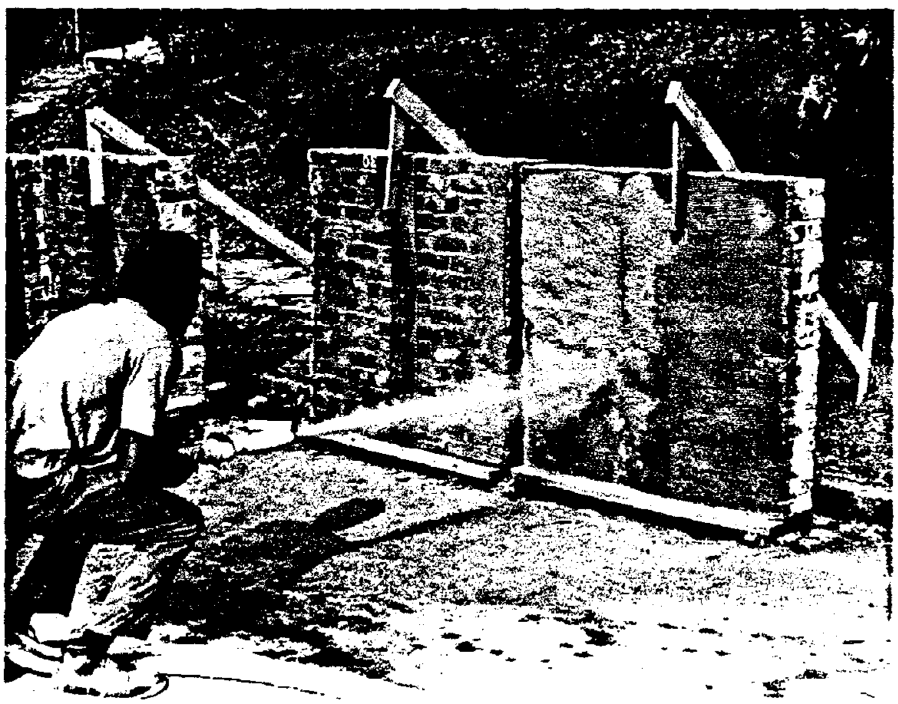


Figure 2.2.7 Nozzleman Shotting Low.

specimens W-1, W-2, and W-3, wet interfaces, were soaked continuously with water for several hours prior to shotcreting to saturate the bricks. Figure 2.2.8 illustrates the use of a water hose to keep the surfaces wet. Within 15 minutes prior to shooting specimens E-1, E-2, and E-3, the epoxy was mixed and then brushed onto the brick surfaces as illustrated in Figs. 2.2.9 and 2.2.10, respectively. The surfaces of specimens E-1, E-2, and E-3 were coated with epoxy just prior to the shotcrete treatment because of the short pot life of the Sikadur Hi-Mod Gel epoxy. Directly after the surfaces of the specimens were coated with epoxy, the nozzleman shot them with the shotcrete treatment as shown in Fig. 2.2.11.

Before the placement of the first lift, the welded wire fabric was positioned into the forms such that the wires were between one and one-half inches from the brick. All 3-inch shotcretements were applied in two lifts. The nozzleman would short about one-half the thickness (Fig. 2.2.12) and let it cure for one full day. The wire fabric always was covered with the first lift. Prior to the application of the second lift, the surfaces were completely wetted to help provide a stronger bond between the two lifts. The author noted that the finished shotcrete was of high quality with limited segregation, excellent coverage, and a good surface finish (Fig. 2.2.13).

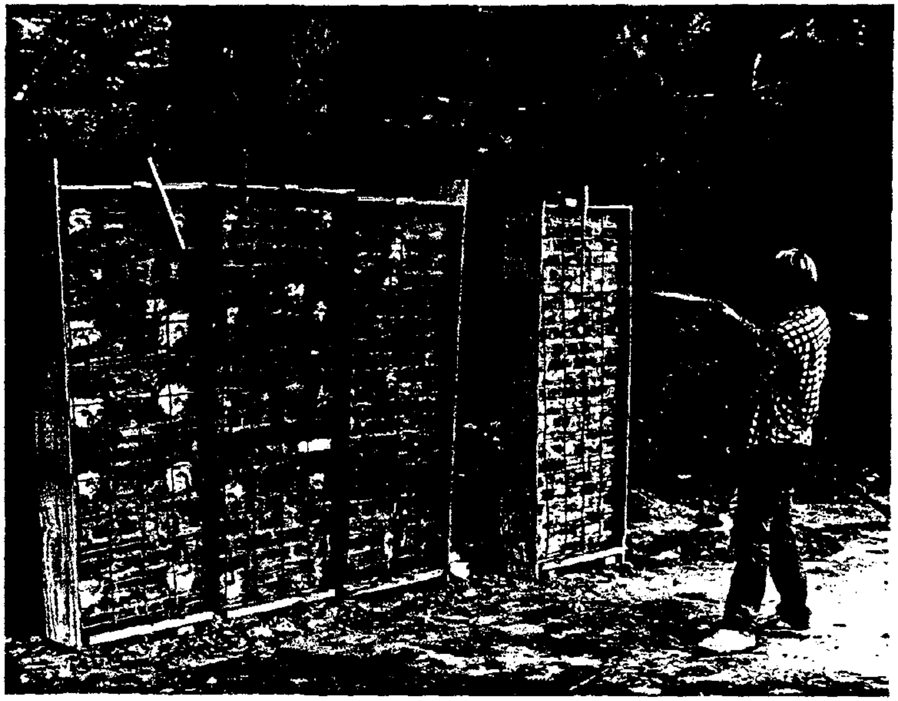


Figure 2.2.8 Saturating Surfaces with Water.

Figure 2.2.9 Mixing Epoxy for Surface Application.

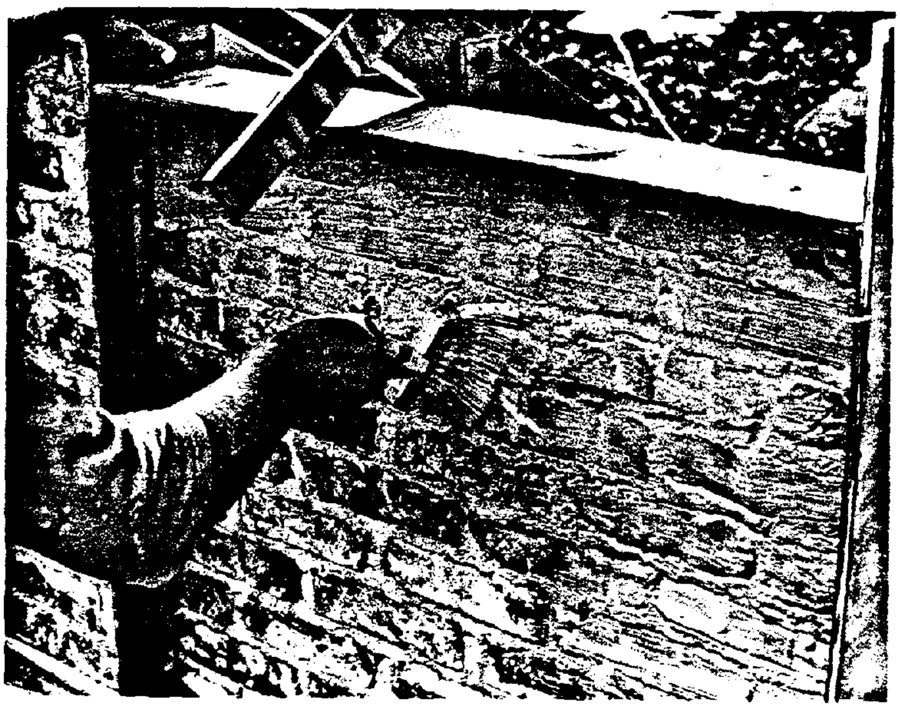


Figure 2.2.10 Brushing Epoxy onto the Surface of the Brick.

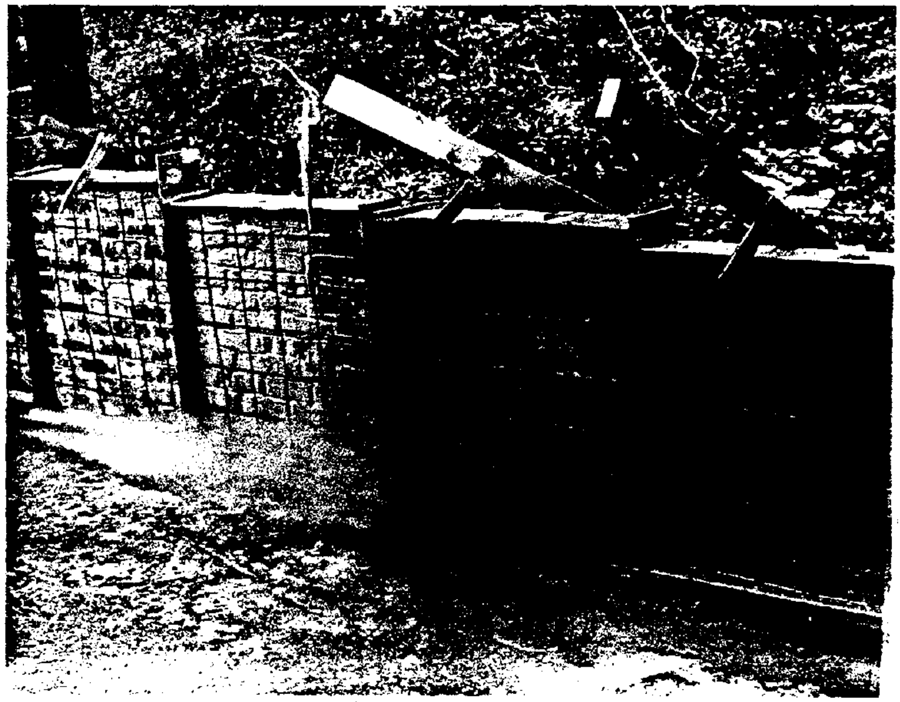


Figure 2.2.11 Applying Shotcrete to the Epoxy-Coated Specimens.

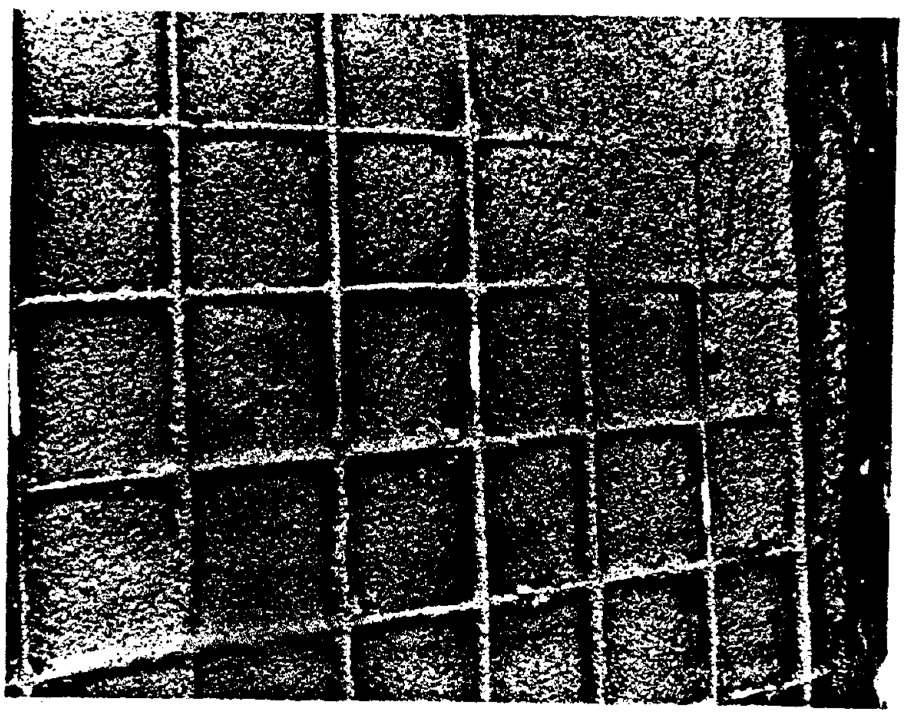


Figure 2.2.12 View of the First Life of Shotcrete.

Figure 2.2.13 View of the Surface of the Shotcrete.

After the specimens were completed, they were covered with an asphalt impregnated paper moisture barrier and were allowed to cure for two weeks. The specimens were not wetted during the initial cure and were exposed to all weather.

After two weeks, the specimens were moved with a fork-lift to the structures lab just a few hundred yards from where the specimens were built. No damage occurred to the specimens during the transportation to the structures lab.

2.3 Comments on Construction

The quality of the masonry was satisfactory. The joints were not struck or raked and the bricks were not soaked as was required to model the inner wythes of brick masonry buildings of the late nineteenth and early twentieth centuries. The weak Magnolia Mortar mix provided a workable mortar mixture; however, because the dry solid soft bricks soaked water from the mortar, weak brick-mortar bonds were created. Weak brick-mortar bonds were acceptable because weak brick mortar bonding was common in inner wythes of older masonry buildings. It was evident that the brick-mortar bonds were weak; when sample C-1 was picked by the C clamp for placement into the testing machine, the specimen broke under its own weight.

Solid soft bricks were to be used in the construction of the samples; however, about 5% of the bricks in the masonry assemblages were solid hard burned brick. The low percentage of hardburned bricks in the specimens assured that the specimens modeled the weaker interior wythes which were made with the underburned bricks.

The placement of the reinforcing was more difficult than expected. It was hard to keep the welded wire fabric in place because of the high velocity and large impact force of the shotcrete. The fabric was to be placed around 3/4" from the outside surface of the shotcrete but, the welded wire fabric ended up between 1½ to 2" from the outside surface.

The shotcrete treatment turned out better than what was expected. The nozzleman was able to produce a consistent mix. The shotcrete thickness ranged from 3.1" to 3.9". The two lift process was accepted because the nozzleman said that a multiple lift process was typically used to produce thick applications of shotcrete. The gunned surface provided by the nozzleman was extremely smooth, providing a good surface to attach instrumentation.

Table 2.3.1 contains the resulting thickness of brick, shotcretes and the percentages of steel to area of shotcrete cross section.

2.4 Material Properties

Old solid, soft brick from the Atlanta Civic Center were used as the brick in the masonry assemblages described in this report. The Atlanta Civic Center was built between 1928 to 1932 and was demolished by the Atlanta Wrecking and Salvage Company in 1981. The bricks were soft to the

Table 2.3.1 Thicknesses and Ratio of Steel Area to Wall Cross-Sectional Area for the Specimens.

Specimen Number	Thickness Brick (in)	Thickness Shotcrete (in)	Thickness Average (in)	$\frac{A_{steel}}{A_{shotcrete}}$ % one direction
W-1	3 3/4	3.4	7.16	0.17
W-2	3 3/4	3.9	7.67	0.15
W-3	3 3/4	3.5	7.29	0.17
E-1	3 3/4	3.1	6.85	0.19
E-2	3 3/4	3.8	7.53	0.15
E-3	3 3/4	3.8	7.53	0.15
D-1	3 3/4	3.1	6.89	0.19
D-2	3 3/4	3.8	7.54	0.15
D-3	3 3/4	3.4	7.12	0.17

touch and easily scraped with the fingernail. Table 2.4.1 shows the tests results of the initial rate of absorption for five solid soft bricks. The initial rate of absorption ranged from a low of 0.201 oz/in^2 to a high of 0.835 oz/in^2. The mean and standard deviation was 0.50 oz/in^2 and 0.29 oz/in^2, respectively.

Brick-mortar prisms were built during construction of the masonry assemblages. The prisms were of two bricks with a full bed of mortar between the bricks. The samples were allowed to set uncovered overnight. On the next day the piers were placed in the moisture room for testing 28 days later. Table 2.4.2 contains the test results for the 14 compression tests run on the brick-mortar prisms. The ultimate compressive strength f'_m ranged from a low of 1377 psi to a high of 2011 psi. The mean ultimate compressive strength was 1630 psi with a standard deviation of 250 psi.

The mortar used in the construction of the brick masonry assemblages contained Magnolia Mason's mix brand masonry cement, masonry sand and water. The mix was a basic 1:3 ratio, 1 unit of mortar mix to 3 units of sand on a volume basis. Enough water was added to create an acceptable workable mix as judged by the apprentice masons. The resulting mix was similiar to the typical type 0 mortar mix used today. The Magnolia Mason's mix brand was produced by Martin Marietta Cement, Southern Division, Birmingham, Alabama. The mortar mix complies with Federal Specifications SS-C-1960/1FNDASTM C91.

Table 2.4.1 Initial Rate of Absorption for the Brick.

Brick Number	Weight Prior (grams)	Weight After (grams)	Area (in^2)	X cm^2/in^2	X oz/in^2
1	2230	2255	30.7	19.4	0.328
2	2186	2236	32.4	46.3	0.783
3	2179	2231	31.6	49.4	0.835
4	2300	2313	32.8	11.9	0.201
5	2329	2351	32.0	20.6	0.348

Mean 0.50 oz/in^2

SDev 0.29 oz/in^2

Range: low 0.201 oz/in^2

high 0.835 oz/in^2

X=initial rate of absorption, expressed as a gain in weight, corrected on the basis of 30 in.^{2} flatwise area

W=weight, prior to immersion
W_1=weight, after immersion
A=net cross-sectional area of immersed surface, in^2.

$$X = \frac{(W_1 - W)30}{A} \qquad \text{ASTM C67 (7)}$$

Table 2.4.2 Compression Test,* Brick-Mortar Prisms.

Date	Specimen Number	Ultimate Load (lbs)	Compressive Strength (f'_m) (psi)
9-15-81	1	54650	1820
9-16-81	2	60350	2011
	3	44100	1470
	4	53000	1767
	5	41300	1377
9-17-81	6	41800	1393
	7	broken during capping	
9-23-81	8	48300	1610
	9	42100	1403
	10	44900	1497
	11	60000	2000
	12	broken during capping	
	13	broken during transportation	
	14	broken during transportation	

* test run at 28 days

Mean	1630 psi	Range	low	1377 psi
SDev	250 psi		high	2011 psi

The Magnolia Mason's mix brand is a waterproofed-hydraulic mortar mix. During the construction, 2 inch mortar cubes were collected as often as time permitted. The cubes were kept covered for one day while in the steel mold, then placed in the moisture room for testing 28 days later. A total of 22 mortar cubes were made and tested. Table 2.4.3 gives the results of the 22 compression tests for the mortar cubes. The average (mean) ultimate compressive strength of the 2 inch cubes was 460 psi, with a standard deviation of 115 psi, and a range from 310 psi to 700 psi.

The dry mix process was used for the placement of the shotcrete skins. Damp river sand and Type I portland cement were premixed at a ratio of 27 shovels of sand to one 90 lb. bag of cement. The mixture resulted in a 1:3 volume mix which according to ACI recommendations (3, 4), was considered rich. ACI Committee 506 (4) recommended a mix of 1:3.5-4.5. The water cement ratio was governed by the nozzleman, relying on past experinece to produce a 4000 psi plus shotcrete. The sand used in the shotcreting process was a riverbed sand from the Chattahoochee River. The water content of the damp sand was found to be about 5.9% which was acceptable according to ACI recommendations (3, 4). A sieve analysis was done on a sample of river sand. Figure 2.4.1 shows the ASTM standard envelope for sieve analysis and the plot of the sieve analysis for the river sand used in shotcreting. The river sand did not totally fall within the recommended

Table 2.4.3 Compressions Test,* Mortar Cubes.

	Cube Number	Ultimate Load (lbs)	Compressive Strength (f'_c) (psi)
9-14-81	1	680	-
	2	1460	365
9-15-81	3	1660	413
	4	1240	310
9-17-81	5	1900	475
	6	1580	396
	7	2040	510
	8	2800	700
	9	1580	395
	10	2260	565
9-22-81	11	1320	330
	12	1620	405
	13	1380	345
	14	1380	345
	15	1300	325
	16	1360	340
9-23-81	17	2300	575
	18	1820	455
	19	2140	535
	20	2160	540
	21	2300	575
	22	2420	605

* tests done at 28 days after the specimen was taken

Mean	460 psi
SDev	115 psi
Range	
low	310 psi
high	700 psi

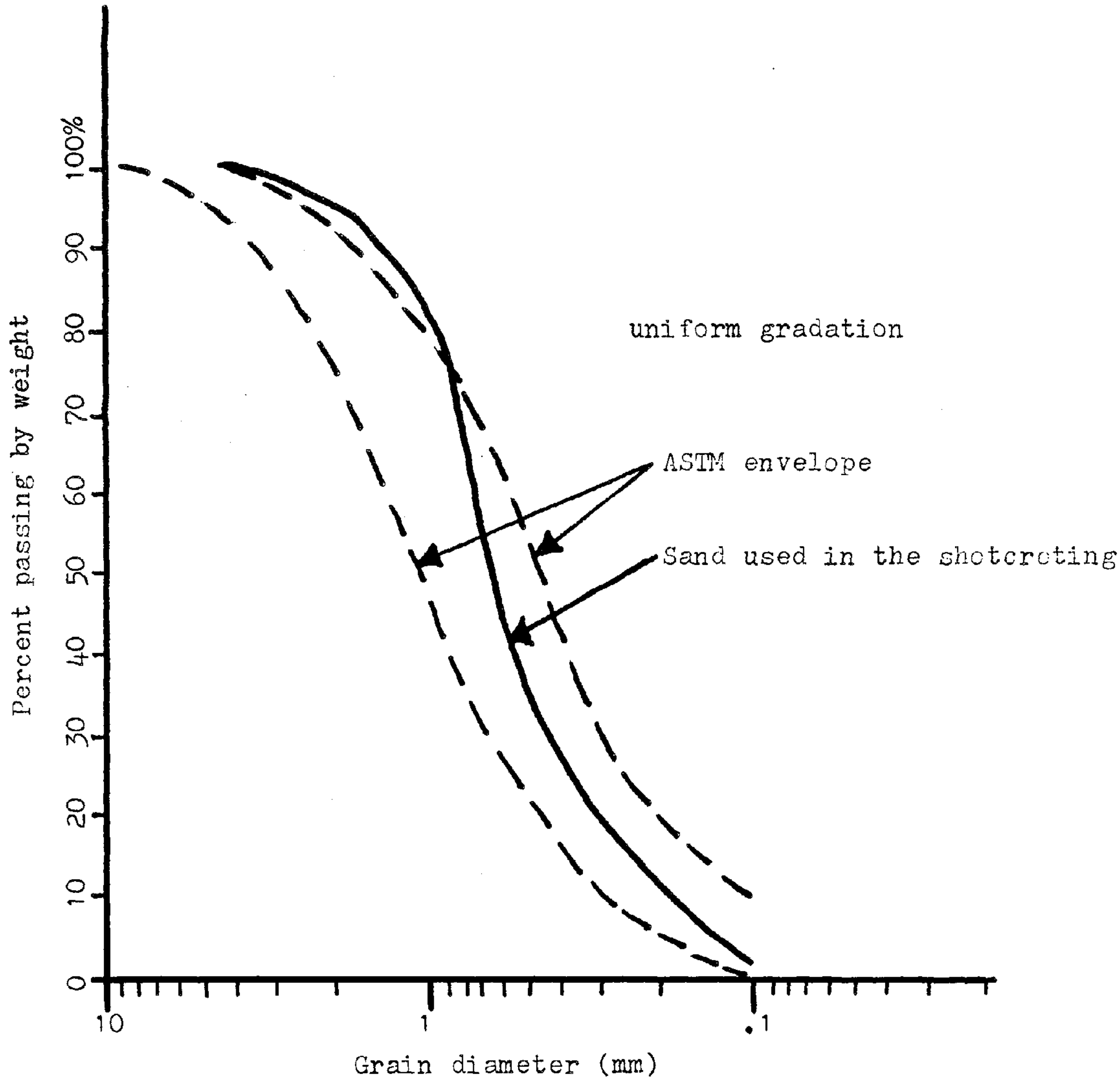


Figure 2.4.1 Sieve Analysis for Shotcrete Fine Aggregate.

band; it had a smaller percentage of grain sizes around the 1 mm size than was recommended. The cement used in the shotcrete was a Type I Coosa Portland cement from the from the National Cement Company of Ragland, Alabama. The cement meets the ASTM C-91 and the Federal Inspection SS-C181 specifications for Portland cements. The properties of the finished shotcrete were determined by drilling 1.4" by 2.8" core samples from the shotcrete reinforced specimens after the specimens were tested. Cores were taken from low damaged areas perpendicular to the plane for the shotcrete panel. A number of core samples were taken from specimens E-2, D-1, D-2, and D-3. Cores were not taken from all the specimens due to time restrictions. The cores were capped with a sulfur capping compound and were tested in compression. The compression tests were done in a 20k INSTRON testing machine; load-deflection curves were plotted during each test. Figure 2.4.2 illustrates a typical stress strain plot for the shotcrete cylinders. The Modulus of Elasticity for each specimen was calculated according to the method as shown in Fig. 2.2.15. Table 2.4.4 gives the resulting Modulus of Elasticity for the shotcrete cores. The mean Modulus of Elasticity was 5.62E6 psi and a range from 4.99E6 psi to 6.18E6 psi. Table 2.4.4 also gives the results of the compression tests on the shotcrete cylinders. To correlate the compressive strengths of the 1.4" by 2.8" cylinders with standard 6" by 12" cylinders, the apparent

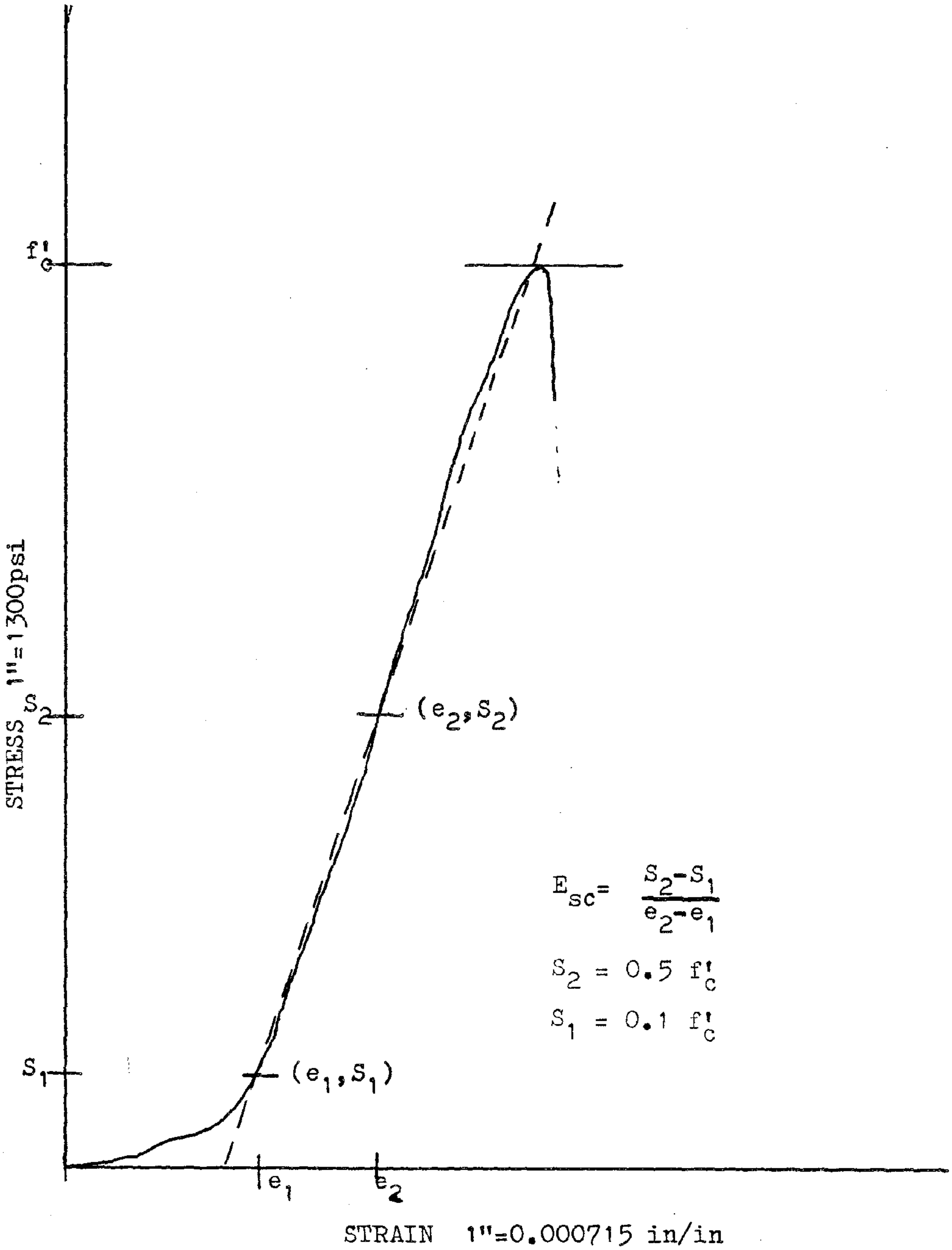


Figure 2.4.2 Typical Stress-Strain Plot for the Shotcrete Cores.

Table 2.4.4 Compressive Strength Tests of 1.4 inch by 2.8 inch Shotcrete Cores.

Specimen Number	f'_c(psi)	f'^{*}_c(psi)	E_{sc}(psi)
E-2	7400	6430	5.18E6
	8500	7390	5.23E6
	6200	5390	4.99E6
	8200	7130	5.09E6
D-1	7800	6780	5.82E6
	8400	7300	5.91E6
	8800	7650	6.18E6
	8200	7130	5.73E6
D-2	8100	7040	5.64E6
	8400	7300	5.98E6
	8580	7460	5.65E6
	6880	5980	5.14E6
	7540	6560	5.27E6
D-3	8120	7060	5.83E6
	7990	6950	5.59E6
	8060	7010	5.78E6
	7540	6560	6.03E6
	8250	7170	6.16E6
Mean	7940	6900	5.62E6
SDev	640	560	0.38E6
Range			
low	6200	5390	4.99E6
high	8800	7650	6.18E6

* Strength of an equivalent 6" by 12" cylinder equals the strength of the 1.4" by 2.8" core divided by 1.15 .

$$f'^{*}_c = f''_c / 1.15$$

strength was divided by a factor of 1.15 (9). The mean value for the corrected compressive strength of the shotcrete (f'_c) was 6900 psi with a standard deviation of 560 psi and a range from 5390 psi to 7650 psi.

The epoxy used for the epoxy enhanced specimens, E-1, E-2, and E-3, was Sikadur Hi-Mod Sikastix 370 dual purpose, 100%-solids, 2-component, moisture-insensitive epoxy adhesive. Sikastix 370 normally is used as a bonding agent between fresh concrete and hardened concrete.

The steel reinforcement for the shotcrete used in the construction of the specimen was 6 x 6 - W3.5 x W3.5 welded wire fabric. The fabric was a 6" square grid using wires of 0.035 in^2 area running in both directions. Sections of wire were cut from the welded wire fabric for material testing. The first section included a wire which ran perpendicular to the length of the roll of the fabric. This first type of section was used to see if the weld affected the strength of the wire. The second type of section was just a short length of wire, also perpendicular to the length. The samples were tested in the INSTRON testing machine where load-deflection plots were produced during the testing. Figure 2.4.3 shows the typical stress strain plot for the wire. The 0.2% offset yield stress and maximum stresses were taken from the plots and were recorded in Table 2.4.5. The 0.2% offset yield stress had a mean, standard deviation and range of 40.5 psi, 1.0 psi, and 39.0 psi to 41.5 psi, respectively, for the 7" lengths with welds. The 0.2%

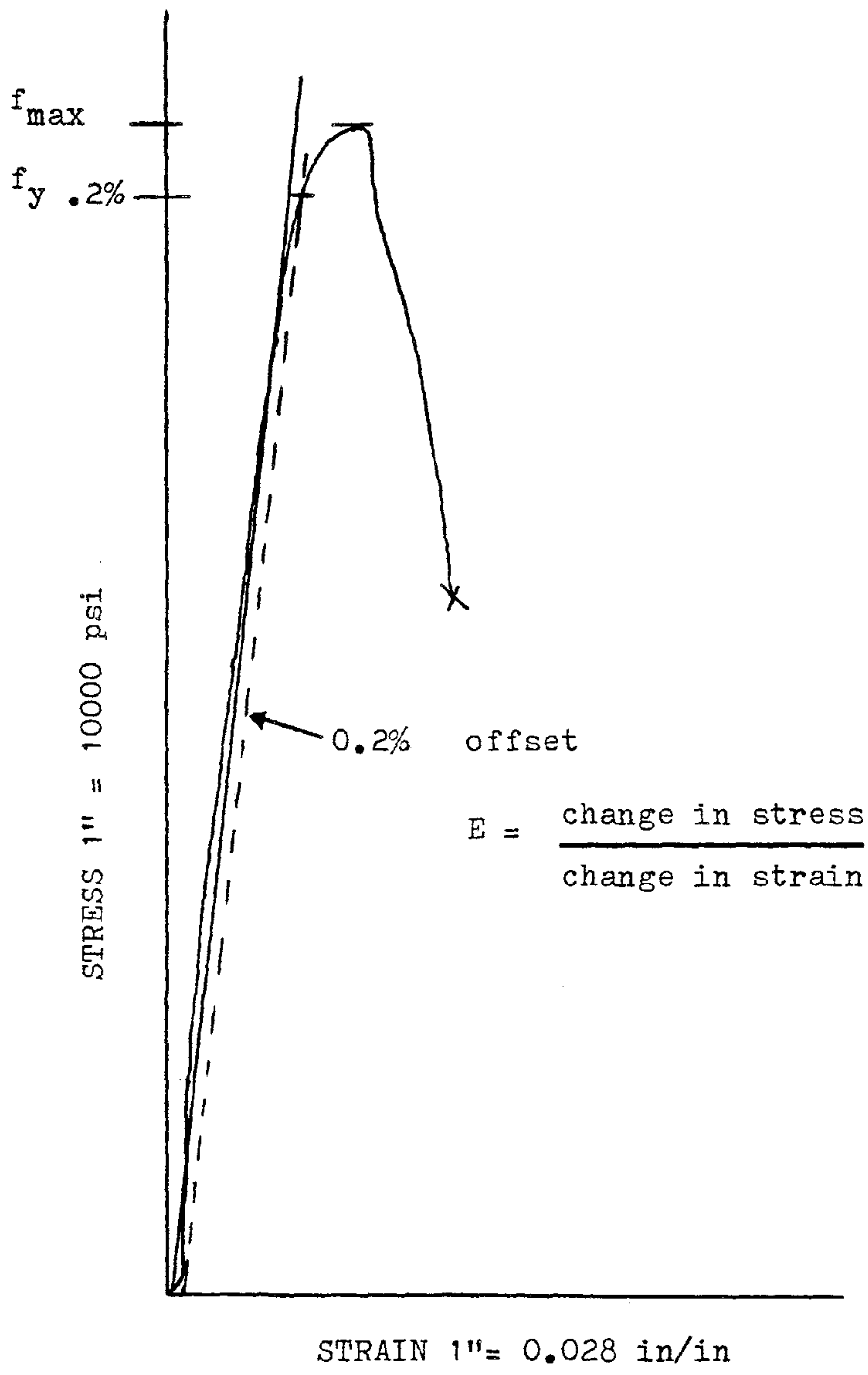


Figure 2.4.3 Typical Stress-Strain Plot for the Welded Wire Fabric Sections.

Table. 2.4.5 Tension Tests on Sections of 6 x 6 - W3.5 x W3.5 Welded Wire Fabric.

	Test	f_m(ksi)	f_y .2%(ksi)	E (psi)
7" lengths with welds	1	41.2	39.0	2.09E6
	2	40.9	39.5	2.24E6
	3	41.7	41.1	2.20E6
	4	41.8	41.1	1.87E6
	5	41.7	41.1	1.83E6
	6	42.8	41.5	2.12E6
	7	41.2	40.1	1.76E6
	Mean	41.6	40.5	2.01E6
	SDev	00.6	1.0	0.19E6
	Range			
	low	40.9	39.0	1.76E6
	high	42.8	41.5	2.24E6
3" lenghts	1	42.5	41.8	1.13E6
	2	42.8	42.2	1.30E6
	3	42.2	41.5	0.97E6
	4	42.9	42.3	1.01E6
	Mean	42.6	42.0	1.10E6
	SDev	0.3	0.4	0.15E6
	Range			
	low	42.2	41.5	0.97E6
	high	42.9	42.3	1.30E6

Areas = 0.035 in.^{2}

offset yield stress for the 3 inch samples had a mean of 41.9 psi, a standard deviation of 0.37 psi and a range from 41.7 psi to 42.3 psi. The mean Modulus of Elasticity for the 7 inch length with weld was 2.0E6 psi while the Modulus of Elasticity of the 3 inch samples had a mean of 1.10E6 psi.

Fibreen 200 SK-20 with asphaltic adhesive was used as the moisture barrier on the freshly shotcreted assemblages. Fibreen 200 is a Sisulkraft product of St. Regis' Laminated and Coated Products Division. Fibreen 200 conforms to the following specifications: VV-B-790, Type I, Grades A, B, and C Style 4, VV-P-271d, Class E-1.

2.5 Test Set-Up and Instrumentation

The ASTM E519-74 (11) test set-up was used in this research project. The E519 testing procedures required the compressive loading of the test panels across the specimens' diagonal as illustrated in Fig. 1.3.15. The specimens were picked up with a large "C" clamp and moved to the testing machine. They were rotated by 45 degrees and placed into grouted loading shoes. The loading shoes were constructed according to the recommendations in the E519 testing procedures. The "C" clamp was designed to clamp onto the specimens via large padded bearing plates actuated by screw drives. At the ends of the screws, next to the bearing plates, were tapered roller bearings. The bearings were designed so that when the clamp was in place, tightened down, and the specimen was picked off the ground, the bearing system allowed the specimen to be rotated easily. After the specimen was rotated, it was easy to place the specimen into the grouted loading shoes in the test frame. The "C" clamp worked well for the shotcreted specimens; however, the control specimens needed to be banded prior to the use of the clamp because the brick-mortar b nd was so weak that the specimen would break under its own weight. Specimen C-1 was broken when it was picked up, and it could not be replaced.

A mortar mix of Type III (high early strength) cement, sand and water was used to grout the specimen ends into the loading shoes. The grout was used to reduce localized bearing stresses in the shoe and to create a good foundation for testing. The mortar was used in both the top and bottom loading shoes.

After the specimen was placed into the grouted bottom shoe, supported in the guides, and released from the "C" clamp, it was pulled via block and tackle the middle of the test machine. The specimen was then leveled up in all directions to make sure that the loading shoes were vertically aligned. First, a plumb-bob was used to level the specimen in its own plane. Next, a torpedo level was used to assure that the plane of the specimen was perpendicular to the base and loading head of the test machine. After the initial alignment was completed, the specimen was wedged in place and rechecked for plumbness.

Next, the top crown of the specimen was coated with mortar and a plastic bag was placed over the mortar. The top shoe was then lowered down onto the mortared crown and carefully leveled. Plastic bags are used between the mortar and shoes so that the shoes would be easier to remove after testing. The specimen was then left overnight in the testing machine to assure that the mortar grout used in the shoes had developed enough load capacity for testing.

On the next day, instrumentation was placed on the specimen. Instrumentation on the horizontal and vertical diagonals as in the ASTM E519-74 test (11) were used on each side of each panel as shown in Fig. 2.5.1. Additional instrumentation was used to measure lateral deflections perpendicular to the plane of the specimen. The lateral instrumentation was used to measure delamination between the shotcrete and brick and the out-of-plane flexural. Dial gauges were used in the instrumentation because reliable electronic devices were not available and because the dial gauges were easy to use. The instrumentation was designed so that it could be used a number of times.

Figure 2.5.2 shows the type of instrument that was used for the measurement of the vertical and horizontal deflections and strains. The instrument uses matched pairs of metal angles with holes in one leg, an aluminum rod with nuts at one end, and a dial gauge with an attachment bracket that hooks onto the aluminum rod. The angles served as

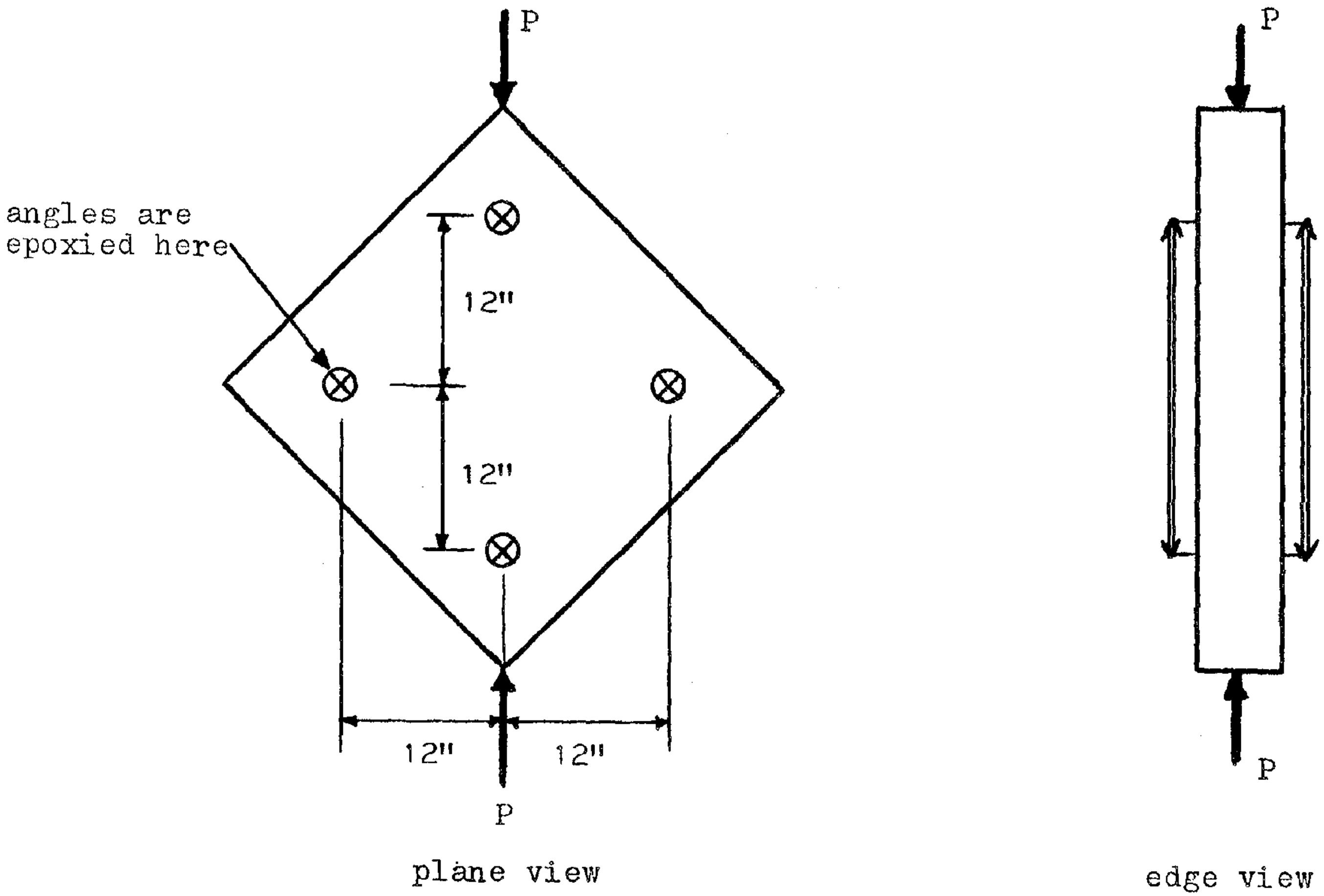


Figure 2.5.1 Test Set-Up, Vertical and Horizontal Instrumentation Positions.

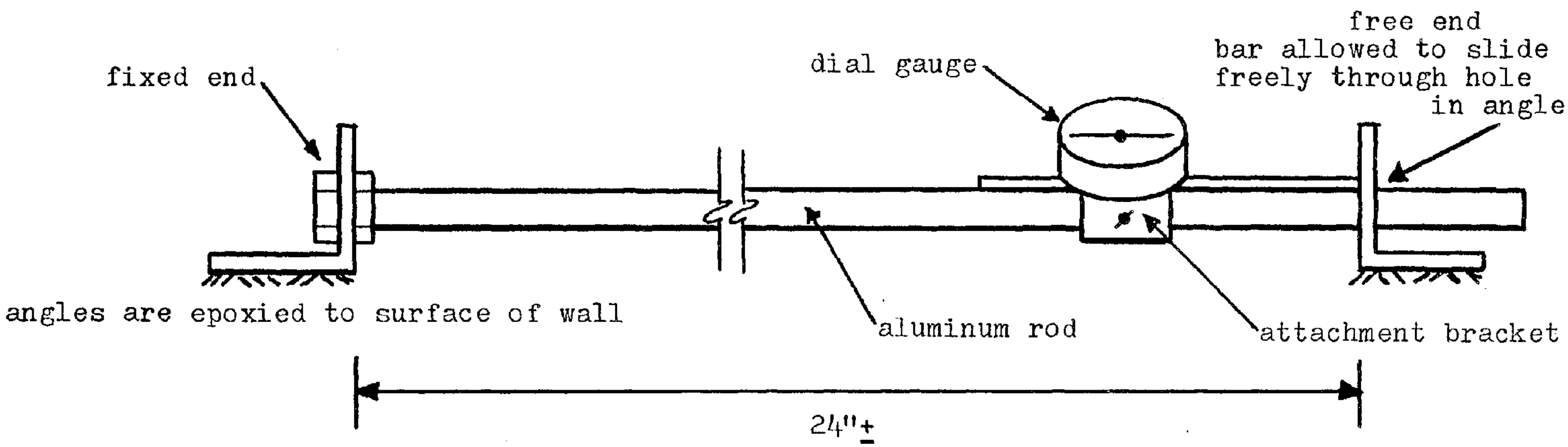


Figure 2.5.2 Vertical and Horizontal Instrumentation.

guides for the aluminum rod. The rod was fixed at one angle by a double nut, and was free to move at other angles. The dial gauge was fixed via an attachment bracket to the aluminum rod such that the tip rested on the angle that allowed movement.

To attach the instrument to the surface of the specimen, Magna bond 58 adhesive was used to epoxy the angles to the surface. The angles were attached to the surfaces on the diagonals of the specimen, 12 inches from the middle as illustrated in Fig. 2.5.1. After the angles were attached, the rod and dial gauge were positioned as shown in Fig. 2.5.2. Two instruments were attached to each side of specimen, one vertical and one horizontal. The horizontal instrument used angles with shorter legs, allowing the vertical rod to pass freely above the horizontals rod. After the instruments were attached, the gauge lengths were measured, allowing the calculation of strains. The gauge lengths were 24 inches plus or minus 1 inch in length.

The lateral deflections were accessed by using a vertical support system that held dial gauges perpendicular to the plane of the specimen. Two support systems were built, one for each side of specimen, with vertical and horizontal movement capabilities which allowed the dial gauges to be positioned near the top, middle, and bottom of the specimen. The positions of the gauges is illustrated in Fig. 2.5.3. Each lateral gauge tip rested on a thin metal plate that

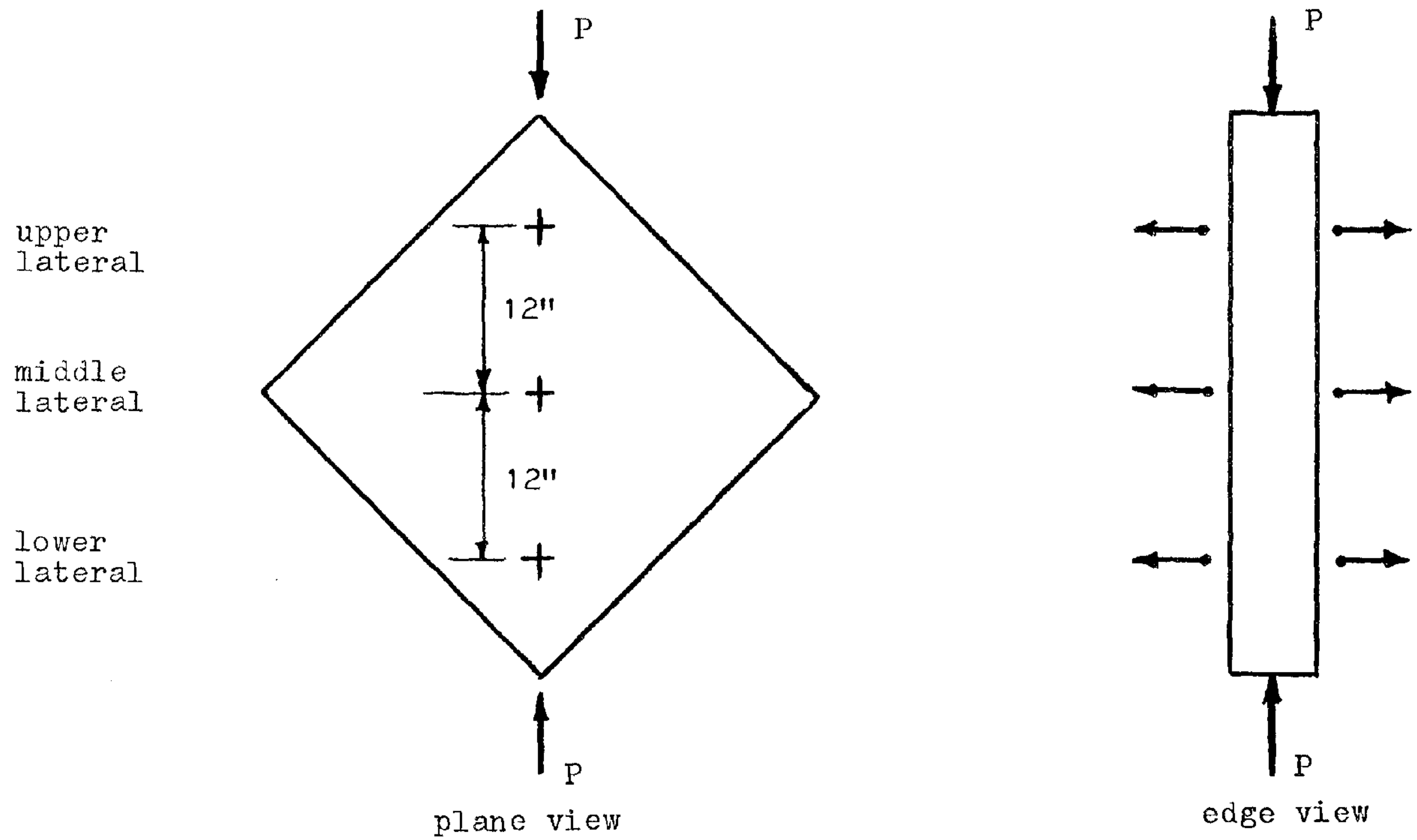


Figure 2.5.3 Test Set-Up, Lateral Deflection Positions.

was epoxied to the specimen. The metal plate provided a smooth, flat surface for the dial gauge tip to rest against.

Figure 2.5.4 shows a view of a specimen positioned in the testing machine.

A 450,000 lb. Riehle, screw-drive universal test machine was used to apply the load.

2.6 Test Procedures

A single full cycle static load test was used to investigate the ultimate load capacity, the adequacy of the interface bond between the brick masonry and shotcrete for the dry, wet and epoxy surface conditions, and the extent to which the brick masonry and shotcrete treatment act compositely. The single full cycle static test was performed in the following manner: 1) loading of the specimen on its initial vertical diagonal to the maximum and taking data at set increments of load or at times when the specimen exhibited load capacity loss or variations, or at random increments when large deflections occurred without significant changes in load capacity; 2) unloading the specimen after crushing was noted and taking data during unloading at set load increments; 3) removing the specimen from the testing machine and rotating it 90 degrees; 4) placing the specimen back into the loading machine with the initial horizontal diagonal as the new vertical diagonal; 5) reinstrumenting the rotated specimen; 6) loading and unloading, taking data as in the first half cycle. The first loading and unloading sequence was termed

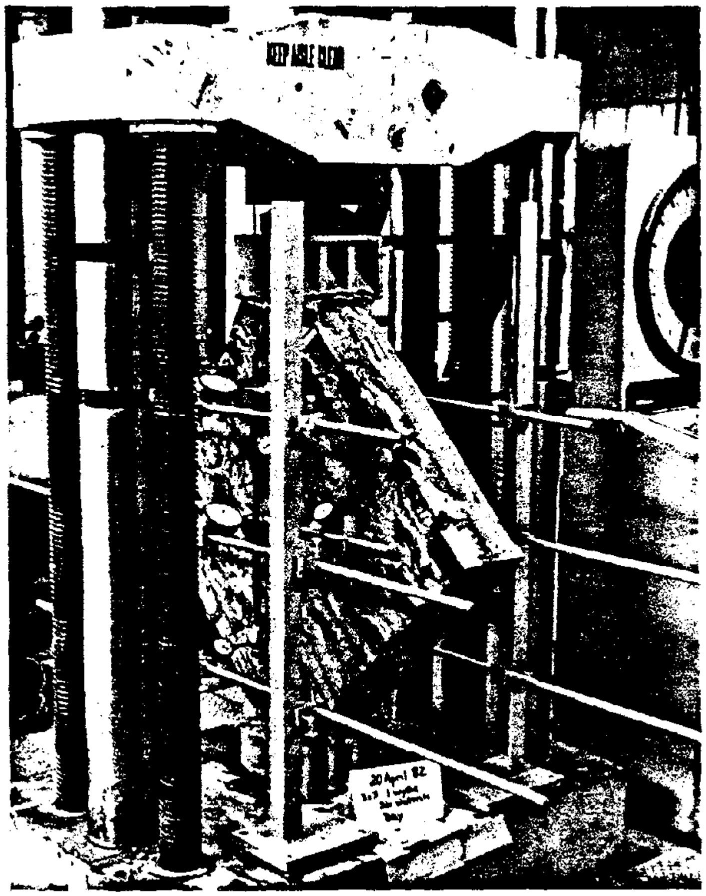


Figure 2.5.4 View of the Gauges on a Specimen in the Testing Machine.

the first or initial half-cycle while the second loading plus unloading sequence was termed the second-half cycle. Figures 2.6.1 and 2.6.2 show the first and second half test cycles. The control specimen C-3 only was tested under the first half-cycle because it was destroyed during that first half-cycle.

The data from the two half cycles were integrated into a single full cycle. The single full cycle data for each shotcreted specimen allowed the comparison of energy dissipation, ductility and ultimate load capacities between specimens.

As each panel was prepared for testing, various measurements were made. The measurements included the angle of the bed joint from the vertical, the length of the bed joint, the four gauge lengths (vertical and horizontal for both surfaces), the thickness of the specimen next to the anchor points of the strain gauges, and the distances between the parallel vertical and horizontal rods on opposite sides of the panel. The specimen thickness and distances between parallel rods were taken at the angle positions (Fig. 2.5.1) to allow the deflections and strains on the horizontals and verticals to be calculated at the surfaces of the specimens if linear strain variation were assumed.

The first half cycle was run after the measurements were recorded. Load increments of 10000 pounds were used until diagonal cracking was observed, and then about

Figure 2.6.1 View of a Brick Masonry Cracking on the First Half-Cycle.

Figure 2.6.2 View of Brick Masonry Cracking on the Second Half-Cycle.

5000 pound increments were used until the maximum was reached. Further increments were related to the behavior of the specimen under load. Data generally were collected at increments of load; however, when the specimen exhibited excessive deflection without a load capacity change, data were taken at increments of deflection. Upon excessive crushing of the specimens during either the initial or second cycle, the specimen was unloaded to prevent its destruction.

During the testing, loads at which visual cracking and audible sounds associated with cracking were noted. At times, a specimen would flex so much out of plane that flexural cracking would occur near the head requiring an unload cycle. Specimen E-3 exhibited this behavior on the second half-cycle.

After the second half-cycle was completed for a specimen, thicknesses of the shotcrete at various points, positions of the welded wire fabric within the shotcrete, sketches of cracking, and interface failure modes were noted. The areas over which the interface failure modes, inplane failure through the shotcrete, brick or at the brick-shotcrete interface, were determined by counting the number of areas of occurrence of the failure modes relative to the size of a brick. The percentage of failure by each mode was then calculated by dividing the equivalent number of brick areas of the mode by the total number of bricks in the plane of the panel. After noting interface failure modes, the specimens were then broken into chunks so that cores from the shotcrete could be taken. When the specimens were broken into pieces, the welded wire fabric in the shotcrete was checked to see if any of the wires had fractured.

2.7 Comments on Testing Procedures

Because the interpretation of data and data points depends on the observer, the author feels that these areas should be discussed.

All physical measurements made by an observer were subject to interpretation. Thicknesses of the specimens at the angles, distances between parallel instrumentation rods, the angle from the vertical to the bed joint, gauge lengths, and bed length were all subject to the observer's judgment.

Load deflection readings were subject to considerable observer interpretation. During the testing, load and dial readings were easily taken in the low load ranges. However, at the higher ranges, the specimen creeped continuously. During testing, once the desired load was reached, the testing machine was turned off allowing data to be recorded; but, because of the creeping effect, all of the dial indicators were still moving. At that point the dial gauges were either allowed to settle or slow down or the corresponding dial gauges were read at the same time. A split second during the creeping condition could cause the readings to be off by 0.01 inches.

3. EXPERIMENTAL RESULTS AND DISCUSSION

3.1 General

The four most prevalent types of cracking that occurred during the testing of the specimens as illustrated in Fig. 3.1.1 were split tension, delamination, flexure, and crushing. Of the four most prevalent types of cracking, the split tension, shear, cracking occurred most often. The split tension was observed in each test. The split tension crack occurred mainly along the loaded diagonal of the specimen as illustrated in Fig. 3.1.1. However, at times two nearly parallel cracks would appear on each side the loaded diagonal. The diagonal crack typically occurred through the brick, along the mortar joints and through the shotcrete. Only the control specimen failed by cracking along only the bed and head joints forming a zig-zag crack which followed the loaded diagonal.

As shown in Fig. 3.1.1, delamination is an inplane cracking condition at or near the interface between the brick masonry and the shotcrete. Delamination occurs in the plane of the specimen and are due to direct tension forces at the interface.

Flexure cracking (Fig. 3.1.1) was observed only on the shotcrete side of a specimen. The flexure cracking was due to the shotcrete's inability to carry tensile forces.

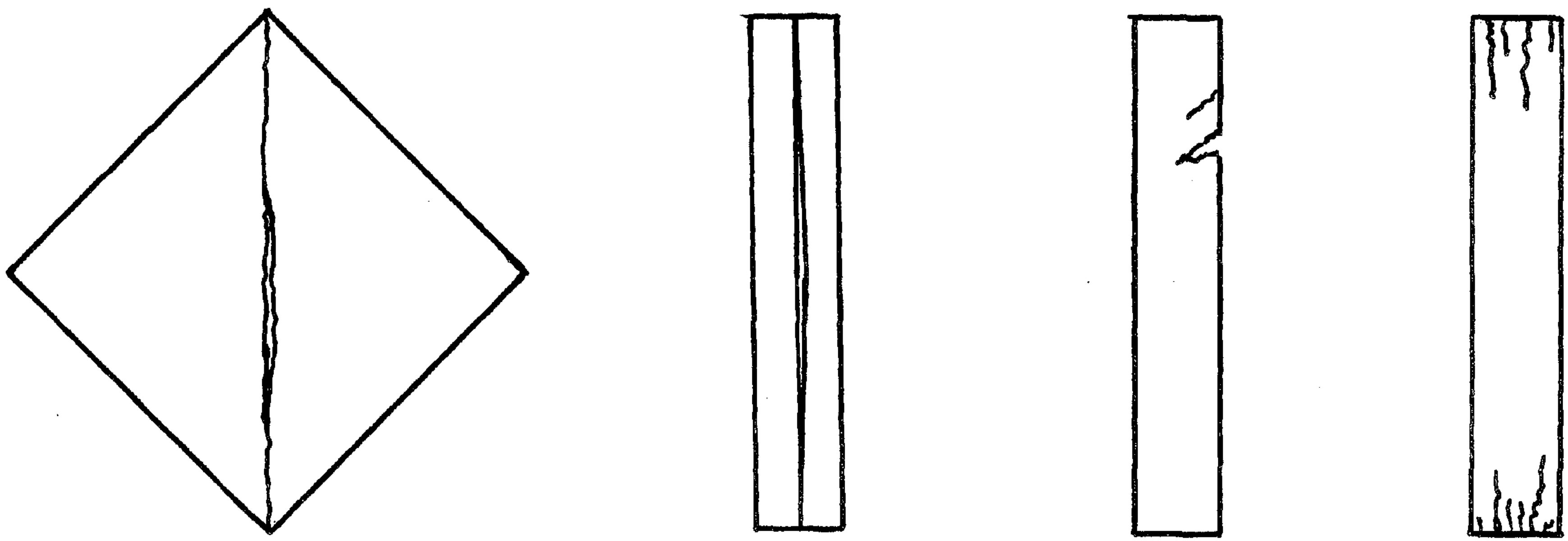


Figure 3.1.1 Various Types of Cracking That Occurred During Testing.

The tensile forces were caused because the specimens flexed under load due to the different moduli of elasticity or rigidity moduli of the brick masonry and the shotcrete.

Crushing cracks (Fig. 3.1.1) were observed at or near the loading shoes. The crushing cracks were due to the excessive bearing stresses at the loading shoes. The delamination, flexure, and crushing crack did not occur for each test.

For each loading cycle, the specimen attained a maximum load capacity. The load capacity, beyond the occurrence of the maximum, slowly decreased upon crushing. The specimens did not fail suddenly; they exhibited somewhat ductile properties absorbing energy during crushing.

The bonding between the brick masonry and shotcrete was strong enough for each specimen after the first loading cycle to allow a second half cycle loading test to be run. However, after the second half test, the specimens were broken up too much to allow the safe loading of additional half cycles.

3.2 Load-Strain

A load-strain hysteresis plot, typical of the nine shotcreted specimens, is presented in Fig. 3.2.1. The specimens exhibited a linear relationship between load and strain during the initial loading between levels (a) and (b) of Fig. 3.2.1. Upon reaching level (c), as illustrated in Fig. 3.2.1, vertical split tension cracking occurred along the loaded diagonal as shown in Figs. 3.1.1(d), 2.6.1 and 3.2.2 with subsequent

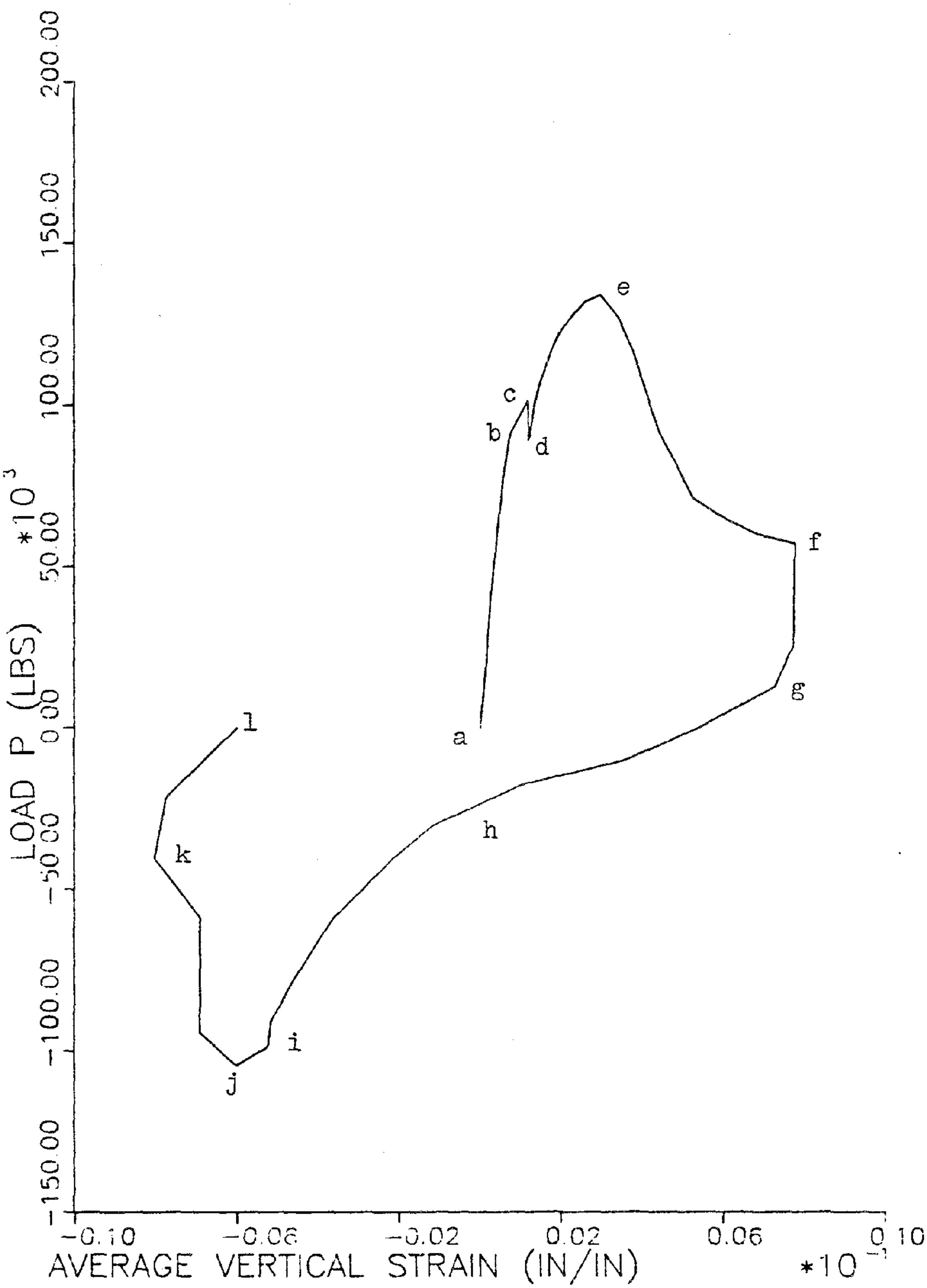


Figure 3.2.1 Typical Vertical Load-Strain Hysteresis Plot.

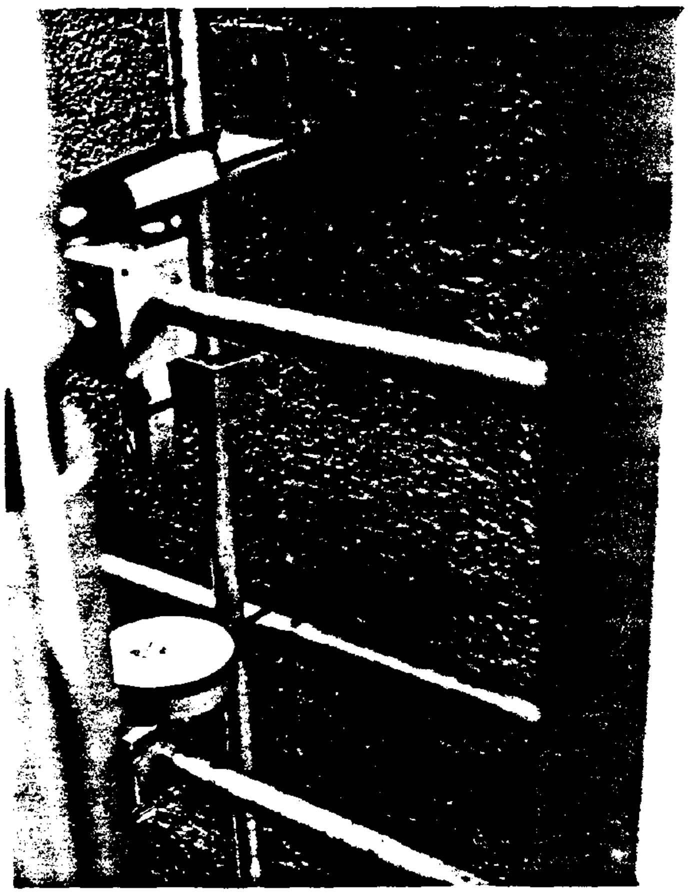


Figure 3.2.2 Close Up View of Vertical (Split Tension) Cracking.

loss of load capacity to level (d). After the formation of the vertical cracking, the relation between load and strain was nonlinear for the rest of the testing. From level (d) to (e), the specimens continued to carry increasing load. At level (e), the maximum load capacity of the specimens was reached. Additional vertical deflection was applied during levels (e) to (f); however, the load capacity continued to fall off. At level (f), generally large flexure or crushing cracks required that the testing cease, and the specimens were unloaded to level (g), the end of the first half cycle. From levels (g) to (h) of the reverse half cycle, the previously vertical crack was being closed with little to no resistance. From level (h) to (i), the specimens started to resist the applied deflections. The load-strain relationship is not linear in this region. At level (i), vertical split tension cracks occurred along the loaded diagonal. From level (i) to (j), little increased load capacity was exhibited as was observed from level (d) to (e) of the first half cycle. Level (j) was the ultimate capacity for the reverse half cycle (Fig. 2.6.2 and Fig. 3.2.3) From (j) to (k) load capacity dropped off with increased strains; at (k) the specimens were unloaded to (l), the end of the second half cycle. Figure 3.2.4 shows the extensive cracking that occurs upon the completion of a full cycle test.

Figure 3.2.5 shows a typical load-strain hysteresis plot parallel to the bed joints of the brick for the nine

Figure 3.2.3 Close Up View of Surface Failure.

Figure 3.2.4 View of Excessive Cracking in the Shotcrete.

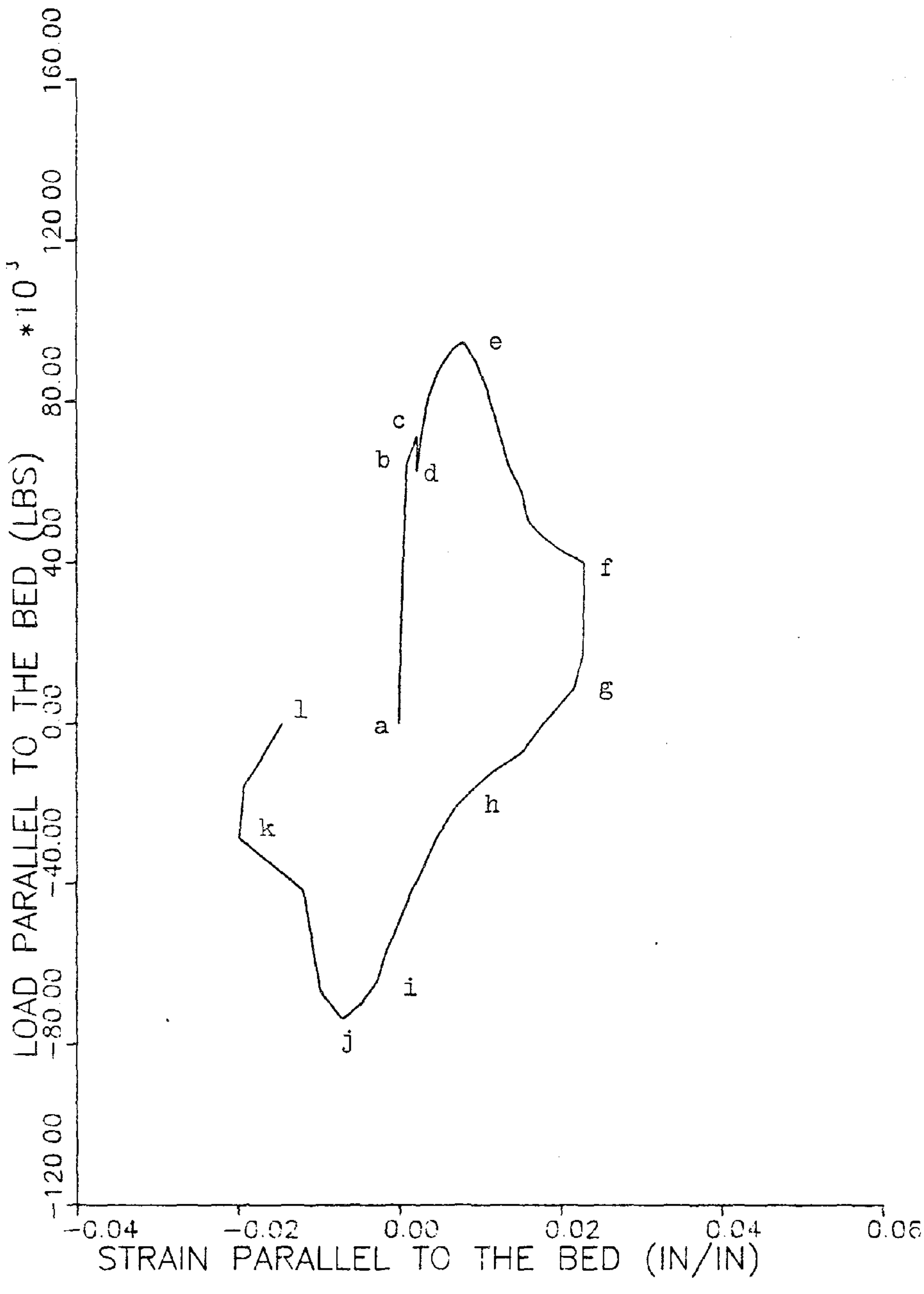


Figure 3.2.5 Typical Load-Strain Parallel to the Bed Hysteresis Plot.

shotcreted specimens. This plot is similar to the previous plot (Fig. 3.2.1); however, the strains parallel to the bed are larger because of the horizontal strain contribution. Levels (a) to (l) of Fig. 3.2.5 have similar significance to that of Fig. 3.2.1 described previously.

Figure 3.2.6 is a full cycle hysteresis plot of the load-stress parallel to the bed for the nine shotcreted specimens. The dotted, solid, and dashed lines are for the wet, epoxy, and dry interface surface conditions, respectively. Initially, the specimens exhibit the same linear behavior; however, around a load of 40 kip, the specimens begin to react differently.

Figures 3.2.7, 3.2.8 and 3.2.9 show the hysteresis plots for the load-strain parallel to the bed for the dry, wet, and epoxy surface conditions, respectively. The three figures are plotted at the same scale for easy comparison. The average strain parallel to the bed at the end of the first half cycle for the wet surfaced specimens was larger that that for the dry surface condition. Likewise, the epoxy surface conditioned specimens, on an average, had larger strains parallel to the bed than the wet surface condition specimens (Table 3.2.1).

The areas of the plots in Figs. 3.2.7 to 3.2.9 approximately represent the energy dissipated in the loading of the specimens. The form of the plots reveal the ductility of the system. By inspection of Figs. 3.2.7 to 3.2.9, the epoxy enhanced interface specimens are more ductile than the

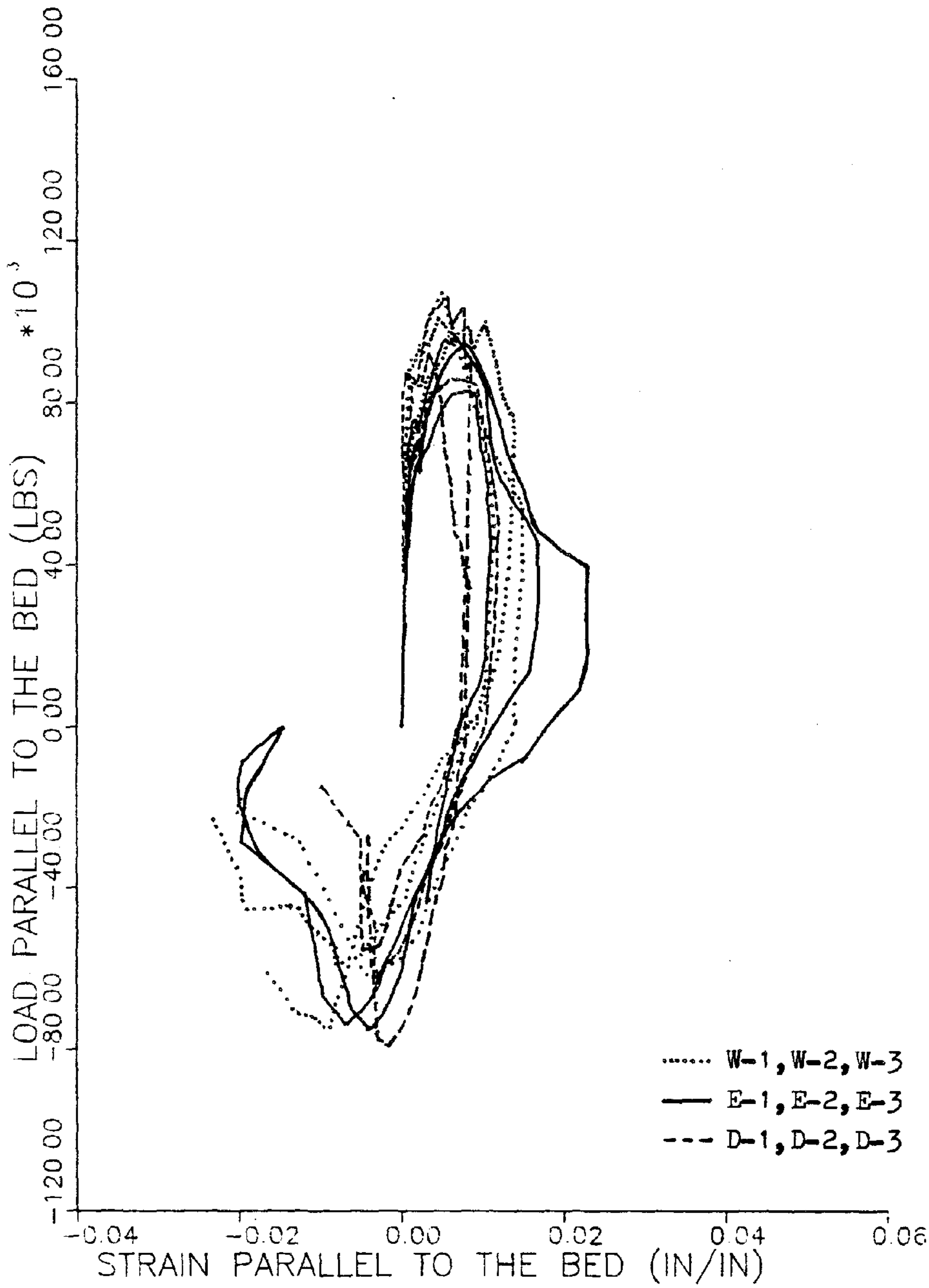


Figure 3.2.6 Multiple Load-Strain Parallel to the Bed Hysteresis Plot.

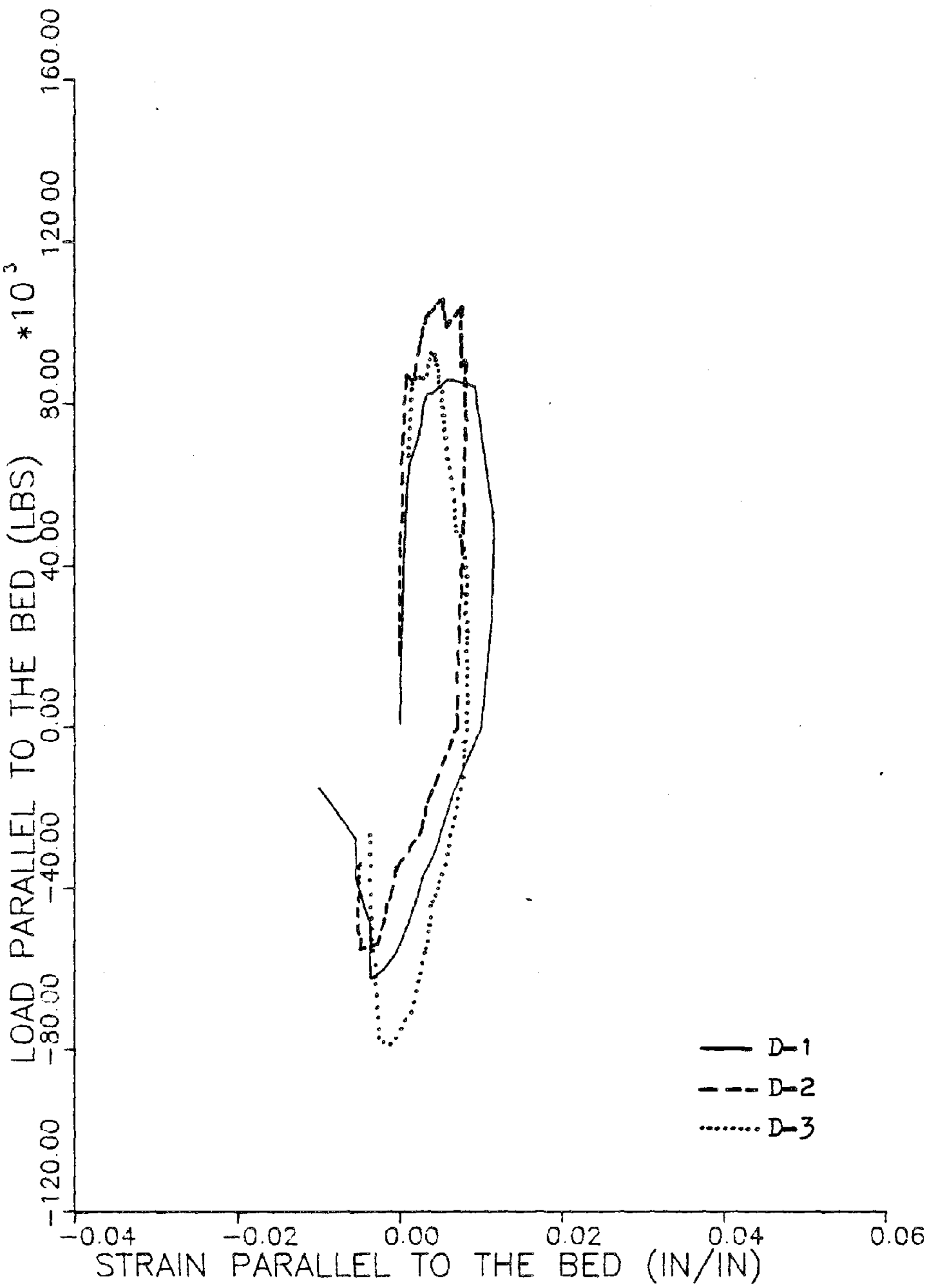


Figure 3.2.7 Load-Strain Parallel to the Bed Hysteresis Plot for the Dry Surface Specimens.

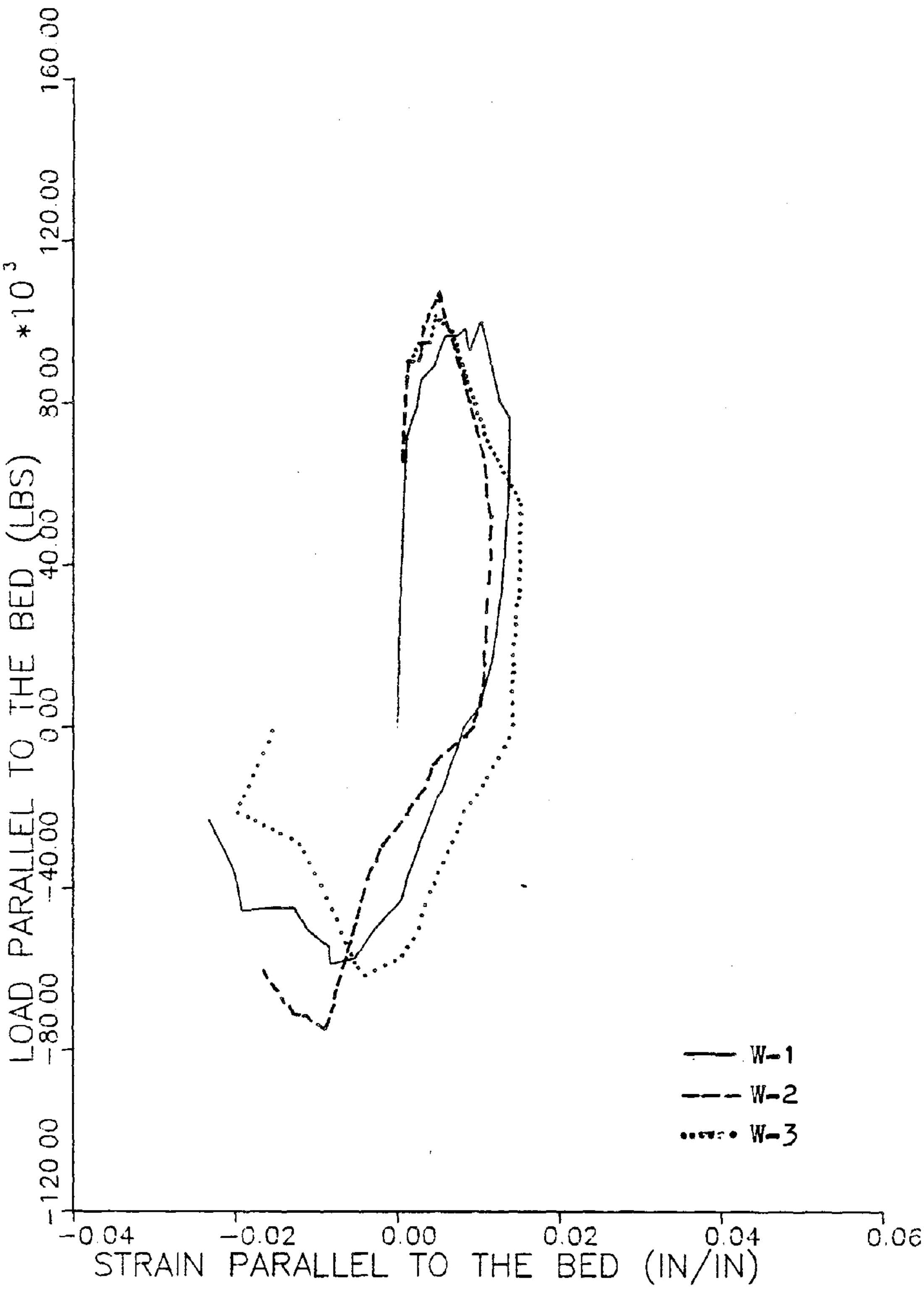


Figure 3.2.8 Load-Strain Parallel to the Bed Hysteresis Plot for the Wet Surface Specimens.

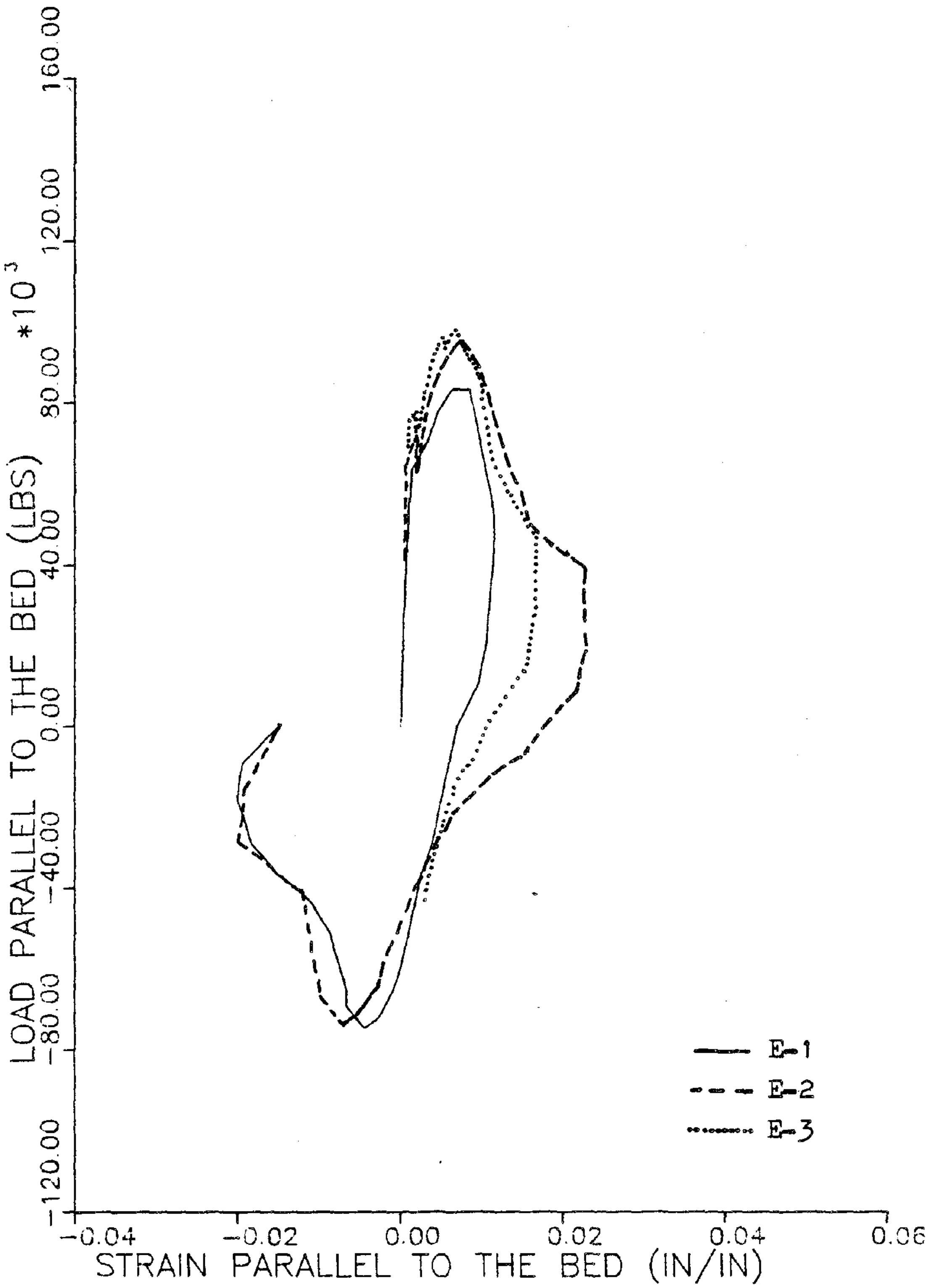


Figure 3.2.9 Load-Strain Parallel to the Bed Hysteresis Plot for the Epoxy Surface Specimens.

Table 3.2.1 Minimum and Maximum Load and Strains for Vertical and Parallel to the Bed Hysteresis Plots.

	Specimen Number	P_{min}(lbs)	P_{max}(lbs)	ϵ_{min} (in/in)	ϵ_{max}(in/in)
VERTICAL	W-1	-.832E5	.142E6	-.735E-2	.437E-2
	W-2	-.102E6	.144E6	-.846E-2	.339E-2
	W-3	-.872E5	.142E6	-.585E-2	.638E-2
	E-1	-.104E6	.119E6	-.856E-2	.406E-2
	E-2	-.105E6	.134E6	-.805E-2	.776E-2
	E-3	-.609E5	.138E6	0.00	.763E-2
	D-1	-.842E5	.122E6	-.559E-2	.468E-2
	D-2	-.822E5	.148E6	-.730E-2	.157E-2
	D-3	-.112E6	.132E6	-.285E-2	.227E-2
PARALLEL TO THE BED	W-1	-.588E5	.100E6	-.233E-1	.138E-1
	W-2	-.754E5	.107E6	-.166E-1	.115E-1
	W-3	-.617E5	.102E6	-.201E-1	.151E-1
	E-1	-.748E5	.832E5	-.200E-1	.116E-1
	E-2	-.739E5	.947E5	-.198E-1	.230E-1
	E-3	-.431E5	.976E5	0.00	.168E-1
	D-1	-.626E5	.861E5	-.989E-2	.117E-1
	D-2	-.550E5	.106E6	-.512E-2	.835E-2
	D-3	-.789E5	.933E5	-.397E-2	.806E-2

dry surface condition specimens. Likewise, the wet surfaced specimens are more ductile than the dry surface specimens. The extent of the ductility of these specimens can not be compared to that of ductile response of a reinforced concrete or steel beam; the observed ductility of the specimens allows comparison between specimen groups. As is shown in Table 3.2.2, the energy dissipated parallel to the bed by the epoxy surface specimens is almost 2½ times that for the dry surface specimens while only a fifth more energy is dissipated by the epoxied over the wet surface specimens.

The dry surface specimens, as shown in Fig. 3.2.7, crack initially on the diagonal resulting in a change in the shape of the load strain curve. The specimens reach an ultimate load capacity with an increased crack width. The specimens are allowed to reach this ultimate because of the presence of the reinforcing welded wire fabric. Delamination has occurred by the time the ultimate load capacity was reached. The specimens, at that point, were composed of two parts, the weak brick masonry and the reinforced shotcrete. The masonry was so weak that the wall was essentially composed of the shotcrete which was loaded at a large eccentricity. The steel fabric could not be utilized to its full potential because it was placed too close to the interface. The load capacity then dropped off exhibiting a brittle behavior.

The wet surfaced specimens, as shown in Fig. 3.2.8, exhibited similar response as the dry surface specimens; however, less delamination at the interface occurred, so

Table 3.2.2 Calculated Energies.
(in - lbs)

Specimen Number	Vertical Energy	Parallel to the Bed Energy
W-1	239700	600600
W-2	218700	484700
W-3	285500	593900
E-1	244800	497200
E-2	303500	707300
E-3	214000	357400
D-1	201100	379600
D-2	174200	295800
D-3	126700	291200

Energies are calculated by the trapezoidal rule from the data for the hysteresis plots.

The units are inch-pounds.

the shotcrete did not act independently and a small eccentricity was maintained. Load capacity did not drop off because more of the steel reinforcing in flexure was developed resulting in a more ductile system.

Figure 3.2.9 shows the epoxy surface specimens. The epoxy coating created a better bond at the interface of the specimen. The better bond condition allowed the brick and shotcrete to act more compositely, relative to the dry and wet surfaced specimens. This composite action developed more of the flexural-tensile capacity of the steel within the shotcrete. The development of the flexural steel produced a more ductile behavior.

3.3 Flexure and Delamination

Figures 3.3.1 and 3.3.2 show typical flexure plots for the first half and second half loading cycles, respectively. The plots represent the average lateral deflections of the panels measured at the gauge positions shown in Fig. 2.5.3. On the initial loading cycle, as illustrated in Fig. 3.3.1, the specimens moved rigidly in the direction of the shotcrete face at first and then continued to flex out of the plane of the specimen in that same direction. On the second half cycle, Fig. 3.3.2, the specimens immediately flexed to the maximum magnitude of lateral deflection developed during the first half cycle. The amount of flexure in the second half cycle is increased because a hinge, formed from the crack on the horizontal (previously the vertical split

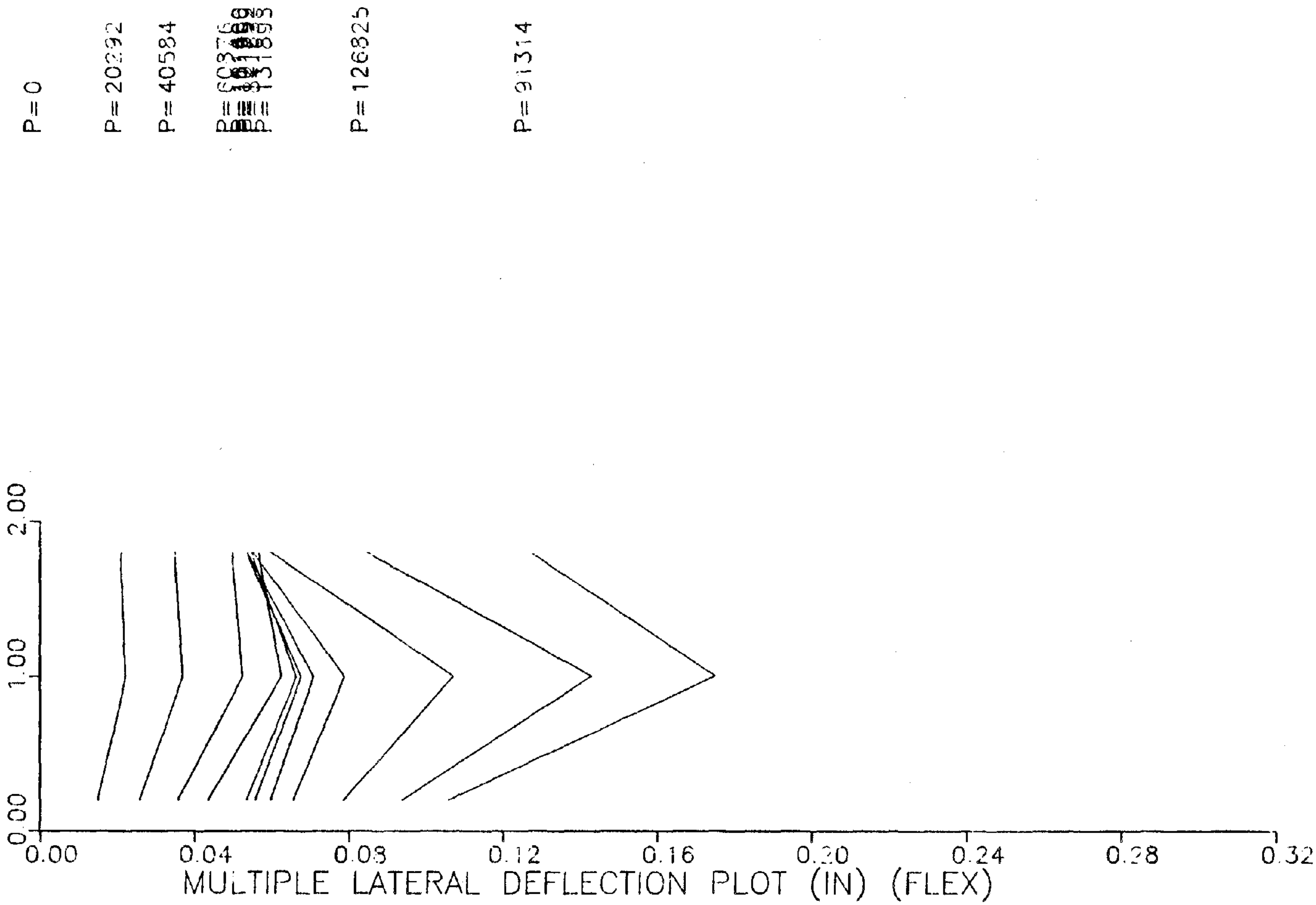


Figure 3.3.1 Typical Flexure Plot First Half-Cycle.

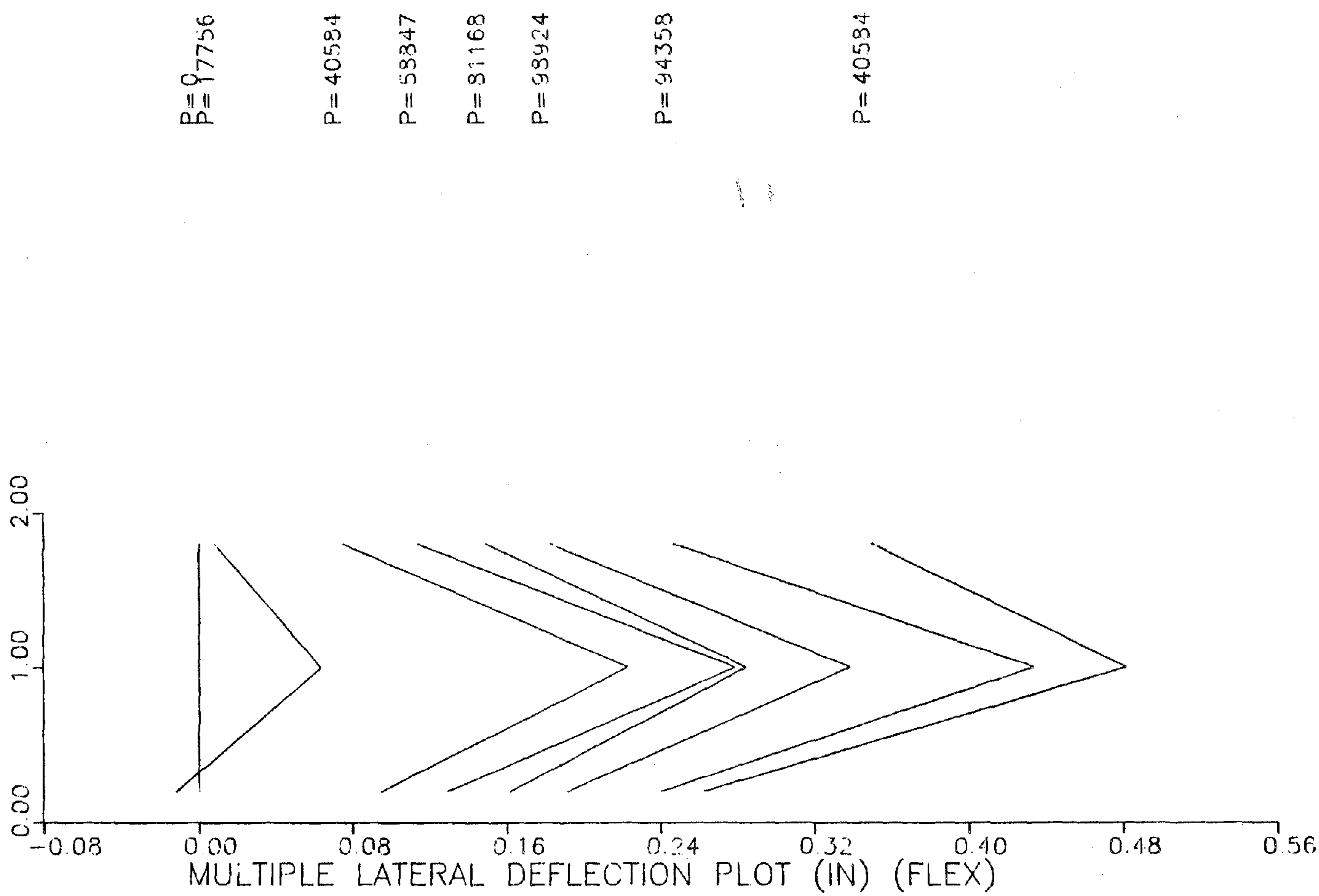


Figure 3.3.2 Typical Flexure Plot Second Half-Cycle.

tension crack from the first cycle) allowed greater lateral movement at the mid-height of the panel. Flexure cracks like that illustrated in Fig. 3.1.1 were formed because of the extreme flexure of the specimens.

The dry surfaced specimens generally flexed more than the wet specimens, with the epoxy specimens flexing the least at the same loads. The amounts of flexure could be attributed to the extent of development of flexural steel in the shotcrete and to the extent of composite behavior.

Typical delamination plots for the first and second half loading cycles are presented in Figs. 3.3.3 and 3.3.4, respectively. The plots represent the inplane lateral delamination (Fig. 3.1.1), for the gauge positions shown in Fig. 2.5.3. Figure 3.3.5 is a photo of delamination of a specimen at the interface between the brick masonry and shotcrete. Figure 3.3.6 is a photo of the spalling of a delaminated piece of shotcrete.

The delamination cracks were caused by direct tension in the plane of the specimens such that the degree of delamination was a function of the band and material strengths at the interface.

3.4 Load Capacities

Ultimate load capacities for the specimens are given in Table 3.4.1 for the first and second half cycles. The ultimate load capacities of the first and second half loading cycles correspond to levels (e) and (j) in Fig. 3.2.1. As

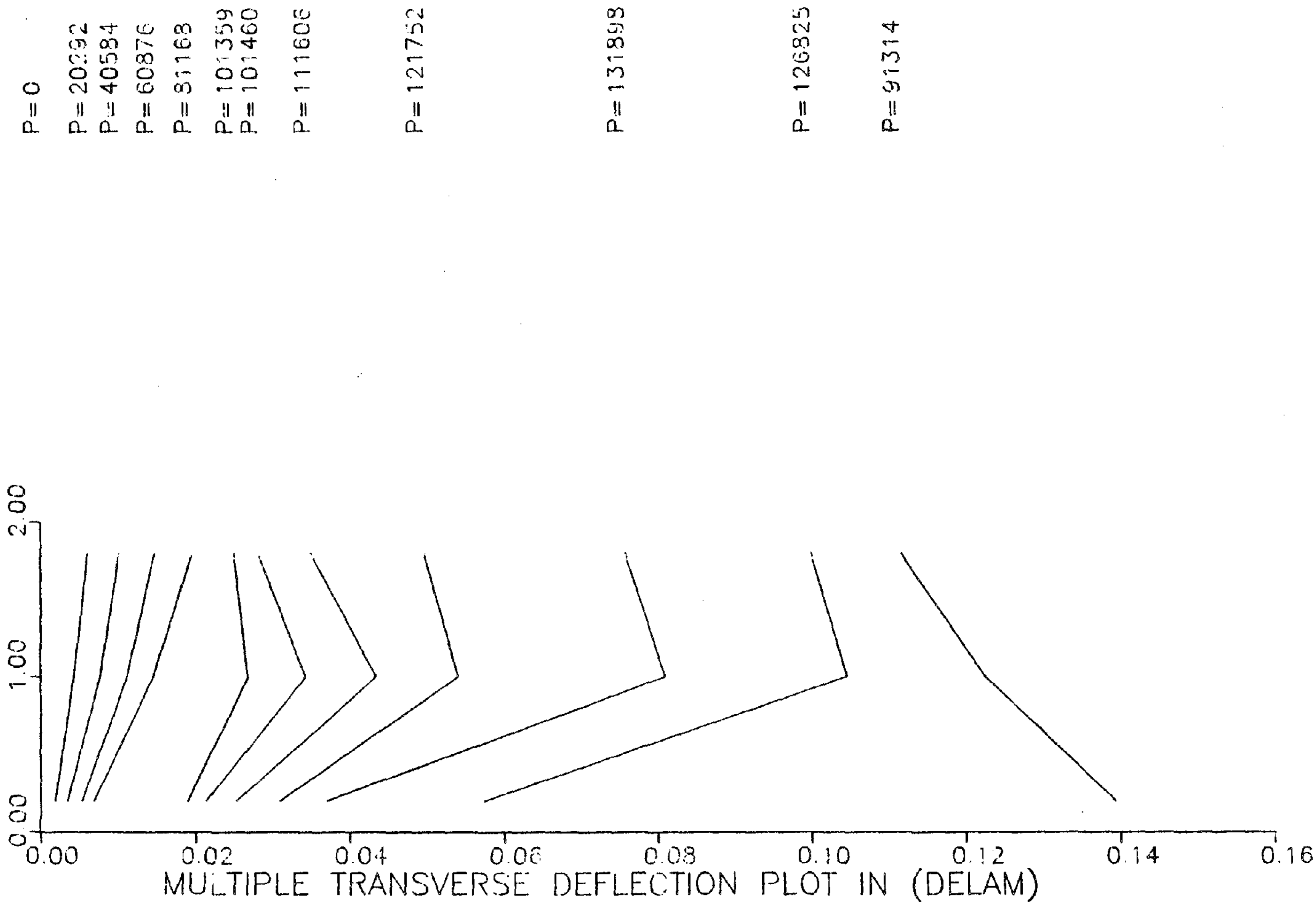


Figure 3.3.3 Typical Delamination Plot First Half-Cycle.

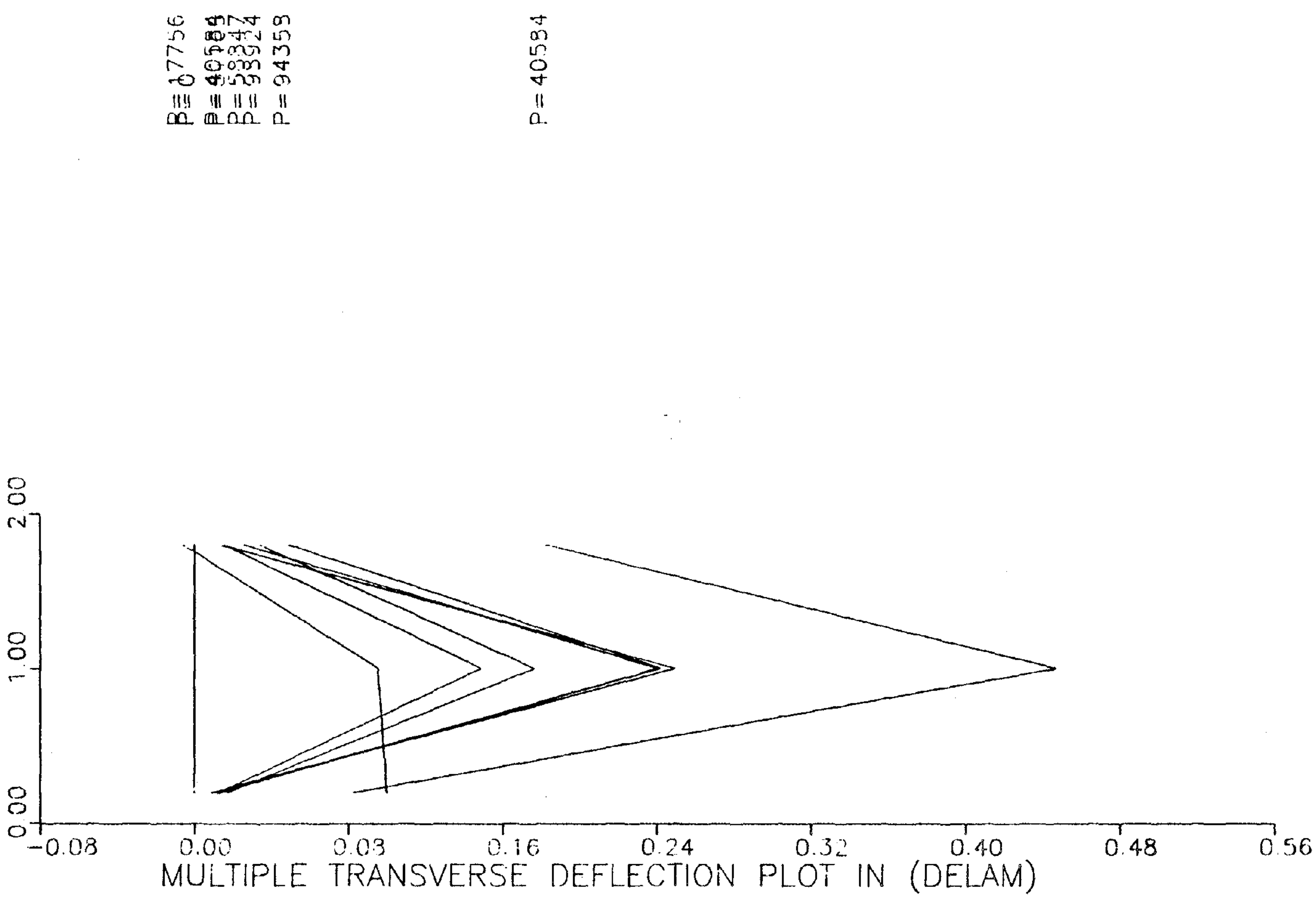


Figure 3.3.4 Typical Delamination Plot Second Half-Cycle.

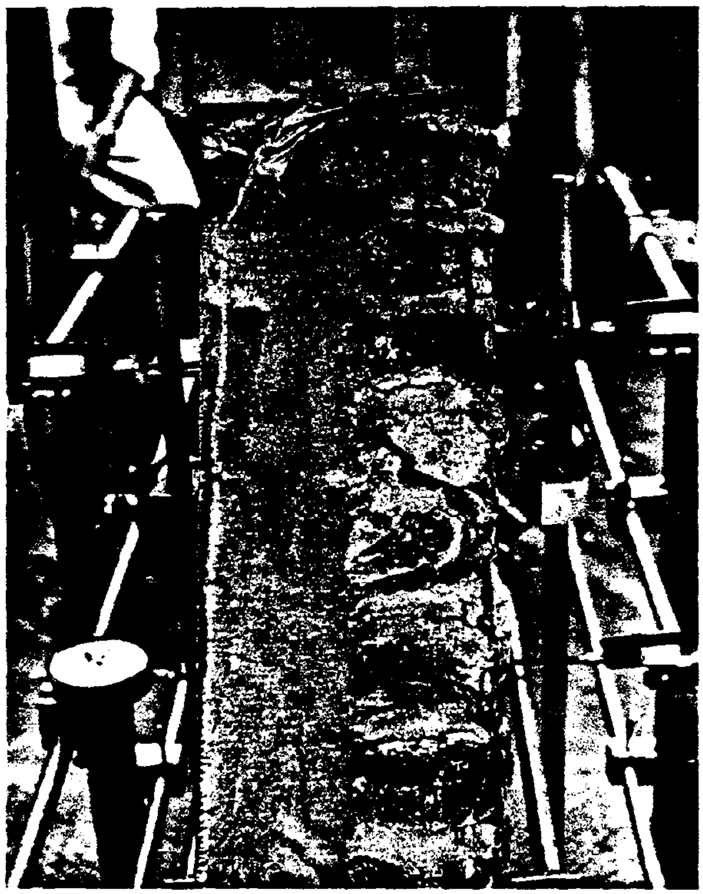


Figure 3.3.5 View of Delamination (Edge View).

Figure 3.3.6 Spalling of a Delaminated Piece of Shotcrete.

shown in Table 3.4.1, the ultimate load capacities for the first half cycle was always greater than that for the second half cycle. The cause for the lower second half cycle ultimate load capacity might be attributed to the fact that the specimen has been damaged.

Also presented in Table 3.4.1 are the percentage of second half ultimate load capacity with respect to the first half ultimate capacity. The mean value for each surface condition reveals that the epoxy-coated specimens attained a larger percentage of the first half cycle capacity on the second half cycle than the dry or wet surface specimens. There was no significant difference between the mean percentage for the dry or wet specimens. The epoxied surface assures that the bricks stay in place after the first half cycle such that the second half cycle has a capacity almost as high as that of the first half cycle.

3.5 Interface Failure Modes

At the brick masonry-shotcrete interface, three failure modes in the plane of the interface were observed. The failure modes were within the shotcrete, through the brick, and at the interface of the brick and shotcrete. A no-failure condition occurred if none of the three failure modes were observed. All four conditions, shotcrete, brick interface (surface), and non failure could occur in one specimen as shown in Fig. 3.5.1. The percentage of failure or non failure were based on the net are of the failure mode relative to the

Table 3.4.1 Ultimate Load Capacities for First and Second Half-Cycles (kips).

	Specimen	$P_u(1)$	$P_u(2)$	$P_u(2)/P_u(1)$ %	Mean %
WET	W-1	142.0	83.2	58.5	
	W-2	148.1	106.5	71.9	62.8%
	W-3	150.2	87.3	58.1	
EPOXY	E-1	121.8	103.5	85.0	
	E-2	133.9	104.5	78.0	78.6%
	E-3	138.0	100.4	72.8	
DRY	D-1	121.8	84.2	69.2	
	D-2	147.6	82.2	55.7	69.8%
	D-3	131.9	111.6	84.6	

$P_u(1)$ Ultimate load for the first half cycle

$P_u(2)$ Ultimate load for the second half cycle

Figure 3.5.1 Interface Value Modes.

total area of the specimen. Table 3.5.1 shows the relative percentages of failure modes for each shotcrete-treated specimen. The failure mode percentages were dependent on the amount of crushing the specimen underwent during the loading cycles and specifically after the second half cycle ultimate capacity was reached.

As shown in Table 3.5.1, very little failure occurred in the plane of the shotcrete at the interface. Failure through the brick was seen the most for the specimens with a wet surface while the interface (surface) bond failure was most prevalent for the dry surface specimens. However, there was more failure at the surface for the wet specimens than at the brick level. Finally, the epoxy enhanced specimens had little failure at the interface, i.e., high percentage of non failure, due to the occurrence of a good bond between the brick and shotcrete of the interface.

In general, the epoxy enhanced bond was stronger than that of the dry or wet surface condition bonds. The better bonding of the shotcrete to the brick for the epoxy-enhanced specimens created more composite action between the brick and shotcrete.

3.6 Other Observations

Loads at which various cracking (Fig. 3.1.1) occurred for the first and second half-cycles for the shotcreted specimens are presented in Table 3.6.1. First vertical (split tension) cracking for the brick and shotcrete on the first

Table 3.5.1 Interface Failure Modes
Percentage of Occurrence

	Specimen Number	Shotcrete %	Brick %	Surface %	No Failure %
WET	W-1	10.4	22.9	39.6	27.1
	W-2	6.3	6.3	15.6	71.8
	W-3	12.5	41.7	39.6	6.2
EPOXY	E-1	2.1	5.2	7.3	85.4
	E-2	-	-	-	90.0
	E-3	11.1	8.9	15.6	64.4
DRY	D-1	7.1	13.1	61.6	17.6
	D-2	9.8	20.7	45.1	24.4
	D-3	13.2	20.9	19.8	46.1

Based on the total interface area

Table 3.6.1 Loads at Which Various Cracking Occurred During Testing. (kips)

	Specimen	First Vert. Crack Brick	First Vert. Crack Shotcrete	First Delam. Crack	First Flexure Crack	First Chrushing Crack	Max. Load Capacity
WET	W-1	108.6	108.6	136.9	136.9	136.9	142.0
	W-1R	71.0	71.0	71.0	-	-	83.2
	W-2	121.8	121.8	-	88.8uc	144.6	148.1
	W-2R	40.6	27.4	81.2	-	-	106.5
	W-3	131.9	131.9	142.0	142.0	-	150.2
	W-3R	50.7	50.7	-	-	-	87.3
EPOXY	E-1	93.3	101.5	101.5	93.3	-	121.8
	E-1R	60.9	50.7	71.0uc	50.7uc	-	103.5
	E-2	101.4	101.4	71.0a	131.9	-	133.9
	E-2R	91.3	71.0b	-	-	-	104.5
	E-3	109.1	109.1	-	109.1a	109.1	138.0
	E-3R	45.7	45.7	50.7	-	-	100.4
DRY	D-1	106.5	116.7	121.8	121.8	-	121.8
	D-1R	50.7	50.7	40.6	-	-	84.2
	D-2	123.8	123.8	111.6	137.0	-	147.6
	D-2R	50.7	50.7	81.7	-	-	82.2
	D-3	121.8a	126.8	126.8uc	-	86.2uc	131.9
	D-3R	81.2	81.2	91.3uc	91.3uc	-	111.6

a occurrance at top of specimen
b occurrance at middle of specimen
c occurrance at bottom of specimen
uc occurrance upon crushing of specimen
R reverse half cycle data

half-cycle occurred at higher loads than on the second half-cycle for all specimens as shown in Table 3.6.1. The first delamination cracks occurred during the first half-cycle, with an increased degree of delamination occurring in the second half-cycles. Initial delamination cracks were prevelant in the dry and wet surfaced specimens; however, for the epoxied specimens, delamination cracking was limited to the top portion of the specimen.

The percentage of first vertical (split tension) crack load for the brick and shotcrete with respect to the ultimate load capacity for both cycles of the specimens are presented in Table 3.6.2. The load percentage of the ultimate load capacity at which first cracking on the vertical occurred in the brick masonry and the shotcrete was higher on the first half-cycle, an average of 83 percent of ultimate, than the second half-cycle, an average of 60 percent of ultimate. The first half-cycle percentage of ultimate was higher than the second half-cycle because the specimens were damaged on the first half-cycle.

3.7 Control Specimen

The control specimen, C-3, was included in the testing program to provide information about the load capacity and load deflection response of the brick masonry alone. Specimens C-1 and C-2 were broken prior to testing and replacement was not possible. The control specimen was tested in the same manner as the shotcreted specimens; however, only a

Table 3.6.2 Percentage of Ultimate Load at Which First Vertical Cracking Occurred in the Brick Masonry and the Shotcrete

Specimen			$\frac{P_{\text{first cracking brick masonry}}}{P_u}$ %	Mean	$\frac{P_{\text{first cracking shotcrete}}}{P_u}$ %	Mean
FIRST	WET	W-1	76.5		76.5	
		W-2	82.2	82.2	82.2	82.2
		W-3	87.8		87.8	
	EPOXY	E-1	76.6		83.3	
		E-2	75.7	77.1	75.7	79.4
		E-3	79.1		79.1	
	DRY	D-1	87.4		95.8	
		D-2	83.9	87.9	83.9	91.9
		D-3	92.3		96.1	
REVERSE	WET	W-1R	85.3		85.3	
		W-2R	38.1	60.5	25.7	56.4
		W-3R	58.1		58.1	
	EPOXY	E-1R	58.8		49.0	
		E-2R	87.4	63.9	67.9	54.2
		E-3R	45.5		45.7	
	DRY	D-1R	60.2		60.2	
		D-2R	61.7	64.9	61.7	64.9
		D-3R	72.8		72.8	

first half loading cycle was run because it was destroyed during the first half-cycle test.

The brick-mortar bond, modeled after brick-mortar bonds of brick masonry buildings from the late nineteenth and early twentieth centuries, was not strong enough to hold the control specimens together when they were picked up by the "C" clamp for placement into the loading machine. Speciman C-1, broke under its own weight when it was picked up without previously being banded. Specimen C-2 was broken by vandals and could not be replaced. Because of the low band strength between the brick and mortar, the last specimen, C-3, was banded with steel bands prior to placement into the test machine. The resulting prestressing force of the bands could not be calculated.

The control specimen, C-3, first was loaded and unloading in the banded state as illustrated in Fig. 3.7.1 from levels (a) to (d). It was not intended to crack the specimen; however, between level (b) and (c), small split tension cracks were observed. At level (d) the banding was cut, allowing the specimen to regain equilibrium. Then at level (d) the unbanded specimen was loaded to failure at (f). The extremely low ultimate load capacity for the unbanded load cycle might be attributed to the previous cracking.

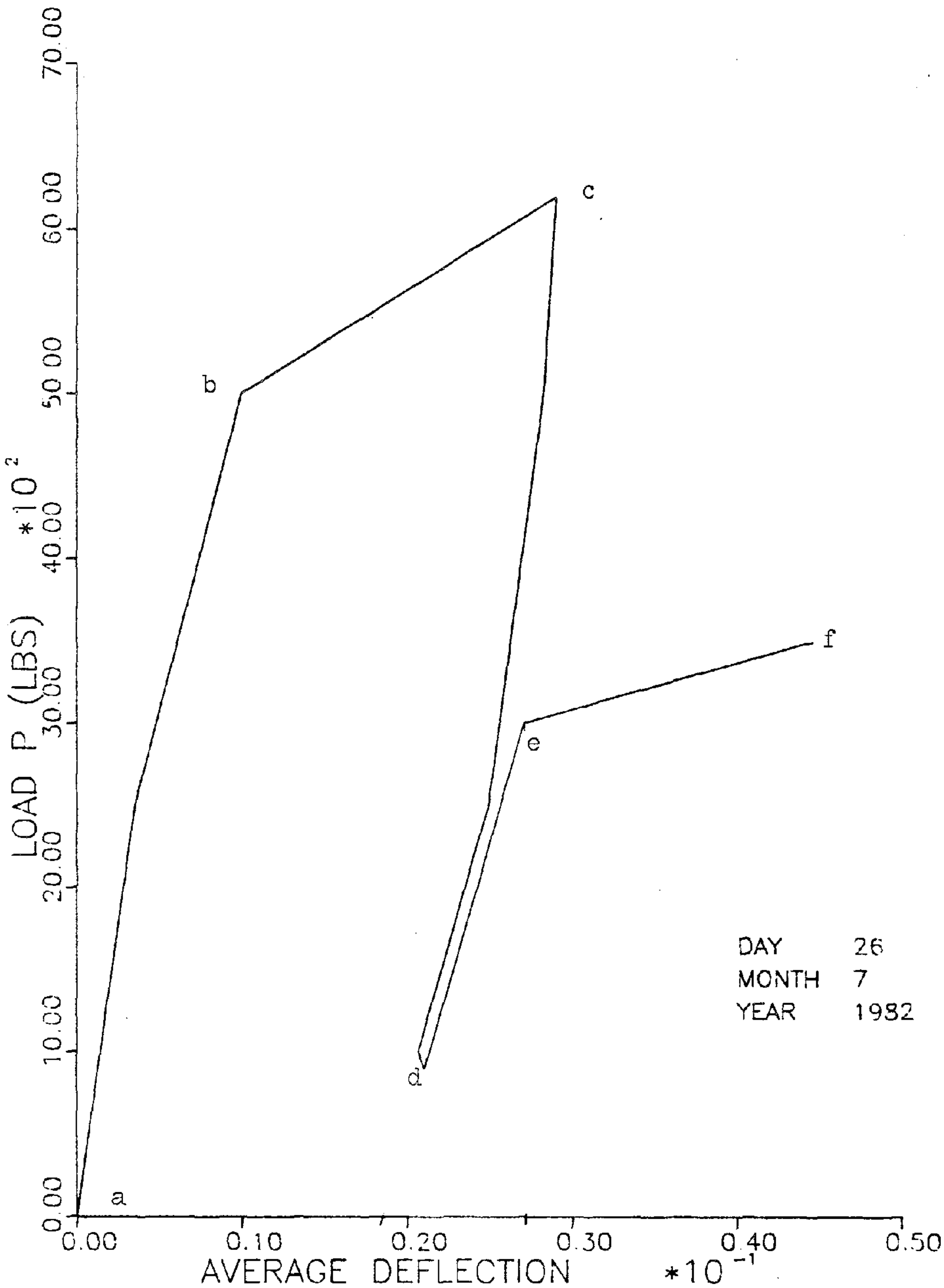


Figure 3.7.1 Load-Deflection for Control Specimen C-3.

4. CALCULATION RESULTS

4.1 Diagonal Stiffness Modulus for the Brick Masonry

The diagonal stiffness modulus for the brick masonry was calculated using the load deflection curve, Fig. 3.7.1, of the control specimen, C-3. The diagonal stiffness modulus, similar to the modulus of elasticity but on a 45-degree angle to the bed joint of the brick masonry, was calculated as the change in stress divided by the change in strain, slope, of the banded load-deflection plot for the initial response between levels (a) and (b) of Fig. 3.7.1. The initial banded response was used because the specimen was cracked prior to determining the unbanded response. The stress was calculated by dividing the applied load by the equivalent (effective) width (Fig. 4.1.1) times the average brick thickness. The effective width is discussed in Appendix C. The change in strain was calculated as the change in deflection divided by the average vertical gauge length. The resulting diagonal stiffness modulus, $E_{m\measuredangle}$, for the brick masonry was 1.72E5 psi. The modulus of elasticity of the shotcrete was 5.6E6 psi (Table 2.4.4). The ratio of brick to shotcrete modulus is 1 to 32.

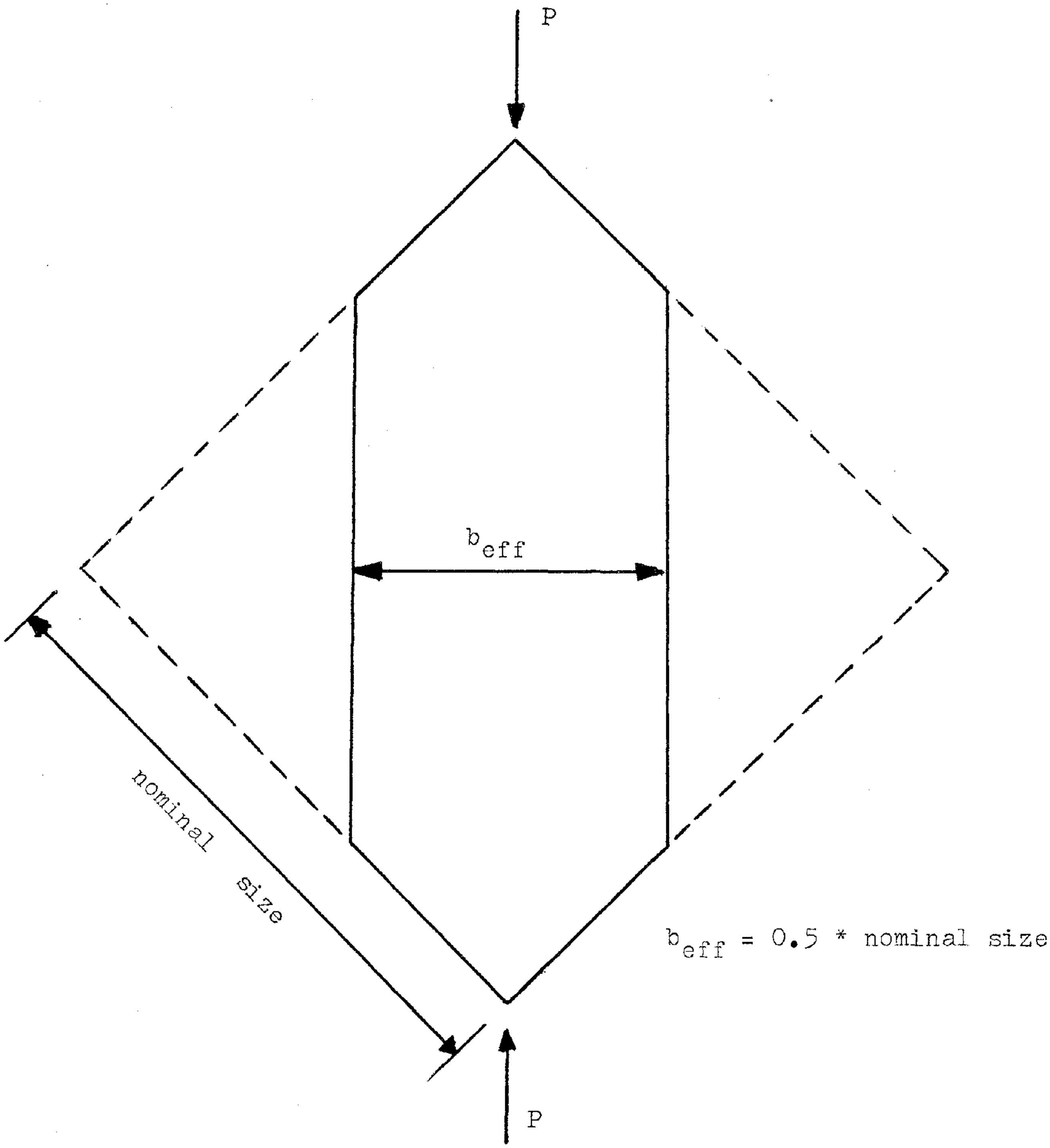


Figure 4.1.1 Effective Width Diagram.

4.2 Load Calculations

Load capacities based on the measured strains as illustrated in Fig. 4.2.1, the effective width of the specimen as illustrated in Fig. 4.1.1 from the findings given in Appendix C, the modulus of elasticity of the shotcrete (Table 2.4.4), and the diagonal stiffness modulus of the brick masonry of Section 4.1, were calculated and compared to the actual measured load for each specimen. The calculated load capacities were evaluated by using the average strain over the thickness of the shotcrete and brick times their thicknesses, stiffness moduli, and effective widths as illustrated in Fig. 4.2.1. The load carried by the masonry and the shotcrete were calculated by the following formula:

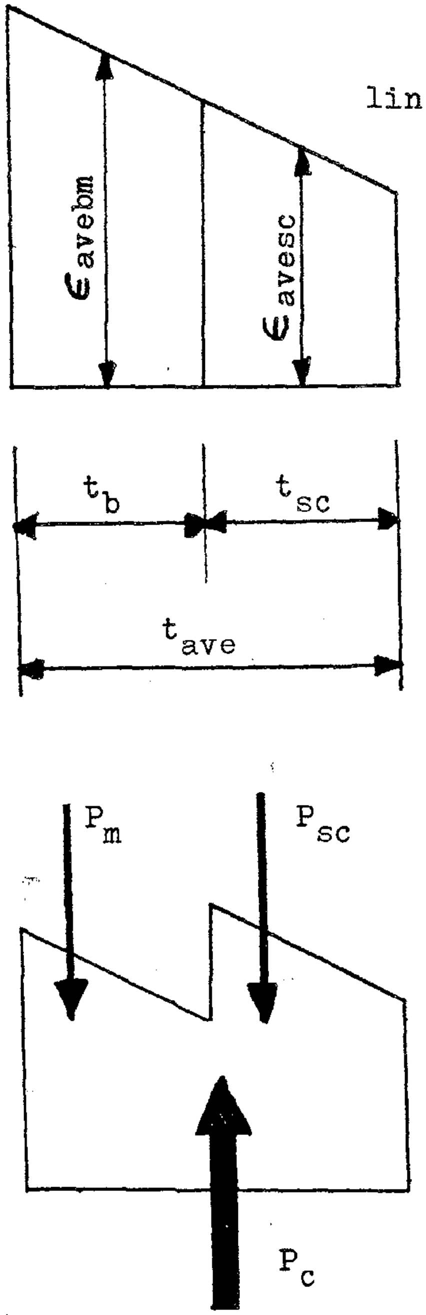
$$P_m = E_{m\measuredangle} * \varepsilon_{avebm} * t_b * b_{eff}$$

$$P_{sc} = E_{sc} * \varepsilon_{avesc} * t_{sc} * b_{eff}$$

with

$$P_c = P_m + P_{sc}$$

where P_m is the load capacity of the masonry, P_{sc} is the load capacity of the shotcrete, $E_{m\measuredangle}$ is the diagonal stiffness modulus for the brickmasonry, E_{sc} is the modulus of elasticity of the shotcrete, ε_{avebm} and ε_{avesc} are the average strains strain diagram



stress diagram

linear strain approximation

E_m	diagonal stiffness modulus for the brick masonry
E_{sc}	mod. of elasticity for the shotcrete
b_{eff}	eff. width
ϵ_{avebm}	average strain brick masonry
ϵ_{avesc}	average strain shotcrete
t_b	brick thickness
t_{sc}	shotcrete thickness
P_m	mosaonry load
P_{sc}	shotcrete load
P_c	P_m+P_{sc}

$$P_m=E_m*\epsilon_{avebm}*t_b*b_{eff}$$

$$P_{sc}=E_{sc}*\epsilon_{avesc}*t_{sc}*b_{eff}$$

$$P_c=P_m+P_{sc}$$

Figure 4.2.1 Calculated Load Theory Diagram.

over the thicknesses of the brick and shotcrete, respectively, (Fig. 4.2.1), t_b and t_{sc} are the thicknesses of the brick and shotcrete, and b_{eff} is the equivalent width of the specimen as shown in Fig. 4.1.1. P_c is the cumulative load capacity of the specimen.

The sum of the masonry and shotcrete load capacities, P_c, was then compared with the load corresponding to the strain condition. Figure 4.2.2 shows a plot of the calculated loads, P_c, vs the actual load measured, P_a, for a diagonal stiffness modulus for the masonry of 1.72E5 psi and a modulus of elasticity for the shotcrete of 5.62E6 psi.

The calculated load capacity, P_c, was larger for all the cases investigated. The calculated loads for the highly strained conditions were much larger than the actual load capacity observed. The calculated values were scattered because constant stiffness moduli for the brick masonry and shotcrete replaced the variable moduli of the experiment.

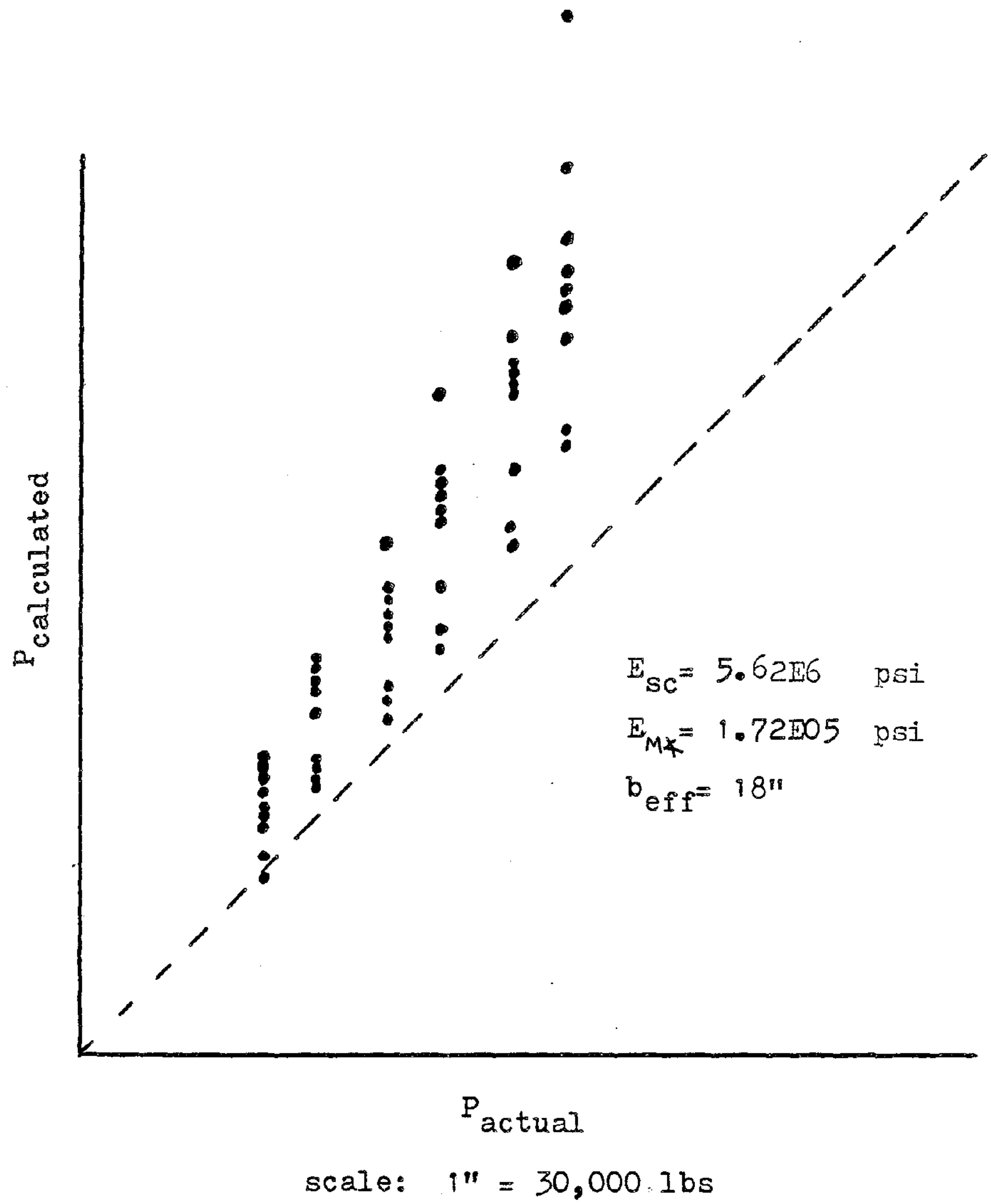


Figure 4.2.2 Calculated Load vs Actual Load, E_s = 5.620E6 psi.

5. CONCLUSIONS AND RECOMMENDATIONS

Nine shotcrete-treated specimens: three with dry interfaces, three with wet interfaces, three specimens with an epoxy-enhanced interface and one control specimen were tested to investigate the interface bonding characteristics and the extent of brick-shotcrete composite action. Some degree of ductile behavior was exhibited by each of the nine specimens; however, the epoxy-enhanced specimens proved to absorb the most energy without as much damage occurring to the specimens. The dry surfaced specimens were the least ductile with the wet surfaced specimens slightly less ductile than the epoxy surfaced specimens. Less delamination and flexure occurred for the epoxied specimens leading to an indication of more composite action between the brick masonry and the shotcrete.

The ultimate load capacities for all the shotcrete-treated specimens were about the same. The shotcrete treatments assured that the brick masonry reached it's maximum load capacity, as evidenced by vertical cracking through the bricks and mortar. The control specimen without shotcrete cracked along the bed and head mortar joints, not through the brick.

The first half-cycle loading caused a decreased ultimate load capacity for the specimens on the second half-cycle. The decreased reversed load capacity was due to the damage that occurred during the first half-cycle. A full cycle, first and second half-cycle, load test was required to separate the brick and shotcrete portions.

Better composite action between the brick masonry and the shotcrete was observed for the epoxy surfaced specimens than the wet surfaced and dry surfaced specimens. The dry surfaced specimens exhibited little to no composite action. The epoxy, wet and dry surfaced specimens are regarded as extremely brittle in reference to a well detailed reinforced concrete panel. When considering the design of a repaired or retrofitted wall for dead, live and earthquake loadings where the ultimate load capacity is not exceeded, a wet surface at the interface would provide composite action properties that are as good as that for epoxied surface brick.

The stiffness of the shotcrete is almost 32 times the stiffness of the brick masonry at a 45-degree angle to the bed joints. For single wythe brick masonry panels treated with shotcrete, the shotcrete governs the response of the retrofitted panel resulting in little to no effective contribution by the brick masonry.

The use of the composite action between the shotcrete and brick masonry in analysis is not recommended in situations where a horizontal diaphragm is connected to the shotcrete treatment of the masonry wall because the brick contributes little to the stiffness. If the horizontal diaphragm is not connected to the shotcrete but is connected at or near the interface of the masonry wall, then shear at the interface and flexure of the brick-shotcrete composite should be considered.

The current practice of analysis of a shotcrete reinforced masonry wall with vertical loads carried by the masonry and lateral loads carried by the shotcrete is reasonable.

The ASTM E519-74 test (11) method geometry does not allow a good quantification of the composite action between the two materials but does allow the qualitive determination. The test procedure is not recommended for the investigation of interface bonding characteristics and composite action of shotcrete reinforced brick masonry assemblages.

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