BEHAVIOR OF WALL PANELS

UNDER STATIC AND DYNAMIC LOADS II

by

JACK KINSTLINGER

B.C.E., Rensselaer Polytechnic Institute

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DEPARTMENT OF CIVIL AND SANITARY ENGINEERING

1954

Signature of Author .

Certified by . . . .
Thesis Supervisor

Signature of Chairman of
Departmental Committee
on Graduate Students . . . .

May 7, 1954

Professor Leicester F. Hamilton
Secretary of the Faculty
Massachusetts Institute of Technology
Cambridge 39, Massachusetts

Dear Sir:

In partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering, I herewith submit to you this thesis entitled, "Behavior of Wall Panels Under Static and Dynamic Loads II".

Respectfully yours,

Jack Kinstlinger

ACKNOWLEDGEMENT

The author wishes to acknowledge his obligation for the cooperation and many helpful suggestions given him by Professors M. J. Holley, Jr. and C. H. Norris, by his colleague Mr. J. M. Cord, by Mr. C. M. Pickett, Jr., and especially by Professor R. J. Hansen, whose invaluable guidance and aid made this investigation possible. The author also wishes to acknowledge his appreciation to the Corps of Engineers, Department of the Army for financial support which allowed this investigation to be conducted.

CHAPTER I

SUMMARY

BEHAVIOR OF WALL PANELS UNDER STATIC AND DYNAMIC LOADS II

by

JACK KINSTLINGER

Submitted to the Department of Civil and Sanitary Engineering on May 7, 1954 in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

GENERAL

This report presents a description of the experimental techniques, the test results, and the conclusions from a series of laboratory tests on the following wall panel materials subject to transverse bending by static or dynamic loads: brick masonry, reinforced brick masonry, brick masonry with reinforced gunite, concrete block, tile partition, steel siding, and asbestos cement siding.

BRICK MASONRY

Both plain and reinforced brick masonry wall panels and beams were tested under static and dynamic loads. Curves of moment vs deflection are presented for spans 3, 6, 9, and 12 ft.

The influence of support fixity, mortar strength, direction of support, and effect of dead load stresses are investigated and reported.

The use of the plastic theory for the design of reinforced masonry is recommended.

GUNITED MASONRY WALLS

The beneficial effect to be expected from applying a reinforced gunite surface to a masonry wall is studied in a limited number of tests. The use of standard plastic design procedures is recommended on the basis of the results.

MISCELLANEOUS WALL PANELS

A smaller number of tests were performed on concrete block panels, partition tile panels; both on 3 ft span only, and on corrugated asbestos-cement and corrugated steel siding both on 3 ft spans only. The effect of standard fastening technique on the corrugated asbestos-cement and steel siding was investigated. On the basis of the limited tests curves of load for buckling and/or failure vs span length are presented for the corrugated asbestos-cement and corrugated steel siding.

Thesis Supervisor: Robert J. Hansen
Associate Professor of
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CHAPTER 2

INTRODUCTION

2.1 OBJECTIVE

The objective of this investigation is to provide the basic information with which to predict failure conditions for typical wall panel construction subjected to transverse load.

Research into the behavior of wall panels loaded transversely was undertaken in August 1951 in the Structural Dynamics Laboratory at Massachusetts Institute of Technology. Results are presented in two reports - the report published in August 1952 (referred to as the first report) and the present thesis report which includes all work done in the research program from its inception in 1951 to the date of completion.

Since in the MIT laboratories dynamic tests could be conducted only on members of short span length, it was decided that the objectives could be met by a determination of (i) the static and dynamic behavior of short span members of the various types of wall panels, and (ii) the static behavior of similar members having longer span lengths. The information gained from these comparative static-dynamic tests could then be used to convert the results of static tests on a few full-scale wall members to a predicted dynamic behavior.

2.2 PREVIOUS KNOWLEDGE

Many tests have been conducted on the various wall panel materials considered in this program. However, very little data is generally available on the behavior of these materials under dynamic loads. Furthermore, the available static test data which might be useful in considering and predicting dynamic behavior is in many instances inconsistent and incomplete. This section presents and discusses some of the more pertinent test results on the materials under consideration.

2.2.1 Plain Brick Masonry

Up to the time of publication of the first report (1)* on this program, available knowledge on the behavior of brick masonry under transverse loads was exceedingly limited, due to the fact that primary interest had been in the behavior of brick masonry under compressive or bearing, rather than transverse, loads.

Experimental studies of the strength and behavior of brick masonry under transverse loads were conducted in 1928 and 1938 by the National Bureau of Standards (2,3), in 1930 by the University of California (4), and in 1931 by the California Institute of Technology (5). All four testing programs have made significant contributions to present knowledge of the subject; unfortunately, however, all have been quite limited in scope so that, while representative values of strength have been determined, no conclusive patterns of behavior have been deduced. In all cases, the walls were supported at the edges, with concen-

*Numbers in parenthesis refer to references in Appendix A.

trated loading either at midspan or at the third points; in no case was a uniformly-distributed transverse load applied.

These investigations indicated that the modulus of rupture of the walls tested ranged from 30 to 400 psi. Studies of the behavior of brick walls indicate that the strength of bond between brick and mortar is the key property in the transverse strength of brick walls. Tests of the tensile strength of this bond show a range of 1 to 100 psi. Tests have also been made of the shear and bending rupture strength of this bond.

No tests on brick masonry under dynamic or impulsive loading had been conducted prior to this investigation, although the Bureau of Standards did conduct transverse impact tests on walls by dropping sand bags on the specimens (3). It is noted that a wide scattering of results is usually obtained from tests on the transverse (and other) behavior of brick masonry due to the inconsistent properties of the material.

2.2.2 Reinforced Brick Masonry

Although the use of reinforced brickwork has been rather limited, numerous tests and investigations on the behavior of reinforced brick masonry slabs and beams under static loads have been conducted.

An article written by James H. Hansen (7) in 1933 gives data, results, and conclusions of a number of independently conducted investigations into the behavior of reinforced brick masonry slabs and beams.

A review of these tests yields the following findings:

(a) Deflections in reinforced brick masonry beams and slabs are consistent and small within working loads. (Usually they are much less than the allowable deflection of 1/360 of the span length, based on the cracking resistance of plaster).

(b) Upon removal of the load, recovery of deflection--sometimes 100% recovery--is possible.

(c) Granting the assumptions upon which the calculations are based, rods inserted in mortar joints will develop bond stresses as high as 384 psi, with an average of 229 psi when rods are placed in the joints perpendicular to the top face of the beam.

(d) Shear stresses are affected by the manner in which bricks are placed, and, by proper design, can be developed to as high as 154 psi without shear reinforcement.

(e) The introduction of stirrups increase the resistance of the beams to shear.

(f) When properly designed, the tensile strength of longitudinal reinforcement can be fully developed.

(g) The compressive strength of the masonry in beam action is greater than the compressive strength of wall panels in direct compression.

Hansen's calculations were based on the assumption that the value of 20 for n, the ratio of elasticity moduli, would give a fairly accurate distribution of stresses, and that the theories of the Joint Committee on Standard Specifications for Reinforced Concrete (8) would apply.

Although the experimenters found a considerable upward movement of the neutral surface as the load increased, their calculations for a limited number of beam tests showed fiber stresses and values of j and k to be quite close to those of a similarly reinforced concrete beam.

In view of the evidence submitted in his paper, Hansen presented the following conclusions:

(a) The assumptions accepted in the design of reinforced concrete structures can be used in the design of reinforced brick masonry items such as beams and slabs.

(b) The recommended working stress of $0.4f'_c$ can be safely allowed in reinforced brick masonry. This would allow approximately 750 psi for cement-mortar brick masonry, and 500 psi when cement-lime mortar is used, based on tests of full-sized wall panels.

(c) A test of piers made up of two or three brick and mortar joints would give better results on which to base such working stresses and would probably increase the foregoing values by about 10%.

(d) The use of smaller bars ($\frac{1}{2}$ in. round) is more feasible and will result in higher bond strength.

(e) Bond stresses should be limited to 80 psi.

(f) Shear stresses should be limited to 25 or 30 psi, necessitating stirrups or bent-up bars in most beams.

(g) To develop maximum strength, the reinforcing rods should be placed in joints perpendicular to the top face of the beam

(h) The face of the brick that develops the greatest compressive strength should be placed normal to the line of compressive stress in the beam.

(i) The value of the ratio, E_s/E_b is somewhere between 20 and 30, where E_s is the modulus of elasticity of the steel, and E_b is the modulus of elasticity of the brickwork.

(j) The mortar should consist of 1 part cement, 0.25 part lime, and 3 parts sand by volume.

(k) Builders should adhere rigidly to all requirements of good brick masonry, that is, the proper wetting of the brick, and complete filling of joints.

(l) Further well-directed study and testing will undoubtedly justify an increase in the allowable stresses.

A series of tests was conducted by Prof. M. O. Withey at the University of Wisconsin (9) in 1933 on the behavior of brick masonry beams under static loading. The tests were made on numerous beams, differing in the type of brick used, in percentage of steel used, and in the manner of resisting shear stresses.

At one-third the maximum load, a comparison was made between the location of neutral axis as computed from actual deformeter readings of strain, and the location as calculated from the straight line relation used in reinforced concrete beam design. These values of k, determined by strain and by calculation, were in fair agreement.

A further comparison was made between values of steel stress as obtained from deformeter readings and those calculated from the actual moment by use of the conventional formula. The steel stresses computed from strain are much less than those calculated from moment. This was attributed by Withey to the fact that there was considerable tension carried by portions of the brick masonry at the uncracked sections, whereas none is assumed to be taken by the brick in the stress computation based on moment.

A further check between the location of the neutral axis as calculated by deformeter readings, and the location of the axis as computed by use of the parabolic stress distribution formulas listed below, was made at ultimate loads. Again the agreement between the results is fairly close. Values of actual maximum bending moment were compared with the maximum resisting moments calculated from the parabolic formulas:

$$M_b = \frac{2}{3} jkf_b bd^2$$

$$M_s = pjf_s bd^2$$

This comparison was made on 19 beams having percentages of steel ranging from 0.57 to 2.31. Results of this comparison showed that the average value of M_b, calculated on the basis of brick compression, was 41.3% higher than the actual maximum moment while the average value of M_s, computed on the basis of steel tension, was 8.5% less than the actual maximum moment.

The percentage of steel present in the beam influenced to a large extent the degree of agreement between actual and computed values of moment. For a steel percentage of 0.62, the computed M_b was 79% higher than the actual moment, while for a steel percentage of 1.61, the computed M_b was only 8.4% higher than the actual moment. This large influence due to percentage of steel was not in evidence in the case of the comparison of the computed M_s and the actual maximum moment.

Using the data presented by Withey, and computing the maximum moments by means of the Plastic Theory of Design (10), results are obtained which are lower than the values of actual maximum moments by an average of 4.9%. These results would indicate that this method of computation is the more applicable in predicting ultimate moments for reinforced brick beams.

Results of Withey's tests show that with the percentages of longitudinal steel used and an adequate number of stirrups, it is possible to develop the full fiber strength of the beams. The data show that without stirrups it is possible to stress the longitudinal steel to its yield point prior to failure if some critical steel percentage is not exceeded. This value varies with the kind of brick used.

The bond stresses developed were found to yield an average value of 182 psi.

The shear strengths found in double shear tests proved to be a rough measure of the shear strength of beams having no stirrup reinforcement.

Some effort was made in these tests to predict deflections. The computed deflections were based on the formula, (11)

$$D_c = \frac{23}{1296} \frac{\alpha PL^3}{E_b bd^3}$$

where P is the total applied load and $\alpha = 12(1 + pn)/(1 + 4pn)$. The agreement between actual observed deflections and computed deflections in this series of tests was poor, but more recent tests indicate the validity of the formula.

Some of the conclusions of Withey's tests are as follows:

(a) The results show that it is possible to develop a high degree of flexural strength in reinforced brick beams.

(b) With proper design of stirrups and longitudinal reinforcement, coefficients of resistance, M/bd^2, in excess of 500 psi, and maximum shear stresses, v, in excess of 200 psi are obtained.

(c) Where web reinforcement is required, and no longitudinal rods are bent up, a stirrup spacing approximately one-half the effective depth of the beam is satisfactory.

(d) The tests indicate that the formulas used in the calculation of fiber, shear, and bond stresses, and deflections for reinforced concrete beams can, with proper constants, be used in like calculations for reinforced brick beams.

In 1932, a series of tests (12) on the performance characteristics of reinforced brick masonry slabs was conducted at the Virginia Polytechnic Institute. Results indicate that in general, slabs constructed from bricks of higher absorptive capacity exhibited superior performance to those with lower absorption, even though the individual brick strength of the latter is considerably greater than that of the former. The conclusion was therefore drawn that of the three physical properties usually determined in the prescribed brick test, the percentage of absorption assumed the greatest importance for reinforced brick masonry slab construction.

A series of 30 reinforced brick masonry slabs were constructed using soft, medium, and hard burned brick of both the shale and clay variety. These slabs were then successively loaded to destruction. Using the formulas based on the straight line relation in reinforced concrete, a comparison was made between calculated and experimental steel and compressive stresses, from a load of zero to the ultimate load. The curves of bending moment vs calculated and observed stresses are shown in Fig. 2.1.

It was noted that the large discrepancies between calculated and experimental stresses were on the side of safety, and that when similar slabs of reinforced concrete were tested, the same general variation between calculated and observed stresses resulted.

The results of the tests conducted by the Virginia Polytechnic Institute further showed that in practically every case during the testing of the 30 slabs, the deflection at failure was less than the allowable deflection, that the deflection performance of the slabs was very good, and that the instantaneous recovery performance of the slabs, even when well past the design load, was very favorable. These facts warranted the conclusion that all slabs tested during the investigation possessed ample stiffness until well past design load.

2.2.3 <u>Asbestos Cement Board</u>

Prior to the investigation conducted under the first part of this program, the available knowledge on the behavior of asbestos cement board was limited to the results of tests performed at the direction of the manufacturer. These tests were aimed principally at determining values for the modulus of rupture to be used in design procedures. Specimen panels were loaded with concentrated loads, either at midspan or at third points; average values of mod-

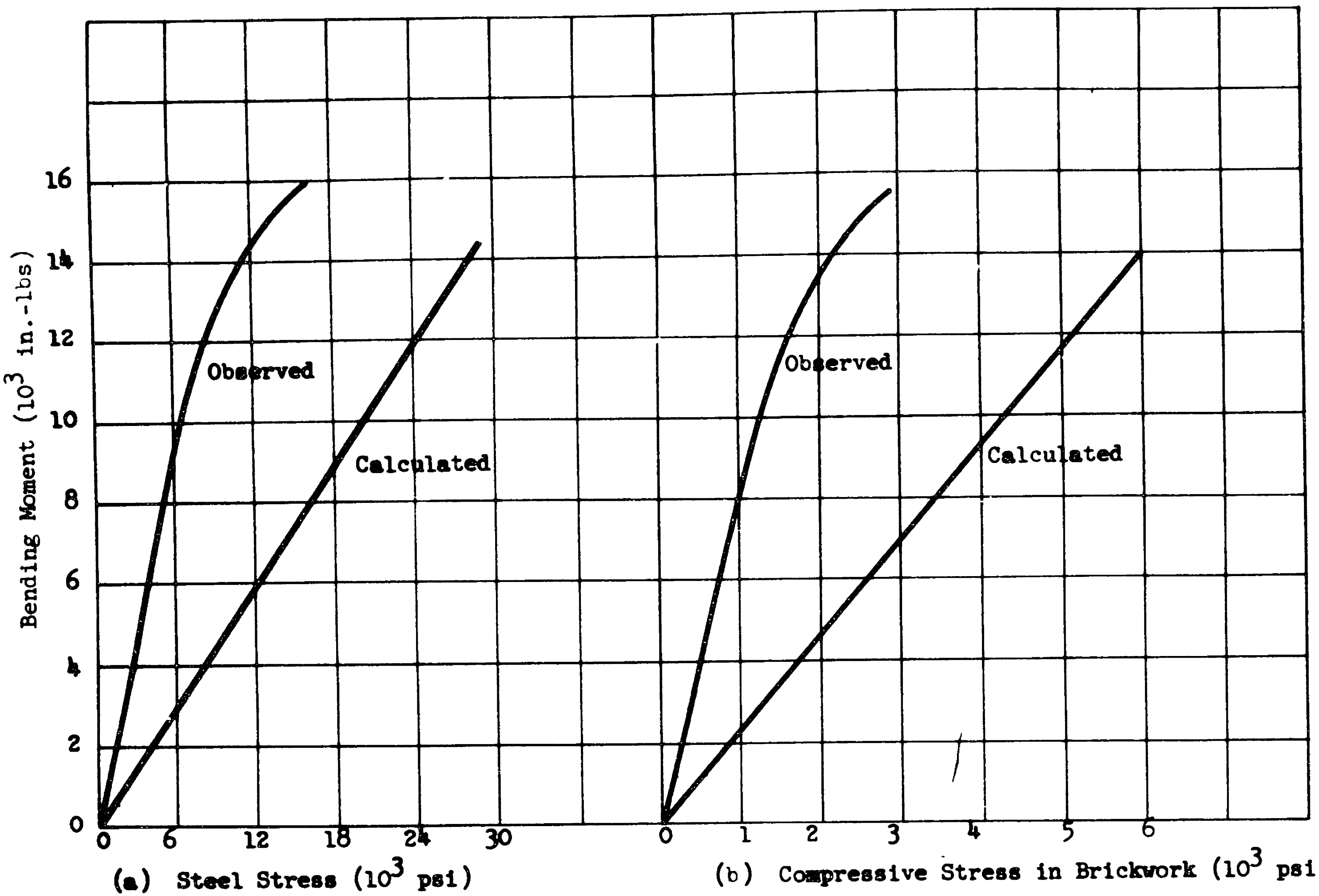


Figure 2.1 Relationships of Calculated and Observed Stresses due to Bending Moment.

uli of rupture thus determined are noted in Sec. 5.1 of the first report.

The tests carried out under the first report of this program yielded the following conclusions:

(a) Transite does not exhibit perfect elastic properties.

(b) Dynamic tests on Transite indicate that ultimate strains and deflections sustained (where the rate of straining is approximately 10^5 microin./in./sec) are 5 to 20%, and 3 to 6% greater than those sustained under static loads for flat and corrugated Transite respectively.

(c) Failure of flat Transite and corrugated Transite over continuous spans greater than 10 in. occurs by rupture in the vicinity of maximum bending moment.

(d) Corrugated and one in. flat Transite that have been saturated with moisture average respectively 81 and 75% as strong statically as dry specimens.

(e) Holes drilled for conventional attachments do not materially affect the strength of Transite.

(f) Damping in the corrugated and one in. flat Transite tested varied from 3 to 5%, and 3 to 4% of critical damping respectively.

See Chapter 5 of the first report for curves of ultimate load vs span length for simply-supported corrugated and flat Transite subjected to static and dynamic loads.

2.2.4 <u>Concrete Block Masonry</u>

Knowledge of the behavior of concrete

block masonry, other than the bearing capacity, comes from a series of experiments on the strength and stability of concrete block masonry walls conducted in 1932 by the University of Illinois (6), and an investigation into the structural properties of stone-concrete block masonry walls conducted in 1938 by the National Bureau of Standards (3).

The investigations at the University of Illinois showed that the modulus of rupture of the walls tested ranged from 18 to 50 psi. It was agreed that the flexural strength was a function of the adhesion of the mortar to the bearing surface of the masonry unit, since all transverse failures occurred at the junction of the two materials. The flexural strength was found to be approximately the same in a wall made with facial mortar bedding as in a similar wall with full mortar bedding. In these flexure tests, the walls were supported laterally by means of horizontal steel beams at the top and bottom edges, and load was applied uniformly along the horizontal center line of the 9 ft span.

It must be noted that these flexure tests were made with the walls supported in the weak direction, where the supports run in the direction of the block courses. The tests made on concrete block slabs included in this report were conducted with the walls supported in the strong direction, with the two supports running at right angles to the block courses.

Investigations into the structural properties of stone-concrete block masonry walls conducted by the

National Bureau of Standards showed that the walls, having a span of 7 ft 6 in., sustained an average ultimate equivalent uniform load (for equal moments) of 34.3 psi. These walls were composed of concrete blocks with dimensions of 7 11/16 in. x 7 13/16 in. x 11½ in. The walls were tested in a vertical position and were supported in the weak direction. The resulting average ultimate strength mentioned corresponds to a modulus of rupture of 22.6 psi.

Apparently no attempt has been made to load concrete block masonry panels with a uniformly distributed load, nor have any panels been tested under dynamic or impulsive loadings.

2.2.5 Partition Tile Walls

The available knowledge on the behavior of partition tile loaded in the transverse direction is very limited.

An investigation into the structural properties of structural clay tile was conducted in 1938 by the National Bureau of Standards (3). These tests were performed on walls having approximate dimensions of 4 ft x 8 ft x 8 in., and 8 ft x 8 ft x 8 in., and consisting of tile with approximate dimensions of 8 in. x 12 in. x 12 in., laid on end in one series of tests, and on side in the second series. The mortar mix used was 1:0.42:5.1 (cement: lime: sand, by volume). All walls had a span of 7 ft 6 in.

The walls with tile laid on end were found to have a modulus of rupture of 25.4 psi when loaded at the third points, while the walls with tile laid on side were found to have a modulus of rupture of 42.0 psi. In all cases, the walls were in a vertical position when tested, and were supported in the weak direction. Each of the specimens failed by rupture of either the mortar, or the bond between the tiles and the mortar.

Apparently no attempts have been made to load partition tile panels with a uniformly distributed load, nor have any panels been tested under dynamic or impulsive loadings.

2.2.6 Metal Siding

Numerous tests have been conducted on various types of metal siding, principally under the direction of the manufacturers and users. These tests, using various loading techniques, were conducted on both simply-supported and restrained end panels in order to evaluate properties such as mode of failure and effectiveness of fastening methods, and in order to verify load deflection curves. All indications point, however, to the fact that prior to the investigation conducted under the first part of this program, no tests to destruction were conducted on metal siding panels using uniformly distributed static or dynamic loading.

The tests carried out under the first portion of this program were conducted solely on simply-supported panels of various types of metal siding. (Standard corrugated aluminum and steel; Galbestos standard corrugated sheets; and Galbestos V-Beam), and yielded the following conclusions:

(a) All materials tested, with the exception of Galbestos 18 gauge standard corrugated sheets, failed by local buckling of the corrugations near midspan. Galbestos 18 gauge standard corrugated sheets failed by bending into a parabolic curve showing no indications of buckling.

(b) Dynamic tests on Galbestos 18 gauge V-Beam indicate that ultimate strains and deflections sustained (where the rate of straining is approximately 10^5 microin./in./sec) are 0 to 17% greater than those sustained under static loads.

(c) Damping in the Galbestos 18 gauge V-Beam panels tested was observed to be approximately 8% of critical damping.

Curves of yield or failure load versus span length, and load versus midspan deflection for various types of metal siding under static and dynamic loads are presented in Chapter 6 of the report of August 1952 (1).

CHAPTER 3

UNREINFORCED BRICK MASONRY WALL PANELS

3.1 GENERAL

Static and dynamic tests of nineteen 3 ft square plain brick masonry wall panels were reported in "Behavior of Wall Panels Under Static and Dynamic Loads", August 1952, (1). The ultimate loads for these 3 ft simply supported spans were extrapolated to longer spans, and curves of cracking load vs span were presented. Since then, static tests have been conducted on 6, 9, and 12 ft simply supported beams, and exceptionally good agreement was observed with the previously extrapolated curve.

The complete series of static and dynamic tests conducted over the past two years is reported below.

3.2 DESCRIPTION OF TESTS

A series of static and dynamic tests were conducted on simply supported walls and beams. The walls were tested with two simply support conditions, while the beams were tested under only one simple support condition but with various span lengths. This chapter deals only with tests of simply supported panels and beams; for the description and results of tests on beams with fixed ends, see Chapter 4 of this report.

A total of 18 simply supported plain brick masonry panels and 13 plain brick masonry beams were tested. All walls constructed for testing in the slab machine were 8 in. thick consisting of two wythes of brick, and measured 38 in. square. (See Fig. 3.1).

Beams were included in this program because they are simpler and more economical to test than walls. The data presented in the report of August 1952 demonstrated that results from beam tests may be satisfactorily applied to walls of similar span length. The brick beams tested were similar to the brick walls in that the same 8 in. wythe thickness was used.

Three types of beams were constructed. The first type was four courses ($10\frac{1}{4}$") wide, made up entirely of stretcher courses; the second was also four courses wide, but with a header course substituted for one of the interior stretcher courses; and the third type was 7 courses (18") wide, with 6 stretcher courses and one interior header course. A typical test beam is shown in Fig. 3.2.

Inspected workmanship was maintained throughout; all interior joints were completely filled with mortar. Members were tested 26 to 30 days after curing in a relatively dry room.

Two types of brick obtained locally were used in the tests; sand struck and water struck. Results of tests on the brick properties are shown in Table 3.1a.

The mortar used in these tests was a 1:1:5 mix (by volume) of portland cement, lime, and sand. Numerous samples of the mortar were tested, corresponding to each particular test member, for compressive strength, tensile strength in bending, and modulus of elasticity. All tests were made from 26 to 30 days after sampling. The results as shown in Table 3.1b varied considerably because the batching and mixing, as in typical

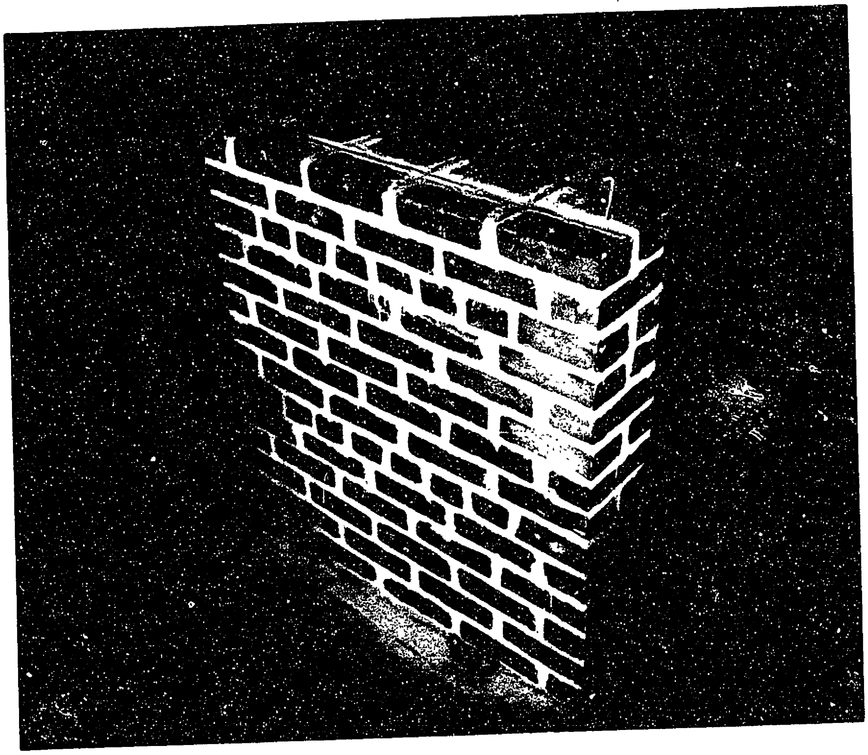


Figure 3.1 Brick Wall Panel

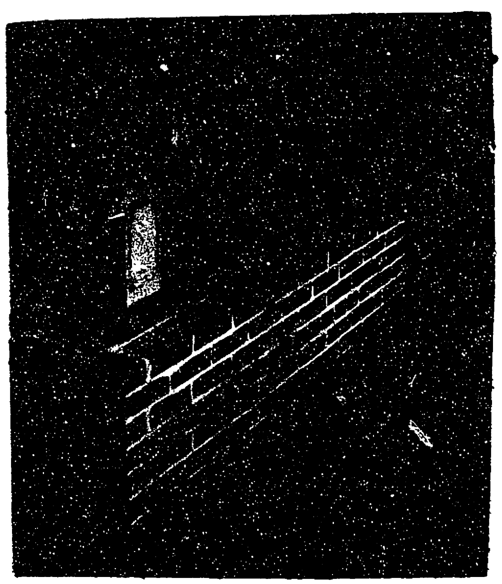


Figure 3.2 Brick Beam

construction practice, were not rigidly controlled.

In order to measure the adhesion of the mortar to the brick, commonly referred to as mortar bond strength, a series of tests, as described in the August 1952 report on page 9, were conducted on the bending rupture, and shearing strength properties of the mortar bond. Results of these tests are presented in Table 3.1c. Shearing strengths presented are values of the failure load, P, for the shear test, divided by the mortar area, A.

Initially, three types of bond patterns were used in the test walls: Flemish, English, and American or common bond. Flemish bond consists of alternate headers and stretchers in each course, while English bond consists of alternate courses of headers and stretchers. American (or common) bond consists of stretcher courses with a header course every sixth course. These bond patterns are illustrated in Fig. 3.3.

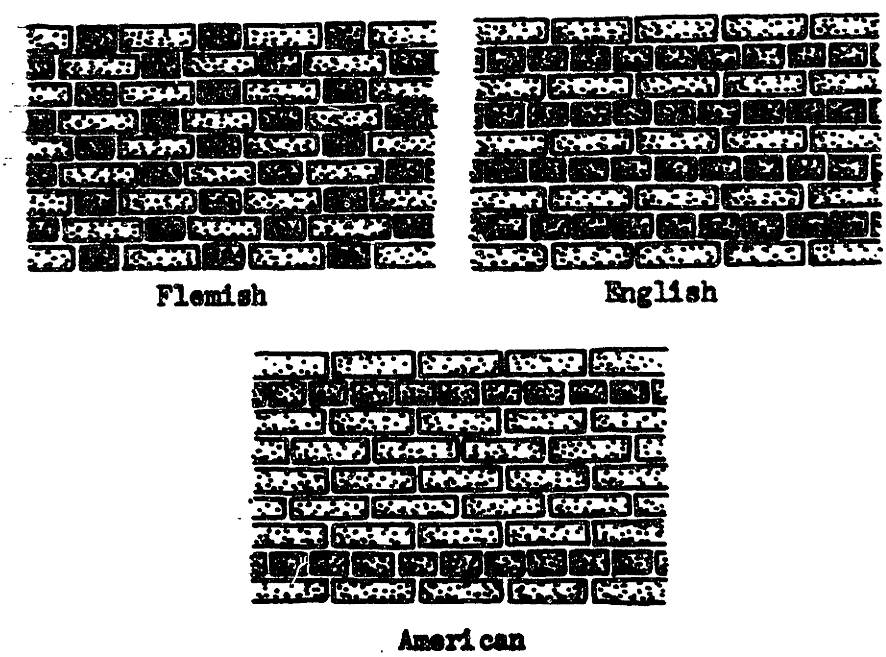


Figure 3.3 Bond Patterns

TABLE 3.1

Brick and Mortar Properties

a. Brick Properties

Type	Modulus of Rupture (psi)	Absorption (ASTM Specifications)
Sandstruck	578-905	-
Waterstruck	1160-1620	3.2% - 9.7%

b. Mortar Properties

Compressive Strength	555 - 1360	psi
Tensile Strength in Bending	123 - 345	psi
Modulus of Elasticity	0.41×10^6 - 2.83×10^6	psi

c. Mortar Bond Strength Properties

	Modulus of Rupture (psi)		Shearing Strength (P/A) (psi)	
	Range	Average	Range	Average
Series 1*	10-113	71	29-45	36
Series 2*	85-130	101	49-100	77
Series 3*	83-162	111	-	-

*The series numbers correspond to different lots of brick. Series 1 corresponds to the brick lot used for tests included in the report of August 1952, while Series 2 and 3 correspond to later lots used in tests for this report.

As the tests progressed, it became apparent that American bond had considerably more resistance to transverse loads than either Flemish or English, due to the fact that the failure plane in American bond must travel a longer distance in shear, resulting in a greater area over which shearing stresses act. Testing of the latter two bonds was then discontinued, and the remaining test program concentrated on walls of American bond.

The support conditions were varied for different static and dynamic tests. Three types of support were used: (1) At each support, the wall rested on a strip of solid rubber approximately 1 in. wide by $\frac{1}{2}$ in. thick, which, in turn, rested on a fixed half-round steel support, (see Fig. 3.4). (2) At each support, a bar, 2 in. wide by $\frac{1}{4}$ in. thick, was plastered to the bottom of the slab, and rested on a fixed half-round steel support, (see Fig. 3.5). (3) A partially fixed support. Tests using this latter type of support are covered in Chapter 4.

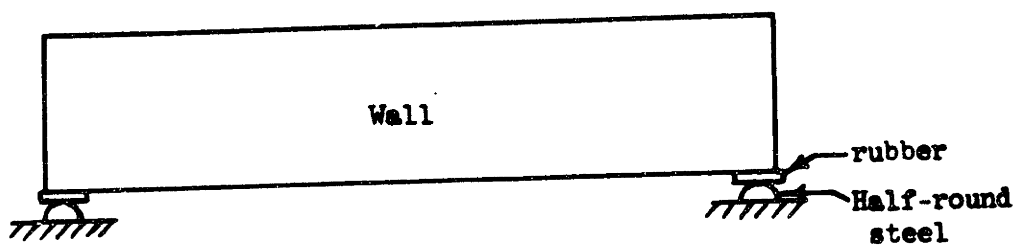


Figure 3.4 Support Condition (1)

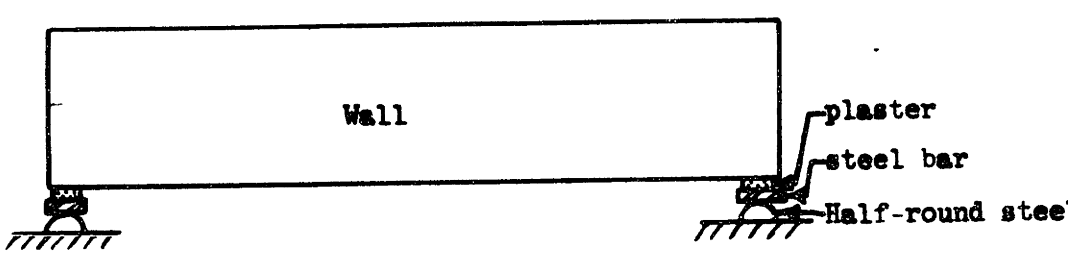


Figure 3.5 Support Condition (2)

The walls were tested in the slab machine with the wall in a horizontal plane. Continuous supports were provided along two opposite edges of the wall giving a span of 36 in.

between supports. The walls were loaded with a uniformly distributed pressure applied either statically, or impulsively with a full load build-up in 3 milli-sec and a load duration of approximately 10 sec.

The brick beams were loaded transversely at the third points of the span by transverse loading of beams plastered to the top face. The load was applied either statically or impulsively with a full load build-up of 1 milli-sec and a load duration of about 10 sec.

The following data were recorded for the various static and dynamic tests: load, midspan deflection, damage or mode of failure, and in a few cases, strains at selected points on the specimen.

In all instances, the effect of the dead weight of the specimen was included in load and stress calculations.

In order to help in the extrapolation of test results from small size slabs to larger span walls, a single static test was conducted on a wall slab of large dimension. This wall, constructed of plain waterstruck brick masonry in American bond and 1:1:5 mortar, was 2 wythes (8") thick, 25 courses (64") high, and had a span of 108 in. Support condition (2) was used, and load was applied at the third points by means of transverse loading beams plastered on the wall face. The wall was built and tested in a vertical position in a loading frame, (see Fig. 3.6). Data recorded were load, deflection at the top, middle, and bottom, and mode of failure.

Figure 3.6 Large Scale Brick Wall Loading Apparatus

An analysis of slab failure was conducted in the report of August 1952 which was based on certain assumptions. A limited number of tests were conducted in an attempt to verify these assumptions. A description of these tests and their results are presented in Appendix D.1.

3.3 TEST RESULTS

The results of static tests on brick walls are tabulated in Table 3.2. A study of Table 3.2 indicates, for the very limited number of tests conducted, that the walls constructed of Flemish and English bond are not as resistant to transverse loads as walls of American or common bond.

Table 3.2 indicates further that the support condition has only a small influence on the transverse strength of the walls. American bond walls of waterstruck brick with support condition (1) have an average modulus of rupture, based on 3 tests, of 211 psi, with a probable error of ±16 psi, while the average modulus of rupture of 3 similar walls tested with support condition (2) was 238 psi, with a probable error of ± 8 psi.

TABLE 3.2

Static Results on Brick Walls

(36" Span - Strong Direction - 1:1:5 Mortar)

Test No.	Description		Support **	Failure Load (psi) Unif.Dist.Press.	Computed Modulus of Rupture***(psi)
	Brick*	Bond			
1	S	American	(1)	9.8	149
2	S	American	(1)	8.6	131
3	S	English	(1)	7.5	114
4	W	Flemish	(1)	9.4	143
5	W	Flemish	(1)	8.2	125
6	W	American	(1)	11.1	169
7	W	American	(1)	14.2	214
8	W	American	(1)	16.4	250
9	W	American	(2)	14.1	214
10	W	American	(2)	16.5	251
11	W	American	(2)	16.4	250

*Type of brick: S = Sandstruck; W = Waterstruck

**Refer to Section 3.1 for description of support conditions

***For sample computations, see Appendix C

The results of dynamic tests on brick walls are presented in Table 3.3. A study of this table indicates that the influence of the support condition on dynamic tests is practically identical to the influence on static tests.

A tabulation of the test results on beams of varying span lengths is presented in Table 3.4. The uniformly distributed load listed for each beam and for the 9 ft span wall is the load which would cause the same midspan moment as the ultimate third point loading.

TABLE 3.3

Dynamic Results on Brick Walls***

(36" Span - Strong Direction - 1:1:5 Mortar)

Test No.	Support *	Failure Load (psi) Unif. Dist. Press.	Computed Modulus of Rupture (psi)**
D1	(1)	$10\frac{1}{2}$	260
D2	(1)	11	270
D3	(1)	$10\frac{1}{2}$	260
D4	(2)	13	320
D5	(2)	$13\frac{1}{2}$	330
D6	(2)	11	270
D7	(2)	$10\frac{1}{2}$	260

*Refer to Section 3.1 for description of supports

**Including dead weight and using a dynamic load factor of 1.6 (For sample computations, see Appendix C).

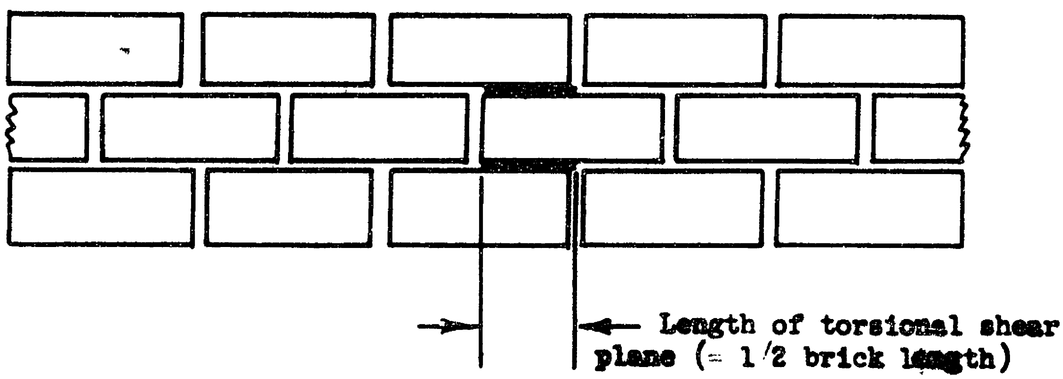
***All walls were constructed of waterstruck brick and American bond.

The results show that the modulus of rupture of a beam having all stretcher courses is approximately 25% higher, on an average, than a corresponding beam having an interior header course. This is explained by the fact that the failure plane must travel a longer distance in shear to pass from one stretcher course to another than it must travel to pass from a header to a stretcher course. (See Fig. 3.7)

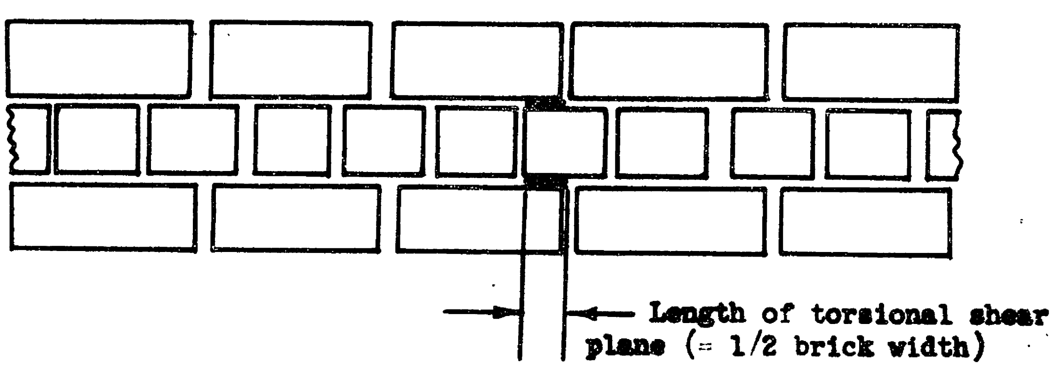
Deflection readings for the simply-supported beams were erratic and of such small and insignificant magnitude that their moment-deflection curves are not presented in this report.

Table 3.4 also includes the results of the test on the full-scale simply supported wall.

A graph of moment per unit width vs deflection at mid-



Running Bond (All stretcher courses)



American Bond (One interior header course)

Figure 3.7 A Comparison of Shear Plane Lengths for Running and American Bond

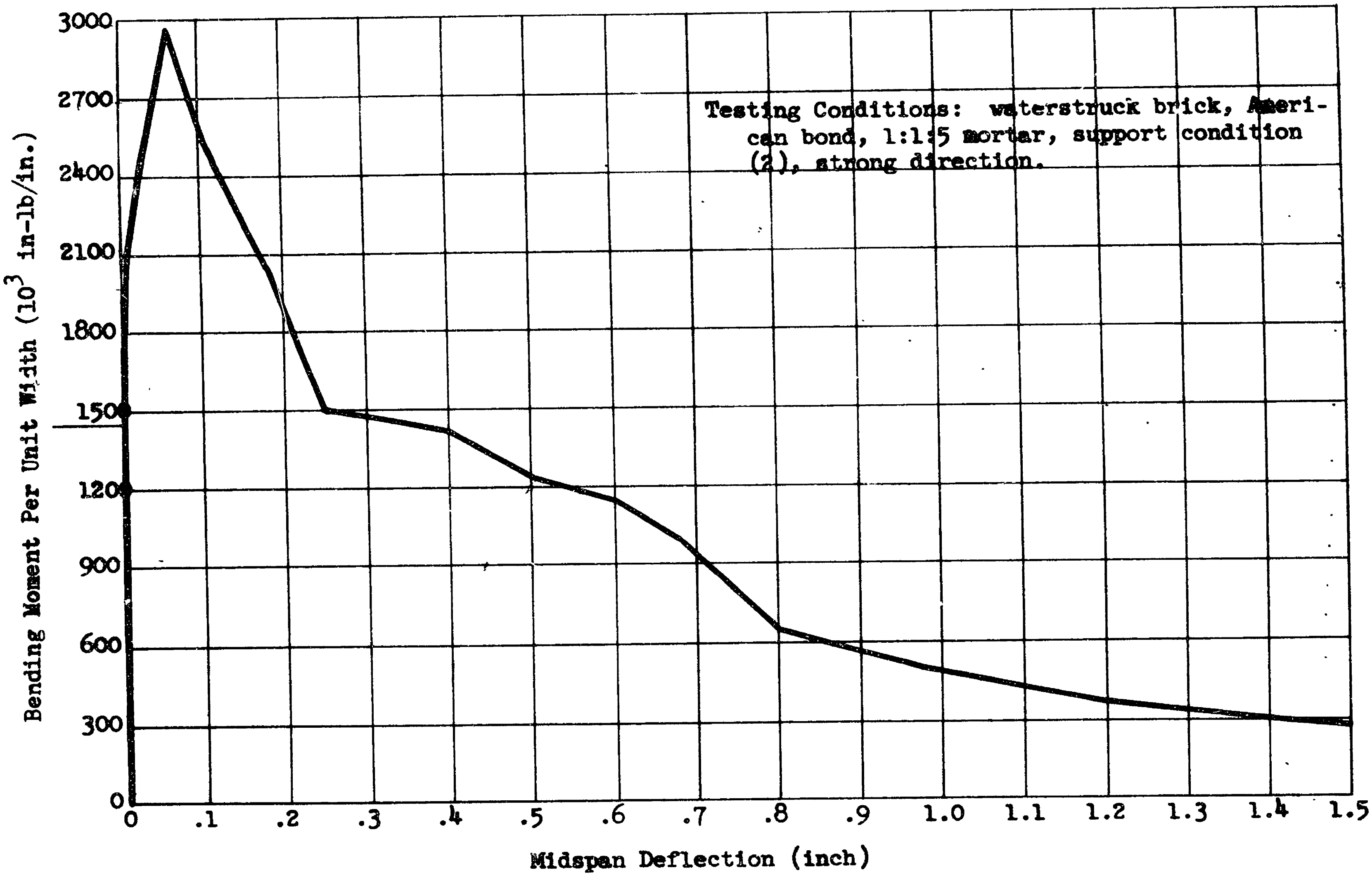


Figure 3.8 Bending Moment per Unit Width vs Midspan Deflection for 8 in. Plain Brick Masonry Wall under Static Third-Point Loading, Span = 108 in.

TABLE 3.4

Results of Beam Tests - Third Point Loading

(Strong Direction - 1:1:5 Mortar)

Beam No.	Span (in.)	Test *	Support	Width		Bond Type**	Mortar		Ultimate Load		Midspan Defl. at Ult. Ld (in.)****	Modulus of Rupture (psi) *****	Remarks
				Courses	In.		Avg. Flex. (psi)	Avg. Comp. (psi)	(kips) (LL only)	psi (incl DL) *****			
D1	36	D	(2)	4	10¼	Am	209	903	2.78	10.63		240***	
D2	36	D	(2)	4	10¼	Run	200	1030	3.54	13.38		300***	
D3	36	D	(2)	4	10¼	Run	-	-	3.72	14.02		320***	
D4	36	D	(2)	4	10¼	Run	251	807	3.54	13.38		300***	
3S1	36	S	(2)	4	10¼	Run	135	-	4.02	15.1		230	
3S2	36	S	(2)	4	10¼	Am	163	-	3.00	11.4		173	Low
6S1	72	S	(2)	4	10¼	Run	152	620	2.09	4.4	.039	268	
6S2	72	S	(2)	4	10¼	Run	147	555	2.35	4.8	.036	292	
6S3	72	S	(2)	7	18	Am	229	1135	3.20	3.9	.036	237	
6S4	72	S	(2)	7	18	Am	165	973	3.30	4.0	.041	244	
12S1	144	S	(2)	7	18	Am	277	783	0.76	1.0	.040	243	
12S2	144	S	(2)	7	18	Am	259	1140	0.83	1.0	.044	243	
12S3	144	S	(2)	7	18	Am	283	1013	0.55	0.90	.052	219	
LS1	108	S	(2)	25	64	Am	251	777	10.50	2.02	.050	277	

*S = Static, D = Dynamic

**Am = One interior course is a header course; Run = All courses are stretchers

***Using a dynamic load factor of 1.6

****Deflection values are from the dead load position

*****For sample computations, see Appendix C.

span is shown in Fig. 3.8 for the test on the large-scale 9 ft span wall.

Among the slabs tested both statically and dynamically, failure occurred primarily through the brick-mortar joints, with a slight amount of rupture across the brick. This failure took place at the midspan, in most cases, in a line roughly parallel to the supports.

Failure in the simply supported beams also occurred primarily through the joints at some location within the region of maximum moment. Failure was rapid and complete in the 3 ft and 6 ft beams. As the beam spans became larger, though, tests showed a tendency of the beams to undergo further deflection under slightly decreasing loads after the maximum load had been applied. This trend was in evidence in some of the 12 ft beams and especially in the full scale 9 ft span wall. In the case of the 9 ft wall, this phenomenon may be explained by the fact that the bottom of the wall experienced some resistance to transverse motion, and that the wall was in a vertical position when loaded.

In the 9 ft wall test, the first cracks appeared almost immediately before the maximum load was reached. After this maximum value was attained, the load resistance of the wall fell off steadily. The deflection at maximum load was about 0.05 in., and the deflection at the approximate total failure at which no further load was resisted was over 2 in. Cracks in the walls were consistent with those obtained in the beam and slab tests, since they appeared in the brick mortar joints in the region of maximum moment, and were approximately parallel to the supports. These cracks reached a maximum width of about 3/4 in. The wall during the test is shown in Figs. 3.9 and 3.10.

In none of the tests on simply-supported walls and beams were shear or bearing failure encountered.

3.4 DISCUSSION OF RESULTS

From the data presented, it can be seen that the static and dynamic behavior of simply-supported beams checks rather well with the behavior of wall slabs of similar span length, considering the limited number of tests conducted. Beams with header courses yielded test results which did not check as well. Three factors which might cause discrepancies between beam test and slab test results are: (a) The deflection resistance of a wall is greater than that of a beam due to Poisson's ratio acting to a greater extent in the wall under bending. (b) The full load rise time for the beam machine in a dynamic test is 1/3 that for the slab machine. (c) The method of loading differed for slabs and beams.

The computed moduli of rupture of the simply supported beams and slabs tested under statically applied loads appear quite consistent with each other. The modulus of rupture obtained by averaging the results of twelve tests is 247 psi with results ranging from 214 to 292 psi. This modulus of rupture corresponds to an ultimate static moment of

Figure 3.9 Large Scale Brick Wall at 0.45 in. Deflection

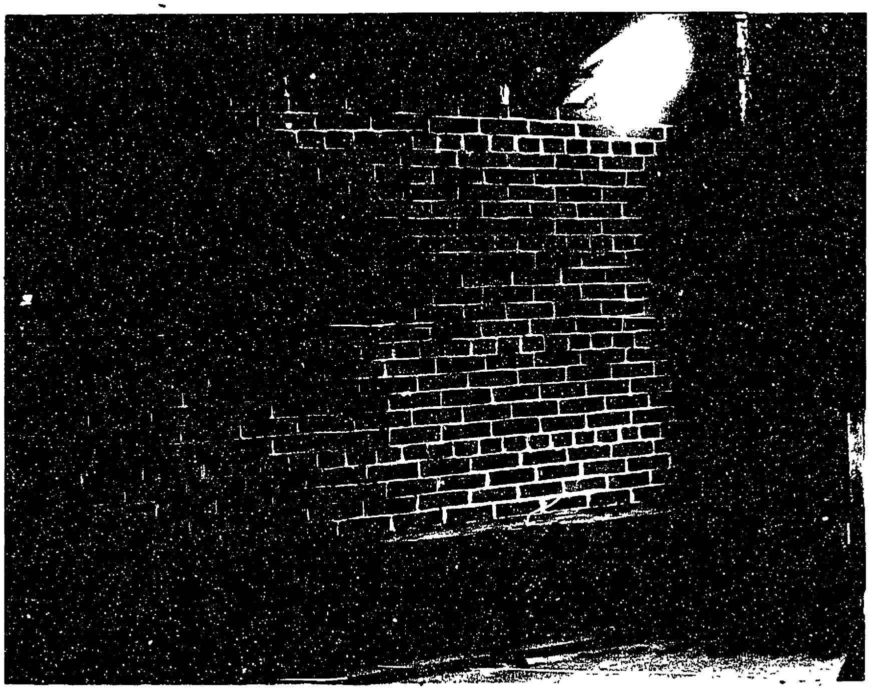


Figure 3.10 Large Scale Brick Wall After Test

2,640 in.-lb/in., for an 8 inch brick wall.

In the report of August 1952, plots of cracking load vs span lengths were presented for both fixed-end and simply-supported cases under static and dynamic loading. These curves are theoretical extrapolations based on the fact that cracking occurs when the wall sustains some critical moment, and that the moment varies as the uniformly distributed load and the square of the span. Hence, the uniformly distributed ultimate load varies inversely as the square of the span length.

In Fig. 3.11, a theoretical curve is presented of ultimate static load vs span length for support condition (2). In plotting this curve, the average observed ultimate load of the 3 ft slabs and beams tested was taken as the ultimate load for a span length of 36 in., and the curve was extended from that point using the relationship of load to span length mentioned in the preceding paragraph. Actual observed ultimate loads for beams of walls of various spans were also plotted. The curve resulting from these latter points assumed a shape similar to the theoretical curve, proving the validity of the relationship between ultimate load and span length. The actual observed load values were taken from tests on American bond walls and beams only. (For similar curves, but based on results on fixed-end spans, see Chapter 4).

The observed peak failure loads for the walls tested dynamically were found to be an average of 75 to 80% of the

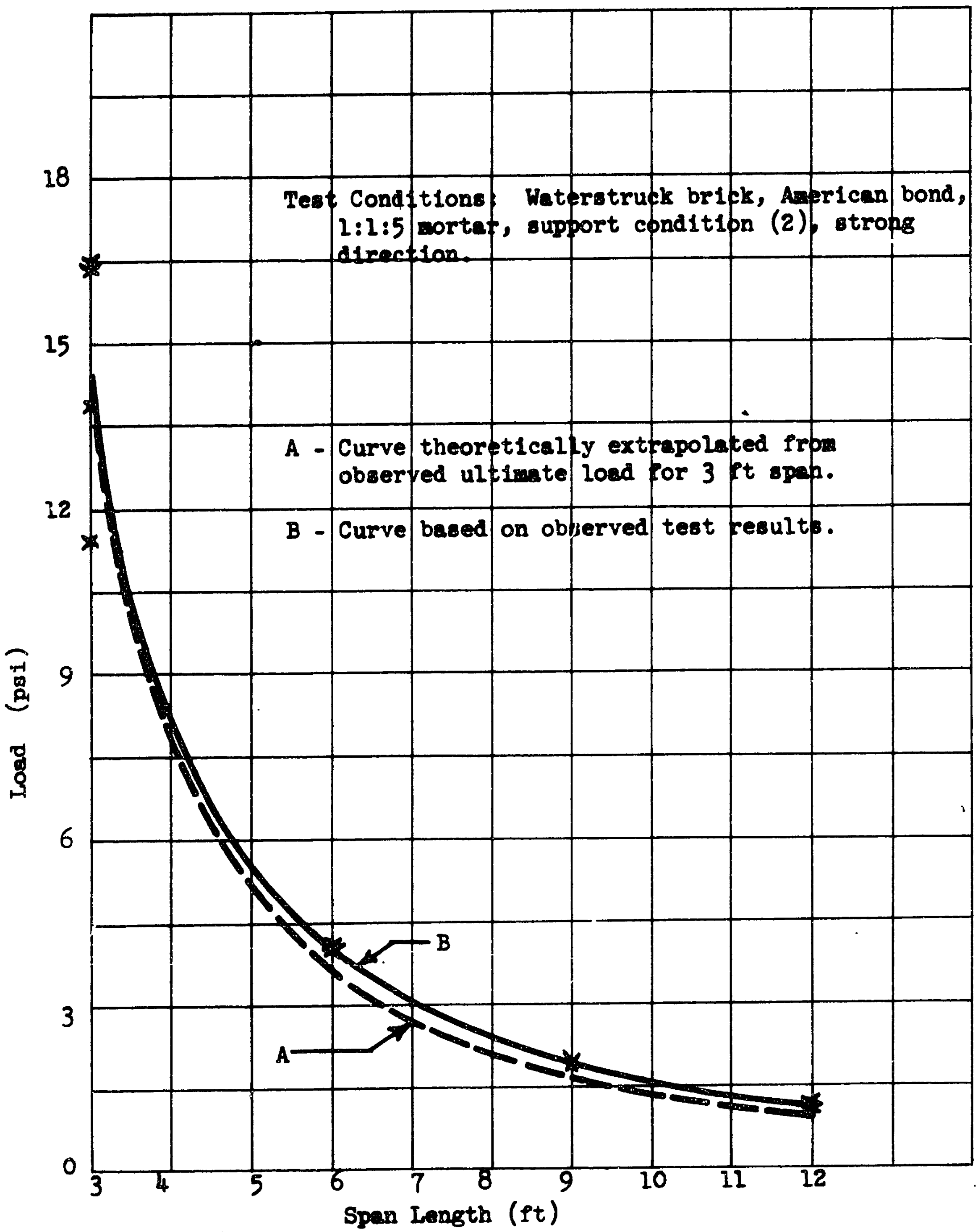


Figure 3.11 Load to Cause Failure vs Span Length for 8 in. Brick Wall under Static Load.

corresponding ultimate static loads. The true dynamic load factor, neglecting modes higher than the first, is approximately 1.6, and the corresponding strength factor is 1.20 to 1.28. (For computations, see Appendix C, Report of August 1952). The report of August 1952 presents a graph of load to cause initial cracking vs span length for 8 in. walls under dynamic loads.

3.5 CONCLUSIONS

Based on the limited number of tests conducted, it may be concluded that:

(a) The ultimate transversely applied uniformly distributed load on brick masonry walls and beams varies inversely as the square of the span length, all other conditions remaining constant.

(b) Failure of simply supported 8" thick slabs and beams under transverse loads occurs primarily through the brick mortar joints at approximately midspan due to bending moment, and no failure due to shear or bearing are encountered in spans of 3 ft to 12 ft long.

(c) Static and dynamic tests on brick beams indicate that the moduli of rupture agree fairly well with those for corresponding brick walls.

(d) Walls of American or common bond are stronger in resisting transverse loads than are walls of English or Flemish bond, depending upon the strength of brick and mortar used.

CHAPTER 4

COMMON VARIABLES IN MASONRY WALLS

4.1 GENERAL

In order to evaluate the effect on the strength and behavior of walls of some of the variables existing in brick masonry construction, a number of tests were conducted on brick slabs and beams in which these variables were altered. End support condition, direction in which the wall was supported, mortar strength, and superimposed dead weight on the wall, were the variables which were studied.

4.2 END SUPPORT CONDITION

The results of tests on simply-supported unreinforced brick masonry demonstrate that the bending strengths of such walls are extremely low. In actual construction, simply-supported panels will not be prevalent since degree of end restraint to the masonry wall would be normally present. This restraint would be provided by framing members or adjoining panels. Section 4.2 considers the strength of walls under conditions of as nearly full end restraint as could be achieved practically. Practical walls would exhibit strengths between those achieved in tests reported in this section and those achieved in tests reported in Chapter 3.

4.2.1 Description of Tests

A total of 17 beams were tested with fixed-end conditions. These beams were all 2 wythes (8") thick, and constructed of locally obtained waterstruck brick, using a mortar of 1:1:5 mix (by volume) of portland cement, lime, and sand.

Two types of beams were used. One type consisted of 4 stretcher courses and was $10\frac{1}{4}$ in. wide; the second type consisted of 6 stretcher courses and one interior header course, and had a width of 18 in. The former beam represented running bond while the latter was a reasonable approximation of American bond.

The support condition used in these tests was similar to condition (3) as described in the report of August, 1952. It consisted of a steel bar plastered to the bottom of each beam end and resting on a steel half-round support which, in turn, rested on a steel beam assembly. A rigid buttressed plate, placed approximately $\frac{1}{2}$ in. from the brick beam ends, was clamped to the steel assembly and made to bear against a bar welded to the assembly. The area between the brick beam and the buttressed plate was then filled with plaster of paris, and the whole assembly was placed in the loading machine (see Fig. 4.1). Third-point transverse loading was applied statically. An 8 ft beam with fixed-end supports is shown in Fig. 4.2 Data recorded during the tests were midspan deflection, ultimate load, mode of failure, and readings of end rotation to ascertain the amount of end fixity.

4.2.2 <u>Test Results</u>

The description of each beam and the test results are presented in Table 4.1. The uniformly distributed

TABLE 4.1

Results of Beam Tests - Third Point Static Loading

(Fixed End Condition, 1:1:5 Mortar - Support in Strong Direction)

Beam No.	Span (in.)	Width		Bond Type*	Mortar		Ultimate Load		Midspan Defl. at Ult. Load (in.)***	Remarks
		Courses	In.		Avg. Flex. (psi)	Avg. Comp. (psi)	(kips) (LL only)	(psi) (incl DL) **		
3-1	36	7	18	Am	201	1079	44.0	91.0	0.393	Reloaded
3-2	36	4	10.25	Run	262	1050	29.0	105.6	0.175	
3-3	36	4	10.25	Run	215	1050	25.0	90.8	0.208	
3-4	36	4	10.25	Run	178	855	30.0	108.9	0.159	
3-5	36	7	18	Am	147	682	35.7	74.2	0.298	Poor Mortar
6-1	72	4	10.25	Run	205	853	10.0	18.7	0.243	
6-2	72	4	10.25	Run	210	587	12.1	22.5	0.471	
6-3	72	4	10.25	Run	-	-	10.9	20.3	0.200	
6-4	72	7	18	Am	126	754	21.75	23.0	0.555	Shear thru headers
6-5	72	7	18	Am	127	682	21.0	22.2	0.718	Shear around headers
6-6	72	7	18	Am	249	860	21.0	22.2	1.175	Shear around headers
8-1	96	4	10.25	Run	159	648	7.25	10.4	0.624	
8-2	96	7	18	Am	197	881	15.50	12.6	0.868	Shear thru headers
8-3	96	4	10.25	Run	266	1162	8.26	11.8	0.738	
12-1	144	7	18	Am	213	796	4.76	3.05	1.325)	Bending moment failure,
12-2	144	7	18	Am	175	581	2.65	1.97	0.790)	no horizontal
12-3	144	7	18	Am	234	675	5.25	3.30	1.786)	separation.

*Am - One interior course is a header course.
Run- All courses are stretchers.

**For sample computations, see Appendix C.

***Deflection values are from the dead load position.

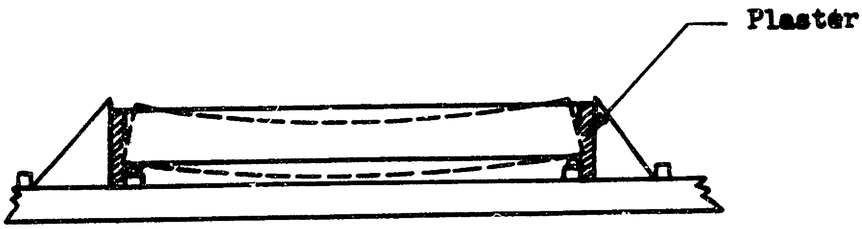


Figure 4.1 End Fixity Achieved in Beam Tests

(dotted line represents beam after tension cracks have opened in plaster.)

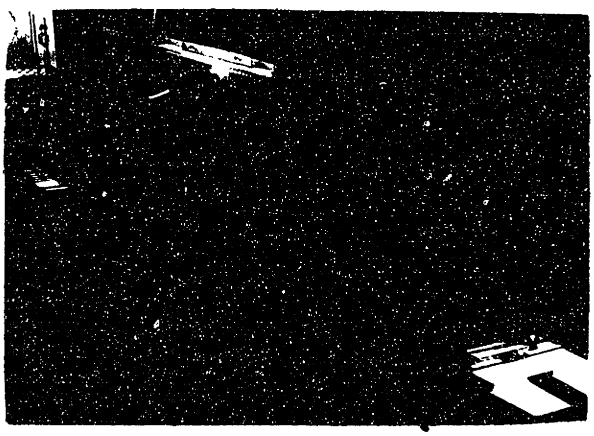


Figure 4.2 Fixed End Supported 8 ft Beam

ultimate load listed for each beam is the equivalent load value which would cause the same total moment, positive plus negative, as the ultimate moment observed in the beam test.

Curves of equivalent load, as described above, vs midspan deflection caused by third-point load application for span lengths of 3 ft, 6 ft, and 12 ft are presented in Fig. 4.3.

Since full fixity was not achieved, some rotation of the ends, combined with cracking and compression failure of the plaster of paris, was observed. (For description of the actual fixity achieved, see Sect. 4.2.3). Curves of end rotation vs applied moment per unit width for 3 ft, 6 ft, and 8 ft spans are presented in Appendix D.2 of Appendix D.

The mode of failure observed during tests on the 3 ft, 6 ft, and 8 ft beams was as follows: as the load on the beam was applied, vertical tension cracks appeared at the bottom of the beam between the bricks and mortar somewhere in the region of maximum moment. When these cracks had progressed to a point about three-fourths of the depth of the beam, a horizontal crack appeared extending out from the vertical cracks along the horizontal mortar joint which separates the two brick wythes. Both horizontal and vertical cracks continued to progress as more load was applied. As the ultimate load was reached, the horizontal crack extended almost instantaneously to one end of the beam, effectively reducing the system to two beams, each having one-half the height and one-eighth the moment of inertia of the original. Further downward deflection under load resulted in a slow and steady decrease in the load resistance of the beam (see Fig. 4.3). A release of the load resulted in the beam recovering part of its deflection. An inspection of the beam after it had reached its ultimate load revealed that the mortar joints near midspan on the top face of the beam and also on the top face of the bottom wythe had failed in compression, thus tending to confirm the belief that the system had failed as two beams. As the beam was deflected beyond the point of ultimate load, severe cracking in all brick mortar joints occurred. Some of the tests on the American bond beams showed the bricks in the header course to be ruptured as the horizontal separation reached them, while in other tests, the horizontal separation occurred in the brick mortar joint separating the two wythes but extended just to the interior header course bricks and then continued vertically up and down the courses, leaving the header bricks intact.

The mode of failure of the 12 ft span beams differed from that of the other beams tested in that no horizontal separations appeared during the loading; instead, the beam continued to act as one unit, and failure was due to compression in the top face mortar joints.

4.2.3 Discussion of Results

A comparison of the ultimate uniformly distri-

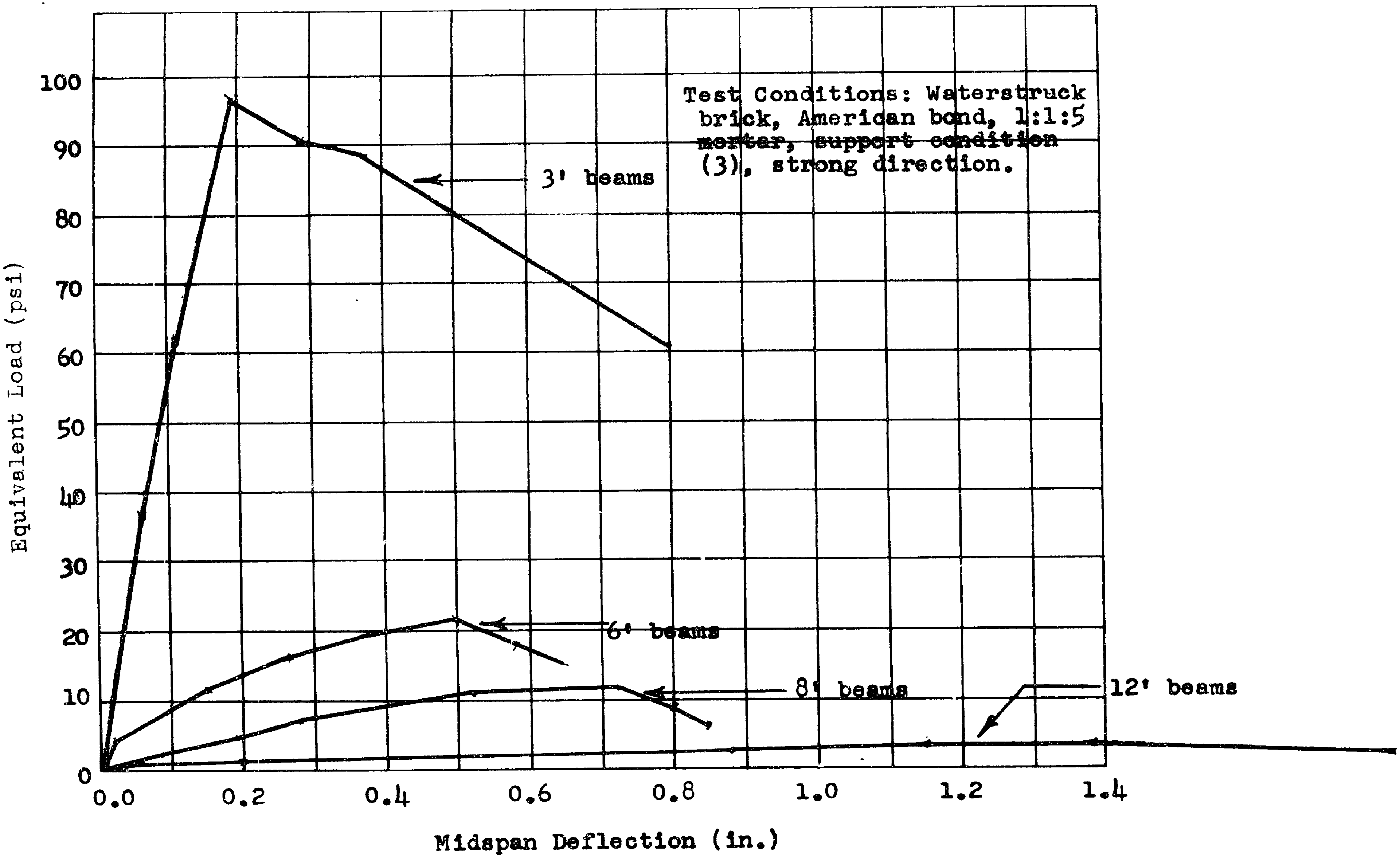


Figure 4.3 Equivalent Uniformly Distributed Load vs Average Midspan Deflection for 8 in. Plain Brick Masonry Beams.

buted loads resulting from tests on simply-supported and fixed-end supported beams is presented in Table 4.2. As expected, fixity increases the strength of a brick masonry member appreciably. This effect, however, decreases as span is increased.

TABLE 4.2

Comparison of Test Results on Simply-Supported and Fixed-End Supported Beams

Span (ft) *	Average Ultimate Load (psi)(incl DL)		Relative Strength Simply-Supported: Fixed-End
	Simply Supported	Fixed End	
3	14.6	82.6	1:5.6
6	4.0	22.4	1:5.6
12	1.0	2.77	1:2.8

*Only American bond beams considered.

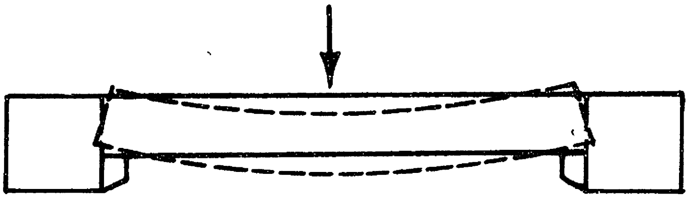
It must be noted that actual full fixity was not achieved in any of these tests because of the lack of tensile strength in the plaster, which led to a separation between the beam ends and the buttressed plate above the neutral axis. The plaster below the neutral axis provided for the transmission of the thrust or wedging force. (See Fig. 4.1)

The end support condition experienced by the beams tested in this series corresponds approximately to that which exists for a brick filler wall between heavy reinforced concrete or steel columns. The columns would resist the outward motion of the compression fibres of the wall but the inward motion of the tension fibres would be resisted only up to the tensile strength of the mortar joint (see Fig. 4.4). The support condition of the beams also corresponds, but to a lesser extent, to that of a brick wall continuous over a number of columns. The ends of each span are fully fixed until the tensile strength of the mortar joint is exceeded, at which point tensile cracks occur and the support condition changes from full fixity to one somewhere between fully fixed and simply supported (see Fig. 4.4).

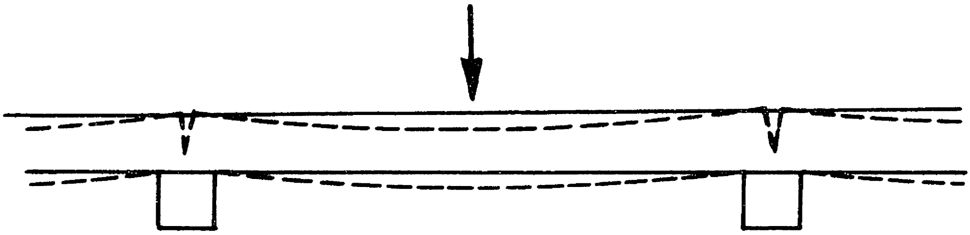
An explanation of the appearance of horizontal cracks between the brick wythes in the region of maximum moment where theoretically no shear exists, is presented as follows: as the beam is loaded and undergoes deflection, parts of the beam resist the curvature corresponding to this deflection. Once the beam has cracked at a up to b (see Fig. 4.5), the section, abcd, must bend according to the curvature of the beam. The only forces which can affect this bending of abcd are tensile forces along cb. It is reasonable to expect that as these tensile stresses exceed the ultimate tensile stress, local failure will occur, resulting in horizontal cracking along b toward c and eventually to the end of the beam, causing failure of the member.

Other investigators have encountered similar horizontal cracking phenomena in a region of zero shear in reinforced and prestressed concrete. However, this phenomenon is more likely to occur in unreinforced members.

In Chapter 3, results show that beams having only stretcher courses possess greater strength than beams



Brick Filler Wall



Continuous Brick Wall

Figure 4.4 End Fixity in Actual Brick Walls

(Dotted line represents wall after tension cracks have opened in mortar joints.)

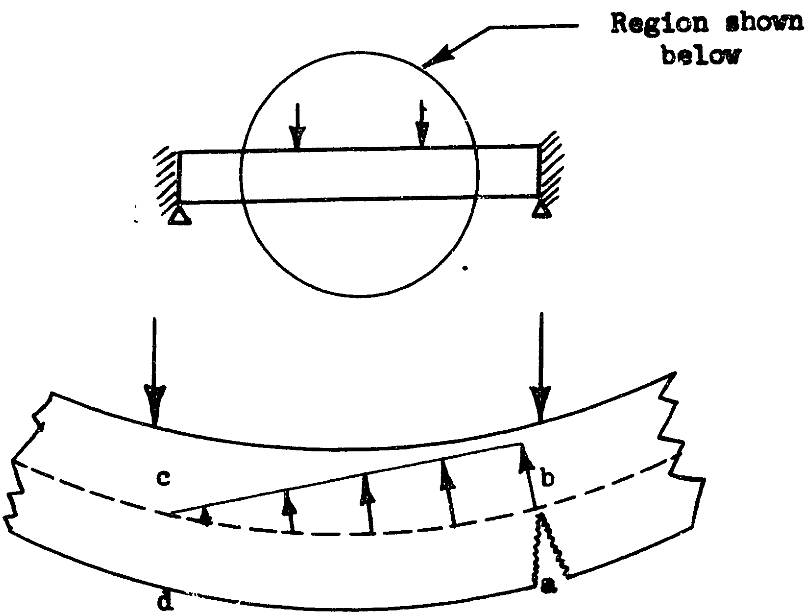


Figure 4.5 Loaded Beam Showing Forces Induced by Curvature

containing one interior header course. When beams with fully-fixed end supports were tested, however, the results indicated that, in most cases, the presence of a header course in the beam resulted in an increase in strength of the beam. This is due to the fact that the header course bricks, possessing a greater strength in tension than the mortar they replace, increase the beam's resistance to horizontal cracking, and thus increase the ultimate load. It would be logical to conclude that if more header courses were included in the construction of brick masonry walls, the wall's resistance to horizontal cracking might be increased to a point where this mode of failure would be eliminated, and the wall might then fail as a single beam at a higher ultimate load, instead of separating into two beams as observed.

Fig. 4.6 presents two curves of uniformly distributed ultimate load vs span length for an 8" brick wall with a fixed-end condition (support condition 3). The theoretical (dotted) curve of ultimate static load vs span is plotted by taking the average of the experimentally observed ultimate loads of the 3 ft fixed-end beams as the uniformly distributed ultimate load corresponding to a span length of 36 in., and then extending the curve according to the relationship stating that the ultimate load is inversely proportional to the square of the span length. Actual observed ultimate loads are also plotted and the resulting average curve is shown in the graph by the solid line. Since

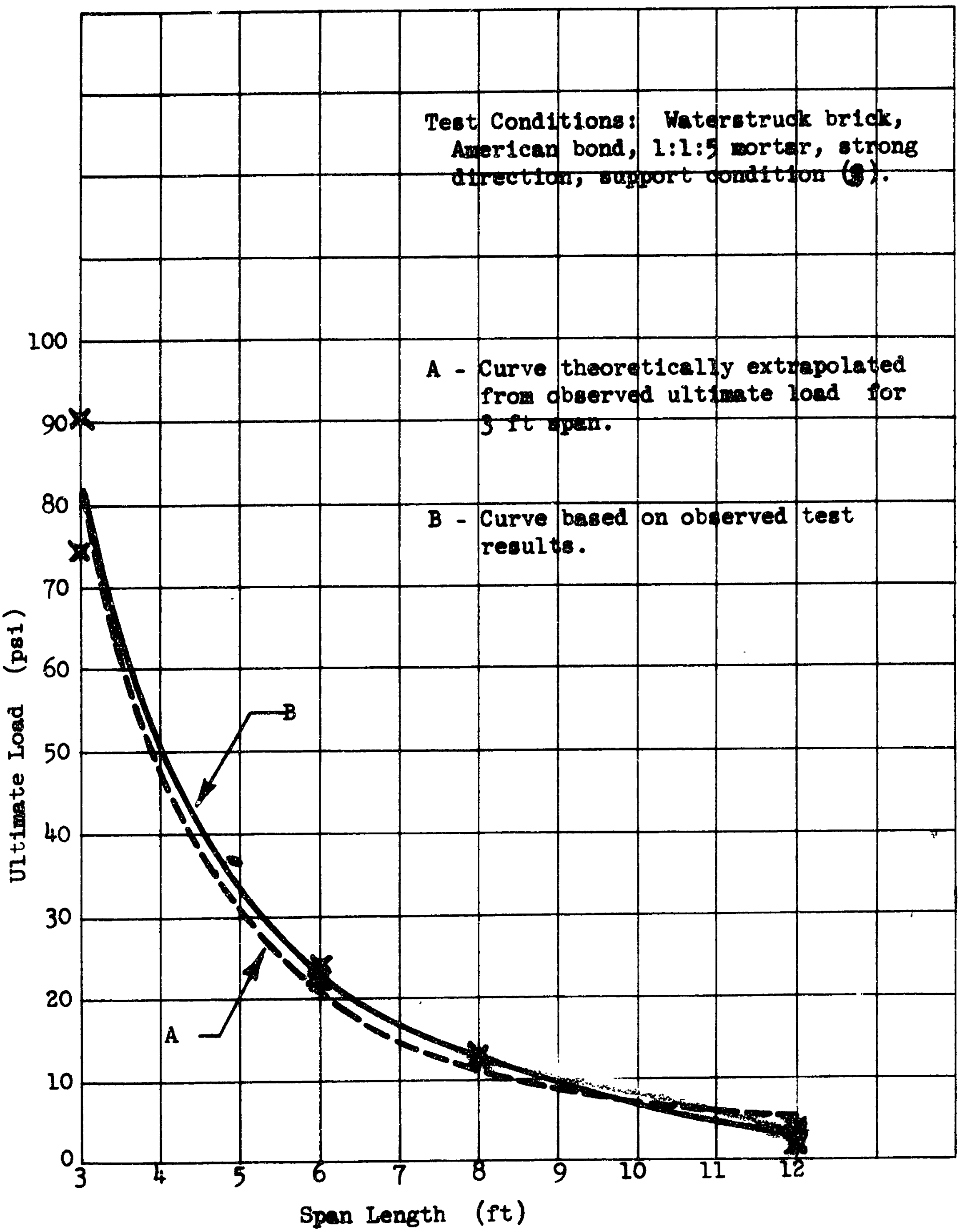


Figure 4.6 Load to Cause Failure vs Span Length for 8 in. Brick Wall under Static Load.

the curve based on the observed results assumed a shape very similar to the theoretical curve, it may be concluded that the ultimate uniformly distributed static load varies inversely as the square of the span length. Only American bond beams were included in the graph.

4.2.4 Summary

Based on the limited number of tests conducted, it may be concluded that:

(a) Fixed-end supports, as achieved in the tests described in this section, materially increase the resistance of a brick masonry wall to transverse bending.

(b) The ultimate transversely applied static load on brick masonry walls with fixed-end supports varies inversely with the square of the span length, other variables being held constant.

(c) Failure due to transverse bending in 3 ft, 6 ft, and 8 ft span walls is initiated by horizontal separation in the brick mortar joint between the wythes; this separation causes the member to act as two walls of one-eighth the original moment of inertia, resulting in total failure. In brick walls having span lengths of 12 ft or more, the member will continue to act as a unit during transverse loading, and failure will be due to compression rupture in the top face mortar joints.

(d) The presence of header courses, by resisting horizontal separation, increases the transverse strength of an 8" brick wall with fixed ends.

4.3 DIRECTION OF WALL SUPPORT

Brick walls may be supported in either of two directions or in both directions. When supported in one direction only, the wall may be supported so that it is strong or weak, depending on the direction of the courses of brick. The strong direction refers to a condition in which the two supports run at right angles to the brick courses, while the weak direction refers to a condition in which the supports run in the direction of the brick courses (see Fig. 4.7). All walls and beams tested outside of those covered in this section were loaded while supported in the strong direction.

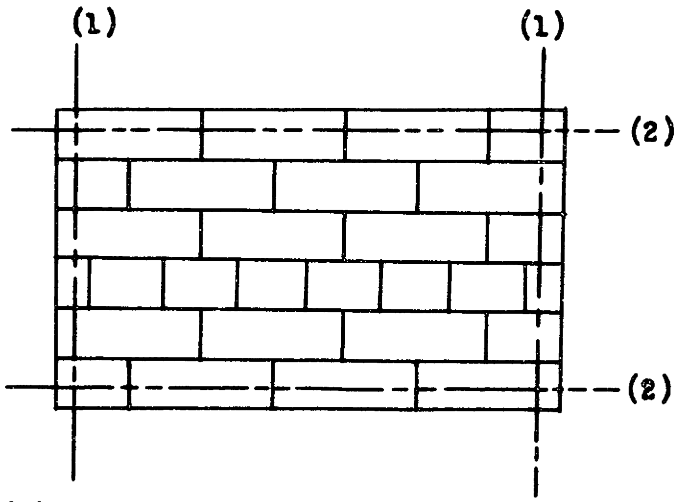
In actual masonry wall construction, support against transverse loads may be provided in either the strong or weak direction or both (two-way panel). Brick panels supported against vertical columns on their outer edges are supported in the strong direction, while brick panels supported along the top and bottom only by spandrel beams or grade beams are supported in the weak direction.

4.3.1 Description

Two identical walls of American bond waterstruck brick and 1:1:5 (by volume) mortar were tested statically in the weak direction, using a span length of 36 in. and support condition (2).

4.3.2 Results

Test No.	Failure Load (psi)	Modulus of Rupture (psi) **
W_1	1.7	26
W_2	0.6*	9



(1) Supports running in strong direction.

(2) Supports running in weak direction.

Figure 4.7 Support Conditions

*W_2 failed under dead weight only.

**For sample calculations, see Appendix C.

4.3.3 Discussion of Results

The results of this limited number of tests indicate that brick masonry walls are much less resistant to transverse loads when supported in the weak direction. This phenomenon can be explained on the basis of a flaw theory, which is described in detail in Sect. 4.4.3b of the August 1952 report.

Results of brick masonry slabs tested in the strong direction are presented in Chapter 3.

4.4 SUPERIMPOSED DEAD WEIGHT OF UPPER STORIES

The tests described in this section were conducted in an attempt to evaluate the effect of superimposed dead load on the resistance to transverse bending of brick masonry walls.

4.4.1 Description of Tests

Seven brick masonry walls were tested in this series. They were two wythes (8") thick and 38 in. square. The slabs were laid in American bond using waterstruck brick and 1:1:5 (by volume) mortar.

The dead weight effect was achieved by stressing high strength rods, 0.20" in diameter, which were placed unbonded between the wythes of brick at right angles to the brick courses. Each rod was provided with a plate at either end to provide the reaction, a stress gauge to measure the force in the rod, and bolts on the threaded ends of the rod to control the force. The complete details of the scheme to cause the dead weight effect are presented in Appendix D.3.

Three walls were prestressed to a load of 2210 lb/ft, and four walls to a load of 4420 lb/ft, the latter load value corresponding closely to the maximum dead load effect found in a typical two-story building. (For computations yielding this value, see Appendix C). The walls, supported in the strong direction on a span length of 36 in. using support condition (2), were tested in the slab machine with a static uniformly distributed load applied transversely.

Data recorded were ultimate load, midspan deflection, and mode of failure.

4.4.2 Test Results

Table 4.3 presents the description of each slab along with the ultimate uniformly distributed load sustained in each test.

A curve of midspan moment per unit width vs midspan deflection for the four slabs which were prestressed to a value of 4420 lb/ft is presented in Fig. 4.8.

The mode of failure of these slabs was similar to the mode resulting from tests on plain brick masonry walls (see Fig. 4.9). It was noted, however, that as the prestressing was increased, there was a tendency of the rupture plane to pass through the brick rather than to travel along the mortar joint; i.e., more bricks were rup-

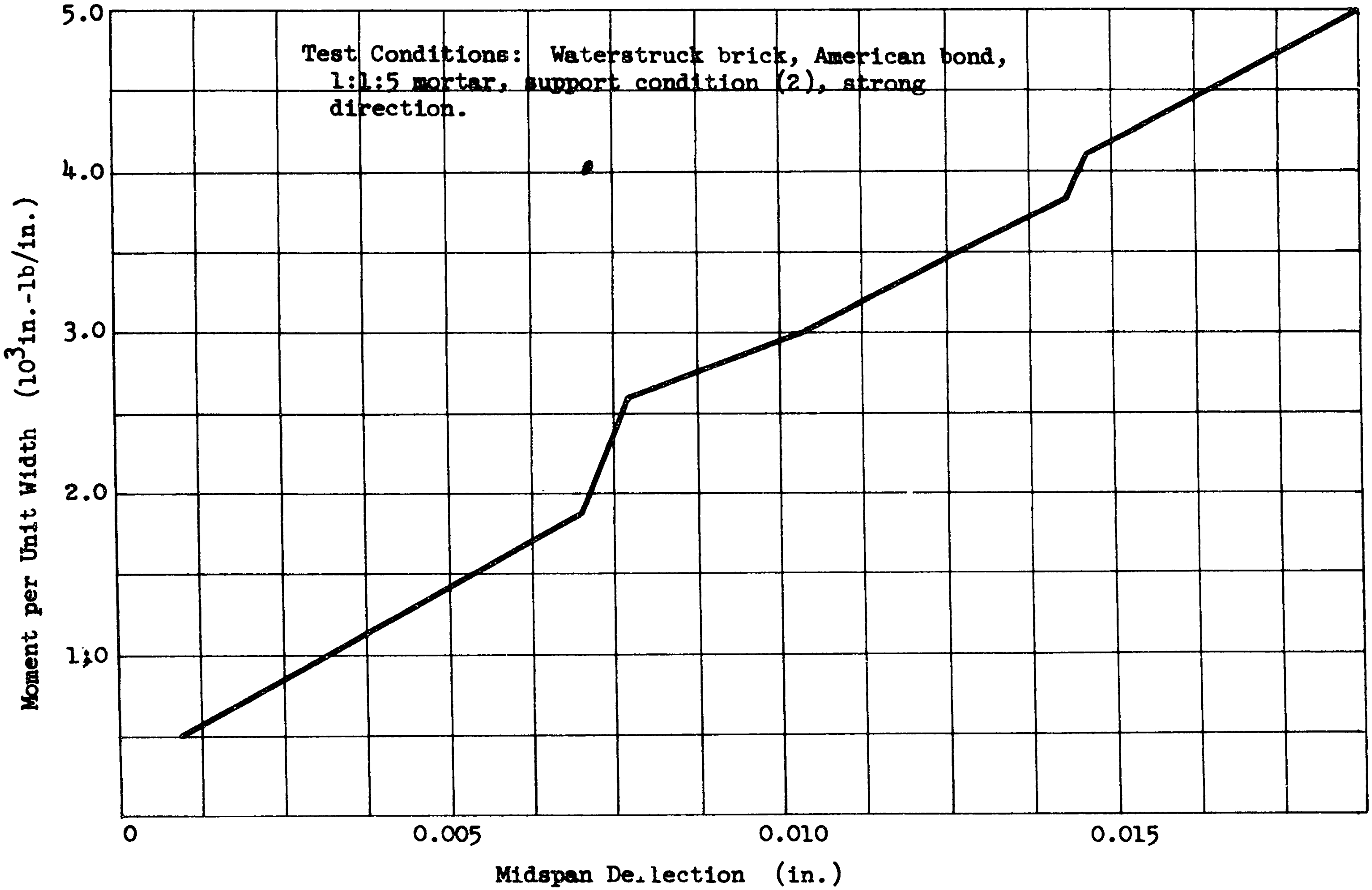


Figure 4.8 Bending Moment per Unit Width vs Midspan Deflection for Static Uniform Loading on 8 in. Plain Brick Masonry Wall of Span Length = 36 in., with Superimposed Dead Load of 4420 lbs/ft.

tured.

TABLE 4.3

Results of Tests on Wall Slabs Under Superimposed Dead Load (American Bond, 1:1:5 Mortar, Support Condition (2), Supported in Strong Direction)

Wall No.	Super-imposed Load (lb/ft)	Mortar		Ultimate psi (incl DL)	Midspan Defl. at Ult. Ld. (in.)*	Modulus of Rupture (psi)**
		Avg. Flex. (psi)	Avg. Comp. (psi)			
B-S-1	2210	127	810	20.8	0.025	316
B-S-2	2210	109	560	25.8	0.016	392
B-S-3	2210	138	968	27.4	0.032	417
B-S-4	4420	205	560	34.0	0.025	517
B-S-5	4420	193	545	27.0	0.011	410
B-S-6	4420	285	1184	25.5	--	388
B-S-7	4420	180	443	23.0	--	350

*Deflection values are from the dead load position.

**For sample computations, see Appendix C.

4.4.3 Discussion of Results

The results of these tests show that the effect of superimposed dead load increases the transverse strength of brick masonry walls. The average moduli of rupture of the walls prestressed 2210 lb/ft and 4420 lb/ft were 375 psi with a probable error of ±20.4 psi and 416 psi with a probable error of ±14.6 psi, respectively, while the average modulus of rupture of 8" brick walls with support condition (2) having no superimposed dead weight was 247 psi with a probable error of ±4.4 psi.

A graph of average modulus of rupture vs superimposed dead load is shown in Fig. 4.10. The graph shows that the rate of increase of transverse strength of

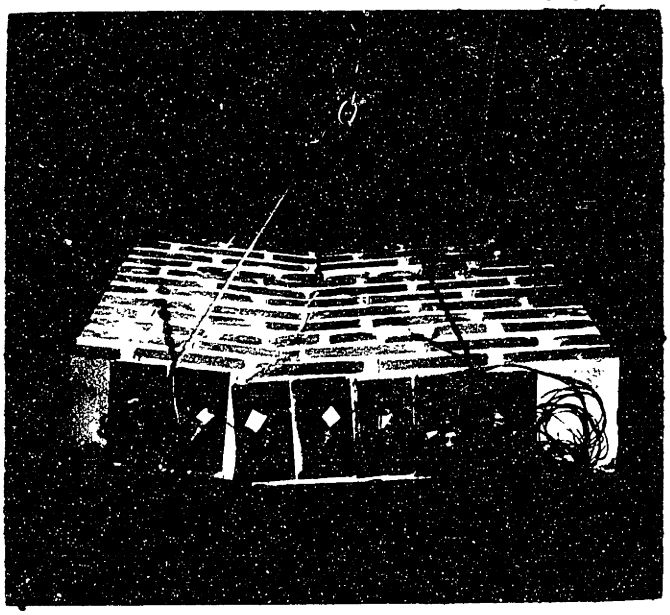
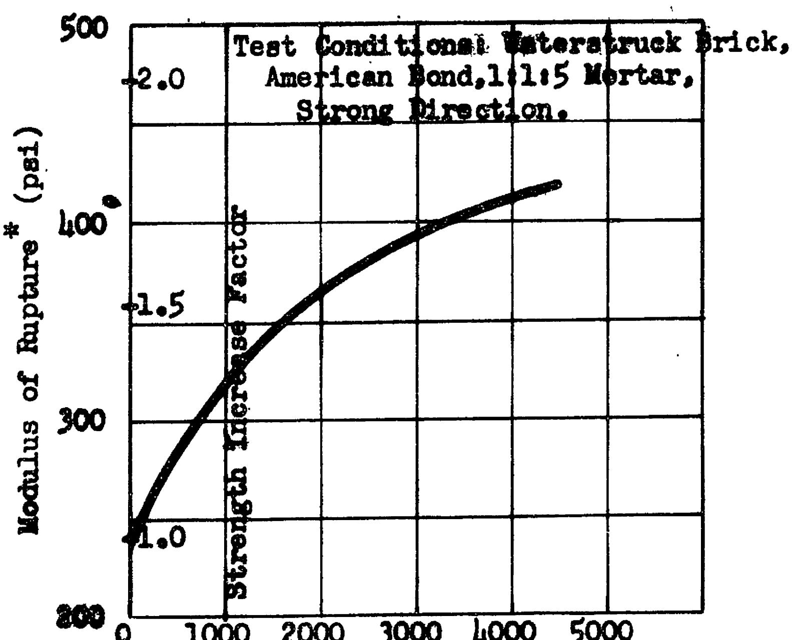


Figure 4.9 Panel BS-2 After Failure



Superimposed Dead Load (lb/ft)

Figure 4.10 Superimposed Dead Load vs Modulus of Rupture for 8" Brick Wall

*For sample computations, see Appendix C a brick wall due to superimposed dead load falls off as the dead load is increased.

Fig. 4.10 also shows the relationship of the superimposed dead load to the transverse strength increase factor for a brick wall of 8 in. thickness.

4.4.4 Conclusions

(a) Superimposed bearing load, applied longitudinally, increases the resistance to transverse bending of plain brick masonry walls.

(b) The increase in transverse strength, due to the superimposed load, falls off as the superimposed load is increased (see Fig. 4.10).

4.5 MORTAR STRENGTH

It seems clearly obvious that a higher strength mortar would result in a stronger brick wall panel. In order to determine the general trend of strength and behavior with stronger mortars, the following series of tests were conducted on wall panels constructed with a representative strong mortar, all other conditions remaining unchanged.

4.5.1 Description of Tests

A total of nine wall panels were tested in this series, three statically and six dynamically. All wall panels were constructed of waterstruck brick in American bond; dimensions were identical to those for walls of regular strength mortar (see Fig. 3.1), namely, 8 in. thick consisting of two wythes of brick and measuring 38 in. square.

All wall panels in this series were laid in a $1:\frac{1}{4}:3$ mortar mix (cement: lime: sand, by volume) in place of the regular 1:1:5 mortar mix previously used. This $1:\frac{1}{4}:3$ mortar corresponds to a type A mortar as specified by the Structural Clay Products Institute (13). This national authority recommends type A mortar as a high-strength mortar suitable for general use, and specifically for reinforced brick masonry and plain masonry below grade and in contact with earth. (The 1:1:5 mortar mix corresponds to a type B mortar, a medium strength mortar recommended for general use in exposed masonry above grade).

Compression and flexural tests were made on samples of this high-strength mortar as used in each wall tested. The average of 36 samples gave a compressive strength of 2,020 psi at 28 days. (The Structural Clay Products Institute specifies a 28 day compressive strength of 2,500 psi for type A mortar). Flexural tests on 36 mortar beams gave an average modulus of rupture of 450 psi.

All wall panels were tested to destruction in the slab testing machine loaded with a uniformly distributed pressure applied either statically, or impulsively with a full load buildup of 3 milli-secs. and a load duration of approximately 10 secs. All wall panels were supported in the strong direction using support condition (1), which corresponds closely to simply-supported edges.

Ultimate loads were recorded for each wall panel tested. In some dynamic tests, SR-4 strain gauges were placed across mortar joints at midspan to determine the fundamental period of vibration, the amount of damping, and the observed dynamic load factor. The behavior and mode of failure of all walls was noted.

4.5.2 Test Results

The ultimate loads for high-strength wall panels are tabulated below (dead weights are included):

TABLE 4.4

Test Results on High Strength Mortar Brick Wall Panels
(36 in. span - strong direction)

Test No.	Type	Mortar Avg. Flex. (psi)	Mortar Avg. Comp. (psi)	Ultimate Load	Computed Modulus of Rupture**
A-S-1	Static	422	2250	19.5 psi	296 psi
A-S-2	Static	375	1925	19.6	298
A-S-3	Static	403	1650	24.	364
Average of 3 static tests				21.0	318
A-D-1	Dynamic	617	1695	less than15 psi	
A-D-2	Dynamic	460	1848	less than13	
A-D-3	Dynamic	448	1830	less than12.5	
A-D-4	Dynamic	406	2175	11	284*
A-D-5	Dynamic	408	1784	11.5	299*
A-D-6	Dynamic	520	2890	13.5	349*
Average of 3 dynamic tests				12	310*

*Using a dynamic load factor of 1.7 (see Sect. 4.5.3).

**For sample computations, see Appendix C.

Tests A-D-1, A-D-2, and A-D-3 were not entirely successful in that the wall panels failed under the first dynamic load applied; therefore, it can only be said that the dynamic ultimate load for these three wall panels is somewhere between zero and the first dynamic load applied in each case.

A comparison of ultimate loads shows the the ultimate dynamic load is from 55% to 65% of the ultimate static load. A graphic comparison of static and dynamic ultimate loads and mortar strengths for this high-strength mortar and the previously used medium-strength (1:1:5) mortar is presented in Fig. 4.11. Note particularly that while the high-strength mortar results in mortar compressive and flexural strengths approximately twice those for medium strength mortar, the static ultimate loads for walls of the former are approximately only one and one-half times those for the latter. Note also that the dynamic ultimate loads for walls of high-strength mortar are approximately only one and one-quarter times those for walls of medium-strength mortar.

Failure in all instances occurred by rupture of the wall near midspan. As would be expected with the stronger mortar, the proportion of bricks ruptured was found to be greater (by about a factor of two) than with medium-strength mortar. Except for this feature, the mode of failure in all cases was identical to that encountered with walls of medium-strength mortar.

The fundamental period of vibration was determined, with the aid of SR-4 strain gauges, to be approximately 0.015 secs. The damping coefficient was observed, from the strain gauge records, to be approximately 7% of critical damping. The observed dynamic load factor varied from 1.5 to 1.7.

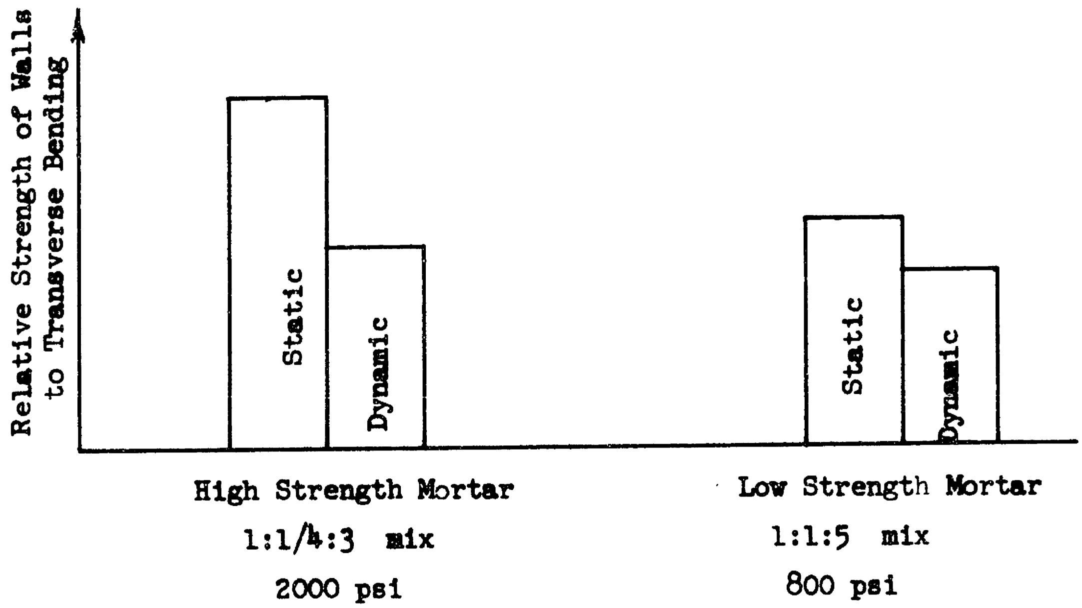


Figure 4.11 A Graphic Comparison of Static and Dynamic Transverse Ultimate Loads to Mortar Strength

4.5.3 <u>Discussion of Results</u>

The results of the static tests are in qualitative agreement with a series conducted by the Bureau of Standards (4); unfortunately, a precise comparison cannot be made because of other variables. Apparently no other investigations, either static or dynamic, on the effect of different mortar strengths have been conducted.

The fact that the wall strengths do not increase directly as the mortar strength may be due to the fact that the bond of the mortar to the brick probably does not increase directly as the mortar strength increases.

The relationship of this bond to wall failure, particularly its tensile and shear rupture, has been discussed previously.

The dynamic behavior of these walls could be theoretically predicted with the aid of a few calculations. The following data was observed:

Initial load buildup	0.003 sec
Fundamental period of vibration	0.015 sec
Damping coefficient	7%

By methods previously described (1), the true dynamic load factor may be computed.

$$D.L.F. = 1.70$$

This value is at the high end of the range of observed dynamic load factors.

If there is no strength increase under this dynamic rate of straining, the ultimate dynamic loads should be to the ultimate static loads as 1:1.70 or about 60%. Since the observed ratio was 55 to 65%, we may conclude that any increased ability to sustain strains and deflections under the dynamic test conditions imposed is practically negligible. Translated into terms of dynamic design, this means that while for medium-strength mortars the static allowable stresses may be increased by, say, 30%, the same for high-strength mortars may be increased by, say, less than 5%, if at all.

4.5.4 Conclusions

Based on this limited series of tests on high-strength mortar, and a comparison of the results with those from tests on medium-strength mortar, the following facts regarding the behavior of brick walls in resisting transverse loads may be concluded. The use of high-strength mortar rather than medium-strength mortar results in:

(a) Increased ultimate static loads,

(b) Increased natural period of vibration, damping, and consequently,

(c) Increased true dynamic load factor,

(d) Practically negligible increased ability to sustain strains and deflections under dynamic loading,

(e) Increased ultimate dynamic loads, but due to factors (c) and (d), this increase is proportionately less than the increase in the ultimate static load.

CHAPTER 5

REINFORCED BRICK MASONRY WALLS

5.1 GENERAL

Tests were conducted on reinforced brick masonry (RBM) panels and beams in order to evaluate the effectiveness of reinforcing steel in increasing the transverse bending strength of brick walls. Information was also desired concerning which design methods would be most applicable for RBM wall panels.

5.2 DESCRIPTION OF TESTS

Eighteen RBM panels and beams were tested in this series; two of these were tested dynamically, the remainder statically. All specimens were constructed of waterstruck brick laid in American bond; the reinforcing steel used consisted of intermediate grade bars with standard deformations, as specified in ASTM specification A 305. The majority of the specimens were laid in a 1:½:3 mortar mix (cement: lime: sand, by volume) which generally satisfies the requirements for RBM construction as specified by building codes of accepted authorities. A few specimens were laid in a 1:1:5 mortar mix. The method of laying up the test specimens is described and illustrated in Appendix D.4. Specimens of two thicknesses were tested, one wythe (4 in.) and two wythes (8 in.). Steel reinforcing ratios were varied from 0.0057 and 0.0082. The panels tested were 38 in. square; the beams tested were 18 in. wide and 6, 8, and 12 ft long. In addition, one full scale wall panel measuring 6 ft by 9 ft was tested.

The 38 inch panels were tested in the slab machine under a uniformly distributed load applied either statically or dynamically with full-load buildup in 3 millisec and a load duration of approximately 10 seconds. The beams were tested in a universal testing machine under a static third-point loading. The 6 ft by 9 ft wall was tested in a loading frame under a static third-point loading. Support condition (2)* was used on all specimens.

A detailed descriptive tabulation of the tests conducted is presented in Table 5.1.

The following data were recorded for each test: ultimate load, mode of failure, midspan deflections, and strain readings from SR-4 gauges on the reinforcing rods and mortar joints. Secondary test data included compressive and flexural strengths of the mortar and of the brick, modulus of elasticity and yield strengths of the reinforcing rods.

5.3 TEST RESULTS

The ultimate loads and other pertinent static test results for RBM panels and beams are presented in Table 5.2.

The ultimate load in psi listed for the specimens tested under third-point loading (all beams and the 6 ft by 9 ft wall) is the uniformly distributed load necessary to cause the same moment at midspan as the ultimate third-point loading. The ultimate load is also represented in

*See Section 3.2

TABLE 5.1

Description of Tests on Reinforced Slab and Beams

(Support Condition (2))

Test No.	Test Type*	Thickness (in.)	Span Length (in.)	d (in.)**	b (in.)**	p (%)**	Steel Arrangement	Mortar Mix	Support Dir.***
RS1	S	8	36	7	35	0.57	3/8" ea. course, 2¾" O.C.	1:1:5	S
RS2	S	8	36	7	35	0.57	ends alt. hooked & bent up	1:1:5	S
RS3	S	8	36	4	35	0.69	9-3/8" vert.jt., st. rods	1:¼:3	W
RS4	S	8	36	4	35	0.69	4" O.C.	1:¼:3	W
RS5	S	8	36	4	35	0.69		1:¼:3	W
R4S1	S	4	36	2.44	35	0.82		1:¼:3	S
R4S2	S	4	36	2.44	35	0.82		1:¼:3	S
R4S3	S	4	36	2.44	35	0.82	3/8" ea. 2nd course, 5.5"	1:¼:3	S
R4S4	S	4	36	2.44	35	0.82	O.C. (6 rods) Hooked ends	1:¼:3	S
R4D1	D	4	36	2.44	35	0.82		1:¼:3	S
R4D2	D	4	36	2.44	35	0.82		1:¼:3	S
6R1	S	8	69¾	7	18	0.57		1:¼:3	S
6R2	S	8	71	7	18	0.57	3/8" ea. course (6 rods)	1:¼:3	S
6R3	S	8	70½	7	18	0.57	hooked ends	1:¼:3	S
8R1	S	8	92	7	18	0.57		1:¼:3	S
12R1	S	8	143	7	18	0.57	3/8" ea. course (6 rods)	1:¼:3	S
12R2	S	8	144	7	18	0.57	st. rods	1:¼:3	S
LS2	S	8	108	7	60	0.57	3/8" ea. course, st. rods 4-3/8" vert.jt., st. rods	1:¼:3	S

*S = Static, D = Dynamic

**See Figure 5.8

***S = Strong, W = Weak

terms of ultimate moment at midspan in inch-kips per inch width of panel or beam. (Sample computations are presented in Appendix C)

A curve of midspan moment vs midspan deflection was plotted for each specimens tested. Each series was considered separately (38 in. panels, 6, 8, and 12 ft beams) and an average curve best representing all the curves in a particular series was drawn (see Figs. 5.1 and 5.2). On the 6 ft by 9 ft panel, three deflections were measured at midspan, one at the center, and one near each edge. Since the readings at the different locations showed very slight variance, the three were averaged and the average curve was plotted of midspan moment vs midspan deflection (see Fig. 5.3).

Two basic modes of failure were encountered as noted in Table 5.2, (a) bond slippage, and (b) bending moment failure.

Panels RS1 through RS5 were designed to develop high bond and shear stresses. Web reinforcing was provided in order to preclude shear failure. The mode of failure observed in these panels was sudden and complete and exclusively due to bond slip. (See Fig. 5.4)

Panels R4S1 through R4S4 underwent slow and gradual cracking and deflection under load until the horizontal mortar joints containing reinforcing steel separated, causing sudden and complete failure. (See Fig. 5.5). At failure, the reinforcing rods had been stressed to approximately 80% of their yield strength.

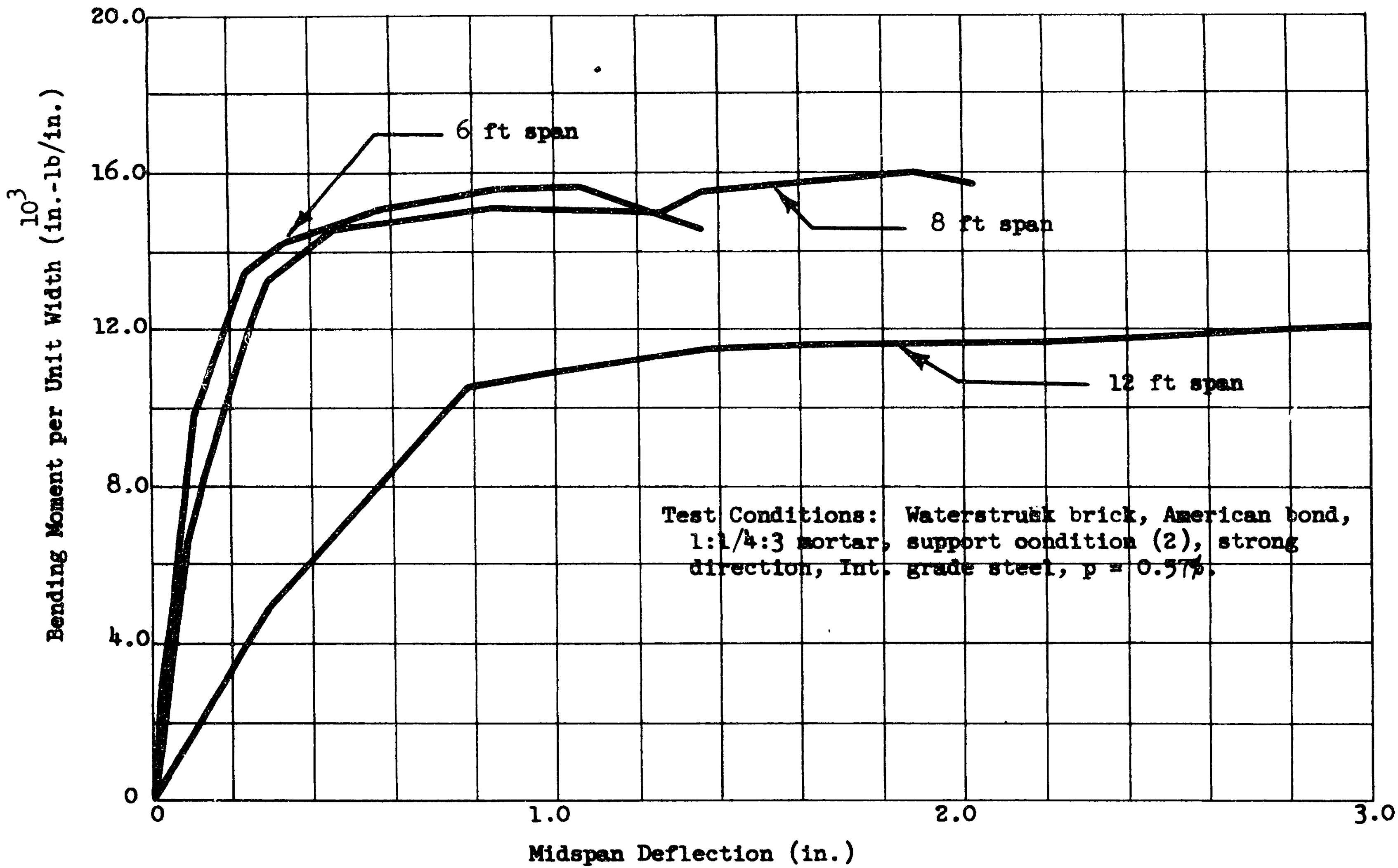


Figure 5.1 Bending Moment per Unit Width vs Midspan Deflection for Third-Point Loading on 8 in. Reinforced Brick Masonry Beams.

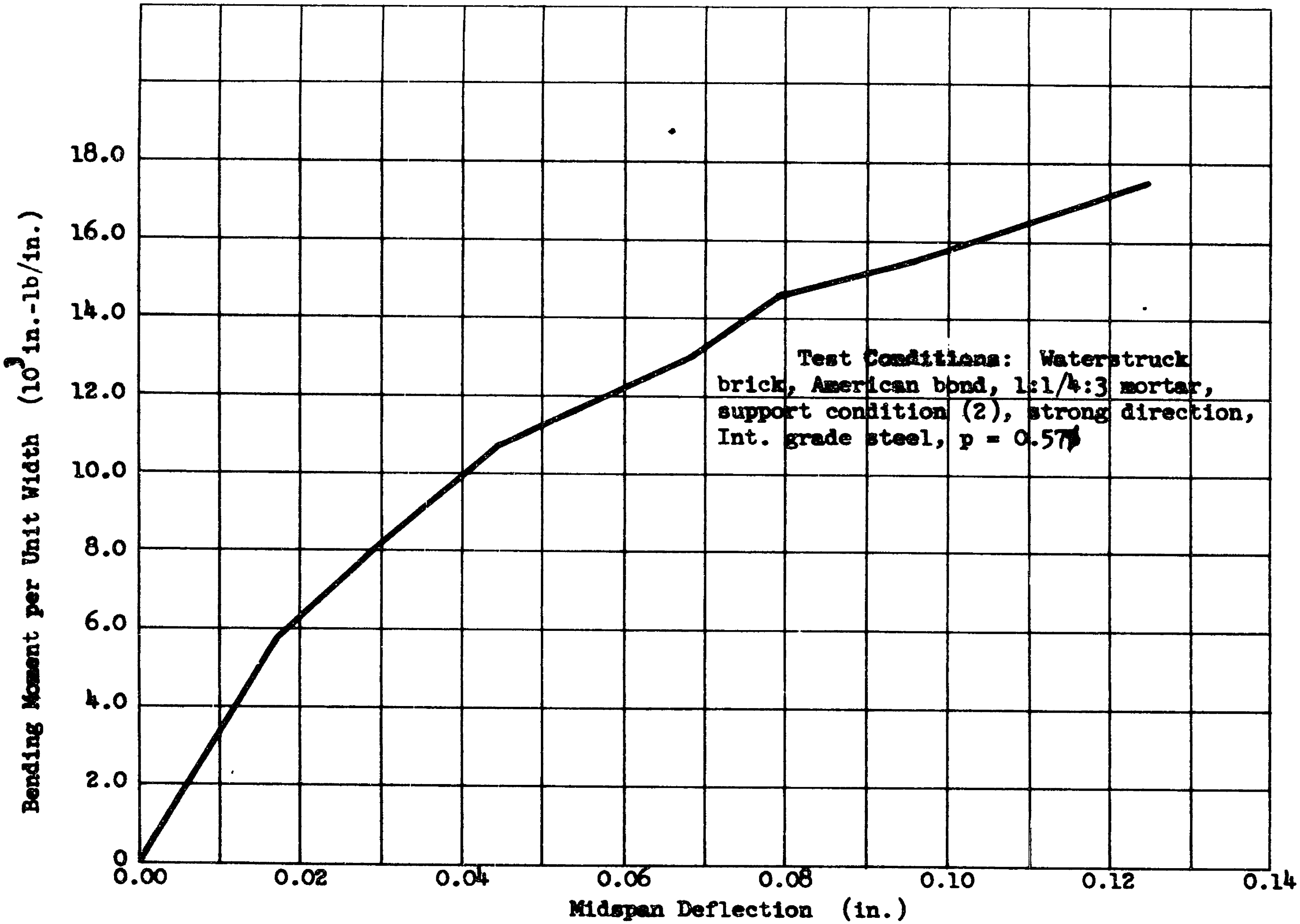


Figure 5.2 Bending Moment per Unit Width vs Midspan Deflection for Static Uniform Loading on 4 in. Reinforced Brick Masonry Walls of Span Length = 36 in.

TABLE 5.2

Test Results on Reinforced Slabs and Beams

Test No.	Steel Yield Stress (psi)	Ultimate Load (LL only) (kips)	Ultimate Load (incl. DL) (psi)	Ultimate Load Moment (in.k/in)	Type Failure	Deflection at Ult. Ld. (in.)*	Remarks
RS1	50,000		63.6	10.30	Bond	0.042	Sudden Failure
RS2	50,000		65.6	10.62	Bond	0.065	Sudden Failure
RS3	50,000		32.0	5.18	Bond	0.130	Failure over one support
RS4	50,000		27.6	4.38	Bond		
RS5	50,000		46.6	7.55	Bond		High
R4S1	50,000		11.8	1.91	(Shear Bond)	0.150	
R4S2	50,000		13.3	2.15	(Shear Bond)	0.130	
R4S3	50,000		10.8	1.75	(Shear Bond)	0.150	
R4S4	50,000		9.2	1.49	(Shear Bond)	0.115	
6R1	59,000	23.5	25.0	15.26	BM	0.397**	
6R2	59,000	22.5	23.9	15.07	BM	0.226**	
6R3	59,000	22.7	24.2	15.07	BM	0.331**	
8R1	59,000	17.4	14.3	15.12	BM	0.408**	
12R1	50,000	8.1	4.8	12.25	BM	0.871**	
12R2	50,000	8.0	4.7	12.11	BM	0.740**	
LS2	59,000	52.25	10.7	15.68	BM	0.563**	Reloaded

*Deflection values are from the dead load position

**Deflection values at plastic load.

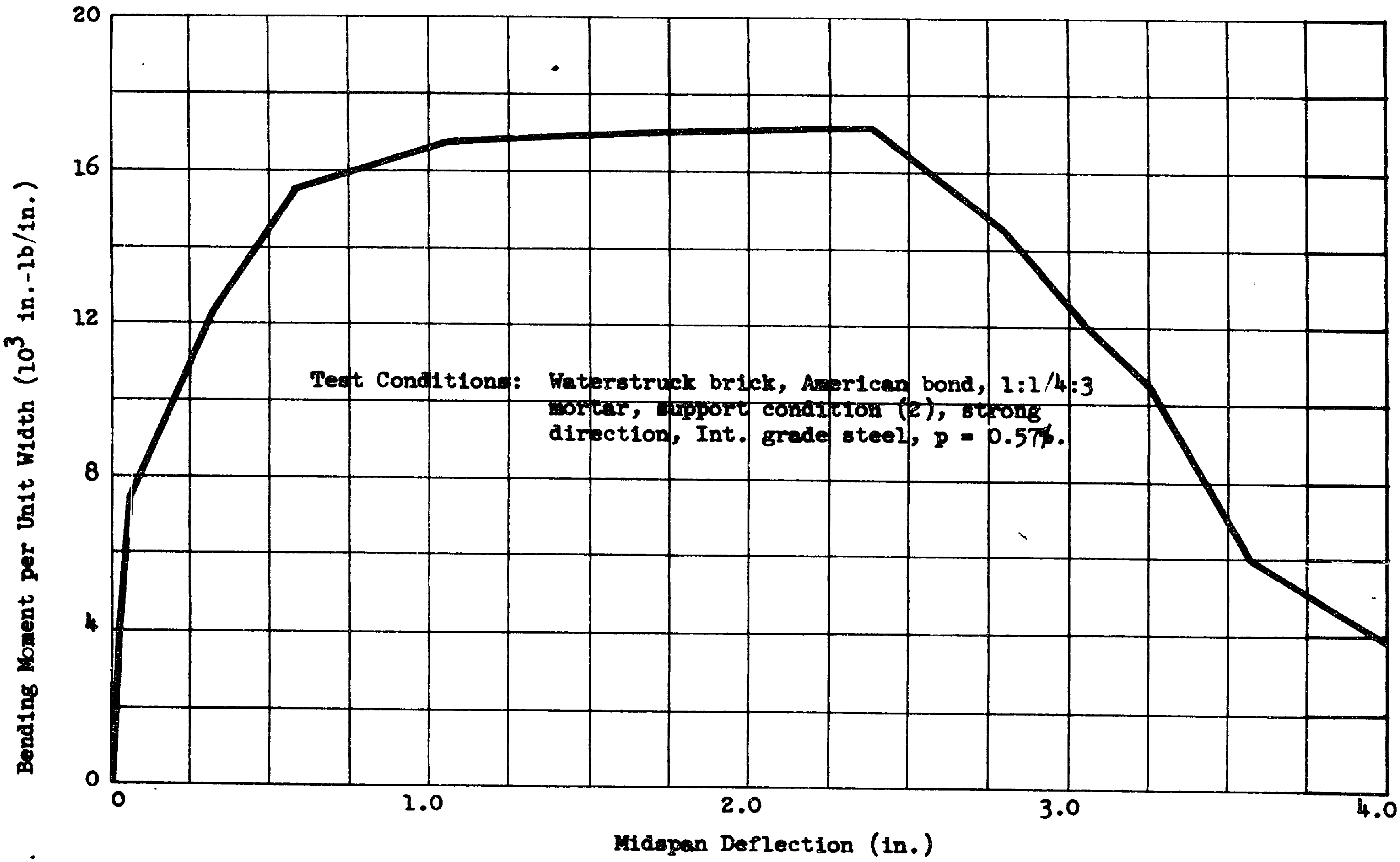


Figure 5.3 Bending Moment per Unit Width vs Midspan Deflection for Third Point Loading on 8 in. Reinforced Brick Masonry Wall of Span Length = 108 in.

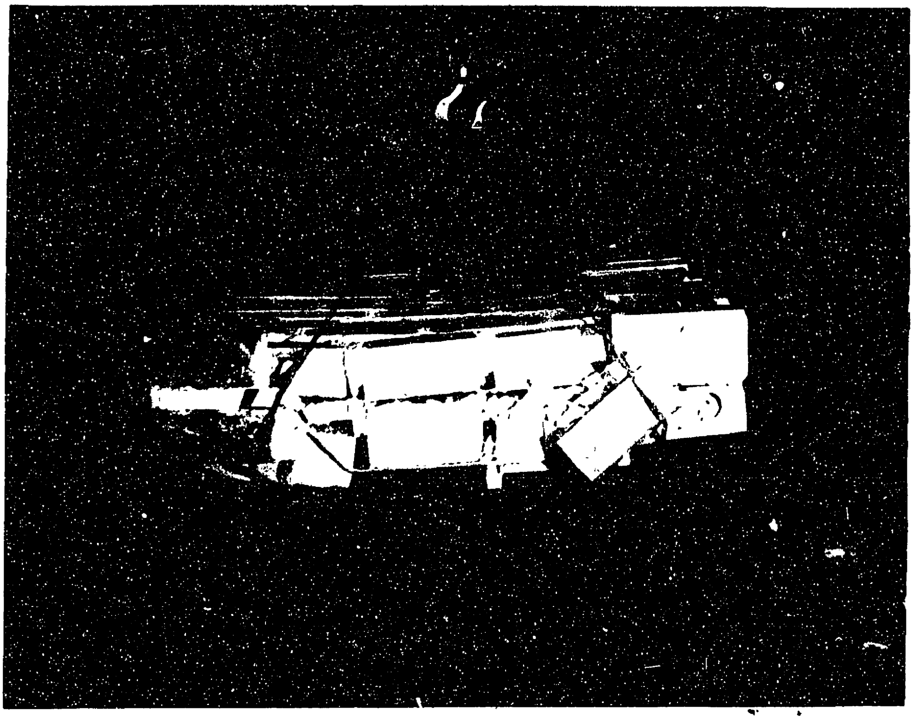


Figure 5.4 Panel RS-1 After Failure

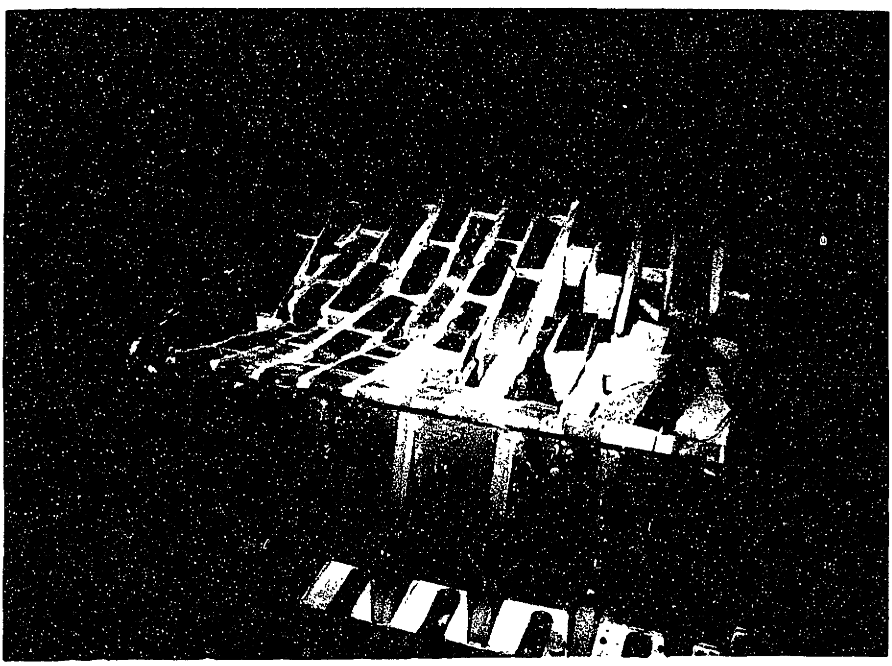


Figure 5.5 Panel R4S-1 After Failure

The remaining test specimens failed by tensile yielding of the reinforcing steel due to excessive bending-moment. This mode of failure is gradual and results in large energy absorptions. The failure is characterized first by cracking in the tensile mortar joints, a gradual opening up of these cracks as the steel is strained, and finally a slow plastic deflection as the steel passes its yield point. After yielding there is a slight and gradual increase in sustained load as strain hardening occurs in the reinforcing steel. Also a slight horizontal separation through the mortar joints between the brick courses occurs as the specimen continues to deflect in the plastic range. A typical failure is shown in Fig. 5.6.

The 6 ft by 9 ft panel, LS2, exhibited this same type of bending-moment failure. This panel, after failure, is shown in Fig. 5.7.

In order to determine the effect of excessive plastic deflection of RBM panels, several specimens were deflected far into the plastic range. Under these large deflections the reinforcing steel yielded plastically and cracks in all mortar joints, particularly near midspan, opened wider. Particular attention was paid to the deflection at which the brick units on the tension side became completely separated and could be removed by hand. This condition occurred at deflections of 1/36th to 1/44th of the span length for 8 in. thick panels on span lengths from 3 to 12 ft.

Two 4 in. thick panels 38 in. sq. were tested

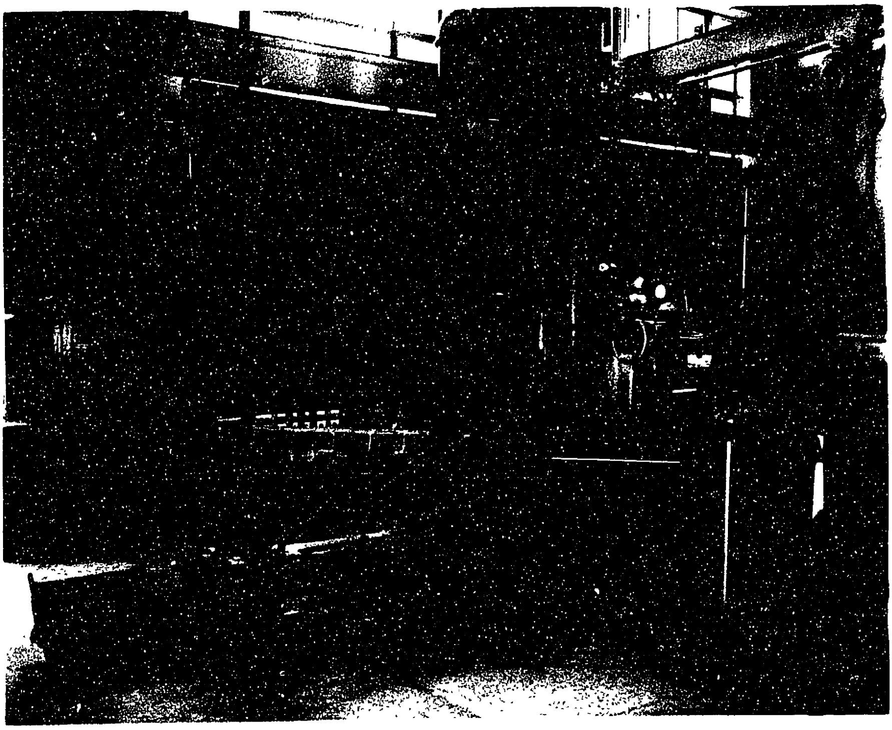


Figure 5.6 Beam 12R-1 After Failure

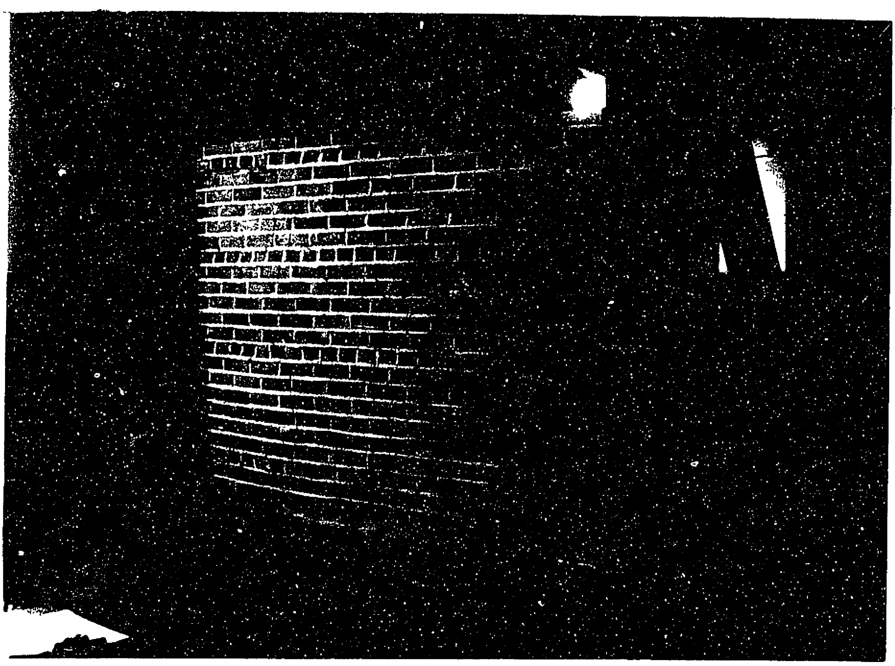


Figure 5.7 Large Scale RBM Wall After Failure

dynamically in the slab machine. The results are presented below:

R-4-D1

tested at 5 3/4 psi - no damage, no permanent deflection

retested at 7 psi - slight cracking on tension side

retested at 8 psi - complete failure over one third panel

R-4-D2

tested at 5 psi - no damage, no permanent deflection

retested at 6 psi - slight cracking on tension side

retested at 6½ psi - complete failure over ¼ panel

These dynamic tests are few in number and were intended only to give an inkling as to the dynamic behavior to be expected from RBM panels.

5.4 DISCUSSION OF RESULTS

The behavior of the RBM specimens during load tests, particularly at load magnitudes approaching the ultimate, and also for all loads in the plastic range, serve to demonstrate the similarity of RBM to reinforced concrete. Naturally, the different materials have different yield and ultimate stress values and moduli of elasticity, so that loads and deflections do not correspond. However, the mechanical behavior of these under-reinforced RBM members is so similar to reinforced concrete that RBM might well be considered a special type of reinforced concrete construction.

Thus, the pertinent question in the consideration of RBM construction is whether or not the principles used in the design of reinforced concrete can be applied to the design of RBM. For the wall panels such as considered in this program, subjected to transverse loads, three types of failure may be encountered; bond failure of the reinforcing steel, shear failure through the masonry, and bending failure due to yielding of the reinforcing steel and/or compression failure in the masonry.

The shear and bond stresses may be calculated by the classical formulas:

$$v = V/bjd$$

$$u = \frac{V}{\Sigma o\, jd}$$

where, v = shear stress, in psi

V = total shear in lbs.

j = .875

u = bond stress, in psi

Σo = total reinforcing steel permeter, in inches.

For b and d, see Fig. 5.8.

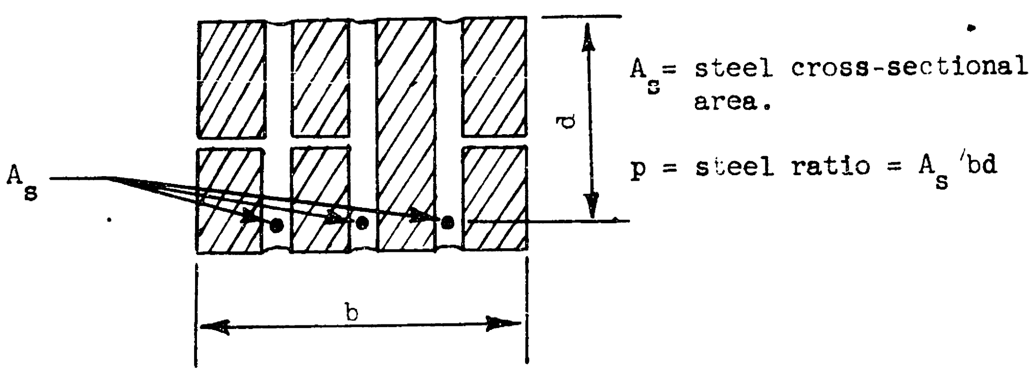


Figure 5.8 Cross-Sectional View of Typical Reinforced Masonry Section

Because shear and bond failure are sudden and occur without warning under static loads, and are ineffective in absorbing much energy under dynamic loads, it is advisable to preclude their occurance by keeping design values of v and u below ultimate values.

The bending moment failure of the walls could be theoretically predicted by use of the plastic design theory (10) extensively used in reinforced concrete design.

This theory assumes a stress distribution at the plastic load as shown in Fig. 5.9.

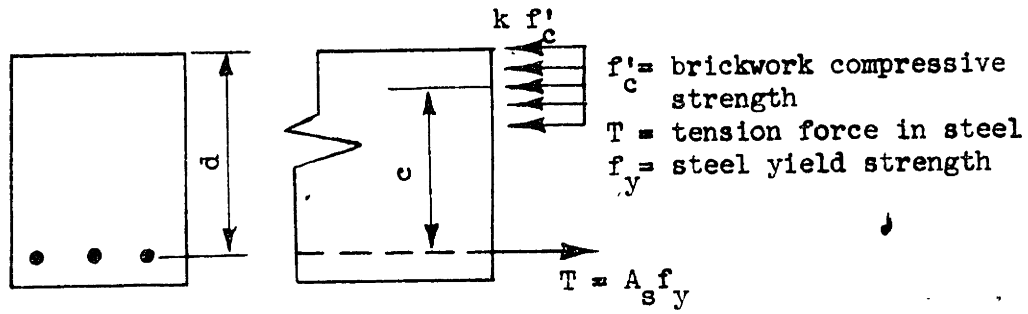


Figure 5.9 Wall Section With Assumed Stress Distribution According to Plastic Design Theory

According to the plastic theory, the plastic moment of a section failing by yielding of the tensile steel* is:

$$M_{PL} = c\ A_s\ f_y$$

By substituting the actual values of A_s and f_y and the observed plastic moment for M_{PL}, it may be seen from Table 5.3 that for the members which failed in bending moment, the resulting value for c comes out very close to the value used for d. Hence, for under-reinforced members, a fairly accurate prediction can be made of the plastic moment by

*Failure by compression in the masonry, which is sudden and not capable of absorbing appreciable energy, may be precluded by under-reinforcing the section.

TABLE 5.3

Computed Results of Tests on Reinforced Slabs and Beams - Bending Moment Failure

Test No.	Observed Ult. Mom. (in.k/ft)	A_s (in.^{2})	p (%)	d (in.)	c (in.)	Pred. Ult. Mom. $A_s f_y d$ (in.k/ft)
6R1	183.0	0.44	0.57	7	7.04	182.0
6R2	181.0	0.44	0.57	7	6.96	182.0
6R3	181.0	0.44	0.57	7	6.96	182.0
8R1	181.6	0.44	0.57	7	6.98	182.0
12R1	147.0	0.44	0.57	7	6.68	154.0
12R2	145.5	0.44	0.57	7	6.62	154.0
IS2	188.0	0.48	0.57	7	6.64	198.0

TABLE 5.4

Computed Results of Tests on Reinforced Slabs and Beams - Bond Slippage Failure

Test No.	Mortar Mix (cement: lime: sand, by vol)	Mortar Comp. Strength (psi)	Shear Stress at Ult. Ld. (psi)	Bond Stress at Ult. Ld. (psi)
RS1	1:1:5	998	187.0	438.0
RS2	1:1:5	825	194.0	452.0
RS3	1:$\frac{1}{4}$:3	2020	165.0	563.0
RS4	1:$\frac{1}{4}$:3	2070	142.0	487.0
RS5	1:$\frac{1}{4}$:3	2190	239.0	822.0
R4S1	1:$\frac{1}{4}$:3	1500	101.0	468.0
R4S2	1:$\frac{1}{4}$:3	970	115.0	518.0
R4S3	1:$\frac{1}{4}$:3	1280	93.0	421.0
R4S4	1:$\frac{1}{4}$:3	1165	77.8	358.0

disregarding the brickwork compression on the wall section and thereby using the formula:

$$M_{PL} = d\, A_s f_y$$

The predicted and actual plastic moments for panels failing in bending, computed by the above-mentioned methods, are summarized in Table 5.3.

Little can be stated concerning design procedure applicable to members with larger percentages of reinforcing steel. However, the steel used in the test specimens may well be considered a practical maximum for RBM construction. In order to add more steel, uneconomical factors must be brought in such as locating the steel where its moment arm is small or causing serious inconvenience to the bricklayer; or an entirely different reinforcing scheme must be substituted.

The ultimate bond and shear stresses for the panels failing due to bond slippage are computed by the previously discussed formulas and tabulated below in Table 5.4. Based on this limited amount of data, the ultimate bond stress may be shown to vary with mortar compressive strength, other factors being held constant, as shown in Fig. 5.10.

Based upon the observations of cracking in the plastic range, it may be presumed that the 8 in. RBM walls tested remain stable and safe when deflected as much as 1/50th of the span length.

The results of this series show that RBM is successful in developing more strength and absorbing more energy than unreinforced brick members. These truths are emphatically brought out by comparison of the moment-deflection curves for the various specimen types.

The two panels tested dynamically sustained roughly comparable damage under dynamic loads of 50 to 70% of the static loads required to inflict the same damage on the panels tested statically. Past testing indicates that a dynamic load factor of 1.6 to 1.8 may be present under these test conditions. If true, this would indicate that the specimen is operating above its static yield strength values, and that in dynamic design the yield point of the steel and perhaps also the compressive strength of the masonry should be increased by a suitable factor as is done in reinforced concrete design.

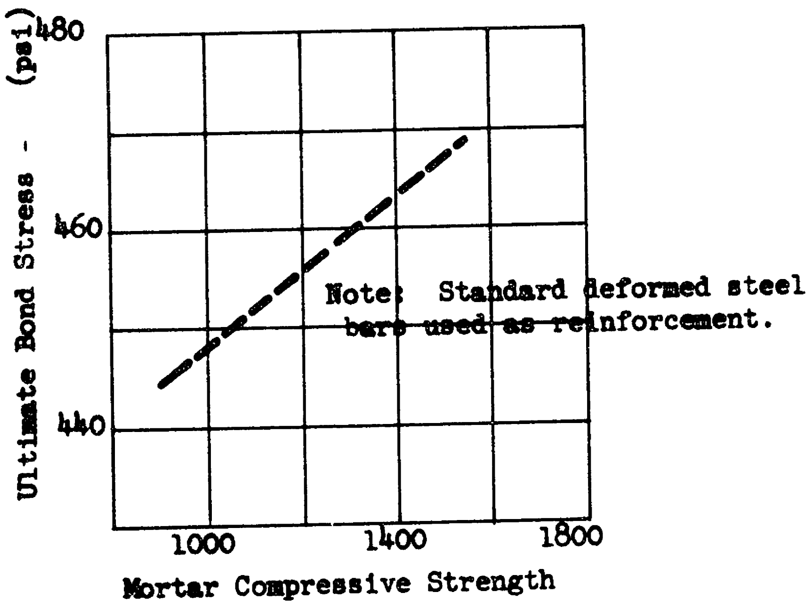


Figure 5.10 Mortar Compression Strength vs Ultimate Bond Strength

Because of the limited number of dynamic tests, nothing further can be concluded. It seems logical to expect that RBM would behave in the plastic range much as does reinforced concrete, but there has been no verification. However, since most RBM walls will be under-reinforced, the steel properties will become the determining factor; in the absence of more complete information it seems that dynamic design as developed for reinforced concrete could also be applied with reasonable accuracy to RBM.

5.5 CONCLUSIONS

Based on the limited number of transverse bending tests on RBM panels and beams, it may be concluded that:

(a) The plastic moments for RBM members having type A mortar (13) and steel ratios of 0.57% or less may be predicted with reasonable accuracy using the plastic theory design method neglecting compression in the masonry and assuming the moment arm c in the formula:

$$M_{PL} = c\,A_s f_y$$

be equal to d, the distance from the compression face of the member to the steel centroid; RBM members 3 ft in span and consisting of type A mortar and having a thickness of at least 4 inches and a steel ratio larger than 0.6% will fail due to bond slippage or shear.

(b) RBM members behave quite similarly to reinforced concrete members under static load, particularly in regards to: deflection, plastic action, ultimate load, bending action, and bond.

(c) Ultimate bond stresses for RBM are presented in Fig. 5.10.

(d) Deflections up to approximately 1/50th of the span length for the span depth ratios and percentages of steel tested may be sustained without danger of complete collapse or instability.

(e) The addition of reinforcing steel to a brick member increases its ability to absorb energy. The 6 ft by 9 ft panel absorbed approximately 36 times as much energy as the similar panel, LS1, which had no steel reinforcing.

CHAPTER 6

REINFORCED GUNITED MASONRY WALLS

6.1 GENERAL

In order to increase the transverse bending strength of existing brick masonry walls, it is necessary to provide the walls with steel reinforcement to resist bending tensile stresses, since plain masonry is relatively weak in tension. One way to provide the existing walls with this tensile strength would be to place steel rods up against the tension face of the wall and then to spray a layer of gunite to make this steel reinforcing integral with the wall.

Gunite is a mixture of cement, sand, and water, applied under air pressure. Some of the advantages of gunite are that it has a strength two or three times that of average concrete, that it possesses a density that makes it a waterproofing medium, that it gives good protection to steel against rusting, and that it resists the action of salt water, frost, and to a large extent, alkalis and acids.

Tests on gunited reinforced brick panels and beams were conducted in order to evaluate the effect of this reinforcing process on the transverse strength and behavior of brick masonry walls panels.

6.2 DESCRIPTION OF TESTS

A total of 20 reinforced gunited panels and beams were tested. The specimens were laid up of waterstruck brick in American bond using a 1:1:5 mortar. Each specimen was cured dry at least seven days and then faced with a 1½ in.

layer of reinforced gunite. (See Fig. 6.1) All members were reinforced with intermediate grade standard deformed steel bars as specified in ASTM specification A305, with the exception of two walls in which two-way woven wire mesh was used. In addition, one 12 ft. long hollow concrete block beam was coated with reinforced gunite; concrete blocks described in Sect. 7.1.1 were laid in 1:1:5 mortar.

The gunite used in preparing the specimens was a 1:4 mix (by volume) of portland cement and sand. Twenty-eight-day tests yielded compressive strengths averaging 4140 psi and moduli of rupture averaging 852 psi.

The method of placing the steel and applying the gunite is described and illustrated in Appendix D. Note particularly that no special method was used to bond the reinforced gunite to the masonry. The exceptionally high adhesive strength of the gunite made it possible to rely entirely on the gunite-

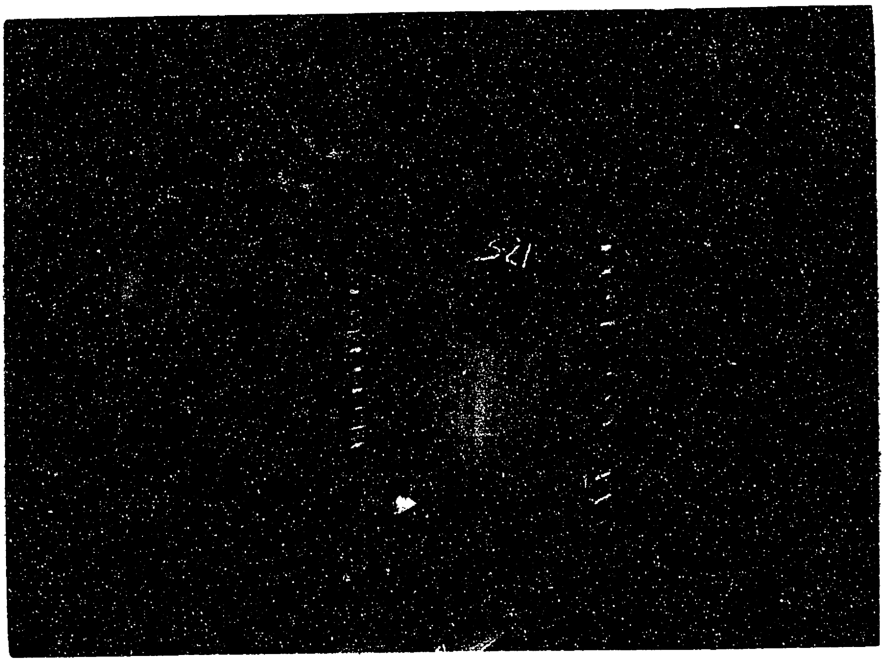


Figure 6.1 Reinforced Gunited Brick Panels

masonry bond.

The steel ratios of the beams and slabs were varied, in an attempt to obtain different failure modes. (For a detailed description of the beams and slabs tested, see Table 6.1.)

Strain gauges were placed on some of the reinforcing rods and on some of the compression mortar joints at midspan in order to obtain actual tension and compression strains during the test. All specimens were cured wet for seven days and then cured dry until testing, which took place 26 to 30 days after the guniting. All walls and beams were tested in support condition (2), and all masonry members were tested while supported in the strong direction.

Static tests on panels were conducted in the slab machine under a uniformly distributed pressure load, and beams were tested statically in a universal loading machine under a third point loading.

Dynamic tests were conducted on panels in the slab machine by applying a uniformly distributed pressure load impulsively with a full load buildup in 3 millisec. and a load duration of approximately 10 secs.

The data recorded during the tests were the ultimate load, mode of failure, midspan deflection, and strain readings from SR-4 gauges on rods and mortar joints, when such gauges were applied.

Four panels of solid reinforced gunite were fabricated and tested statically. The description and results of these

TABLE 6.1

Description of Tests on Gunited Reinforced Slabs and Beams

Test No.	Description	Test Type*	Nominal Masonry Thickness (in.)	Gunite Thickness (in.)	Span Length (in.)	d (in.)*	b (in.)**	p (%)**	Steel Arrangement***
RGS-1	Gunite on Brick	S	8	1½	36	8	35	0.46	
RGS-2	Gunite on Brick	S	8	1½	36	8	35	0.46	3/8"Ø rods at
RGS-3	Gunite on Brick	S	8	1½	36	8	35	0.46	3" O.C.
RGS4-1	Gunite on Brick	S	4	1½	36	4	35	0.46	
RGS4-2	Gunite on Brick	S	4	1½	36	4	35	0.46	3/8"Ø rods at
RGS4-3	Gunite on Brick	S	4	1½	36	4	35	0.46	6" O.C.
MGS4-1	Gunite on Brick	S	4	1	36	3⅝	35	0.12	2-way mesh steel
MGS4-2	Gunite on Brick	S	4	1	36	3¾	35	0.12	2 x 2 12/12
RGD4-1	Gunite on Brick	D	4	1½	36	4	35	0.46	
RGD4-2	Gunite on Brick	D	4	1½	36	4	35	0.46	3/8"Ø rods at
RGD4-3	Gunite on Brick	D	4	1½	36	4	35	0.46	6" O.C.
6RG-1	Gunite on Brick	S	8	1½	72	8	18	0.46	
6RG-2	Gunite on Brick	S	8	1½	72	8	18	0.46	
6RG-3	Gunite on Brick	S	8	1½	72	8	18	0.46	3/8"Ø rods at
12RG-1	Gunite on Brick	S	8	1½	142½	8	18	0.46	3" O.C.
12RG-2	Gunite on Brick	S	8	1½	144½	8	18	0.46	
12RG-3	Gunite on Brick	S	8	1½	143	8	18	0.46	
12RGC-1	Gunite on Conc. Block	S	8	1½	143½	8¼	16	0.42	5-3/8"Ø rods at 3" O.C.

*S = Static, D = Dynamic

**See Fig. 6.8

***All steel rods straight, all reinforcement 2 way.

secondary tests are presented in Appendix D.

6.3 TEST RESULTS

Table 6.2 presents the ultimate loads and other static test results obtained from testing the reinforced gunited masonry panels and beams. The ultimate load in psi listed for the specimens tested under third point loading is the uniformly distributed load necessary to cause the same moment at midspan as the ultimate third point loading. The ultimate load is also represented in terms of ultimate moment at midspan in in.-kips/in. width of panel or beam. (Sample computations are presented in Appendix C)

Average curves of midspan transverse moment per unit width vs midspan deflection for the 4 in. statically tested reinforced gunited masonry slabs and for the 6 ft and 12 ft reinforced gunited masonry beams are presented in Figs. 6.2 and 6.3.

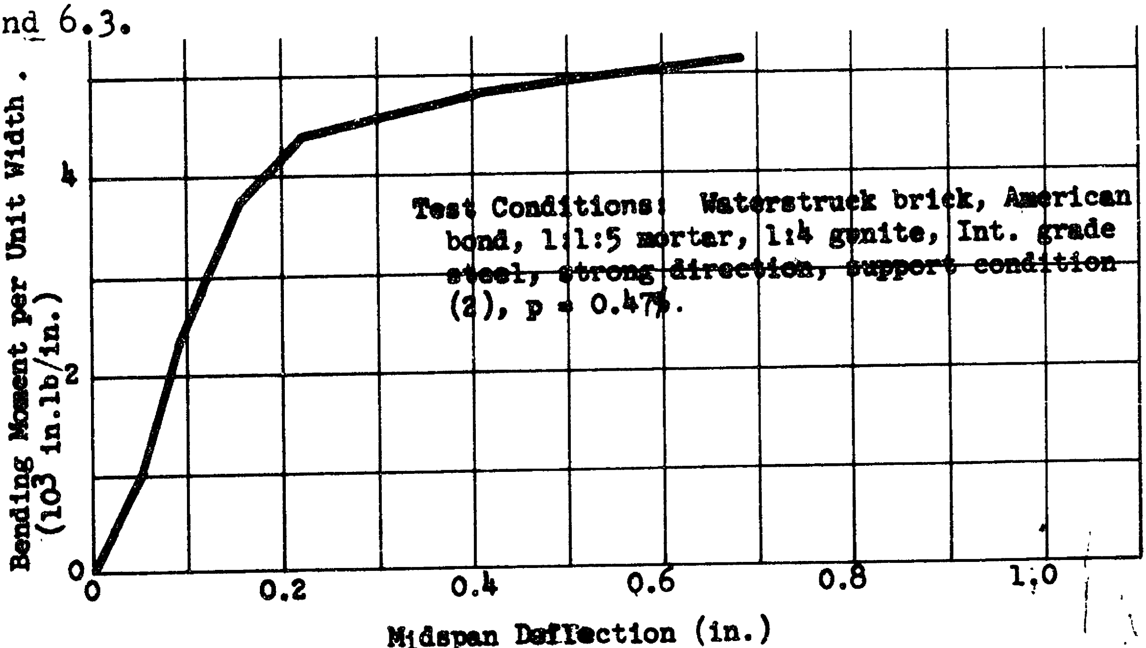


Figure 6.2 Bending Moment per Unit Width vs Midspan Deflection Due to Uniform Load on 4 in. Reinforced Masonry Wall with $1\frac{1}{2}$in. Coat of Gunite, Span Length = 36 in.

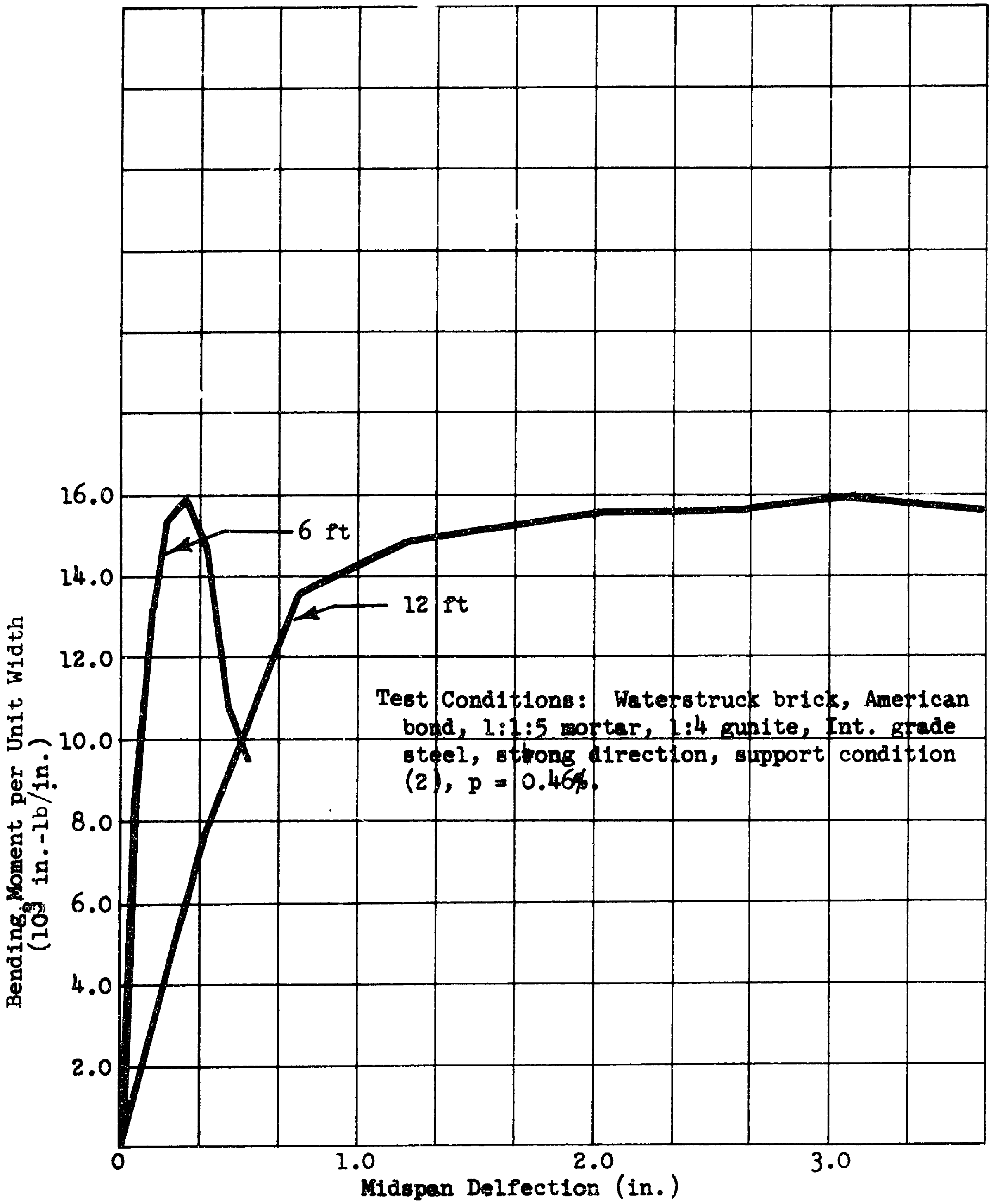


Figure 6.3 Bending Moment per Unit Width vs Midspan Deflection due to Third Point Loading on 8 in. Reinforced Masonry Beams with 1 1/2 in. Gunite Coating.

TABLE 6.2

Test Results on Gunited Reinforced Slabs and Beams

Test No.	Ultimate Load			Type Failure	Deflection at Ult. Ld. (in.)***	Remarks
	kips (LL only)	psi (incl DL)	Moment (in.-kips/in.			
RGS-1		106.7	17.3	Bond	0.085	
RGS-2		85.7	13.9	Bend	0.070	
RGS-3		92.7	15.0	Bond	0.080	
RGS4-1		24.4	4.0	BM	0.240	
RGS4-2		24.4	4.0	BM	0.150	
RGS4-3		28.4	4.6	BM	0.195	
MGS4-1		15.2	2.5	BM	0.066	greatly under-
MGS4-2		10.7	1.7	BM	0.075	reinforced
6RG-1	26.0	27.4**	17.8	Bond	0.191	****
6RG-2	26.5	27.9**	18.1	BM	--	
6RG-3	26.4	27.9**	18.1	BM	0.216	
12RG-1	10.6	6.2**	16.1	BM	0.752	
12RG-2	10.3	6.0**	15.6	BM	1.007	
12RG-3	11.3	6.6**	17.2	BM	0.972	
12RGCo-1	4.0	2.8**	7.3	Shear across block webs	0.340	

Steel rod yield stress = 59,000 psi, steel mesh yield stress = 64,700 psi.

** For sample calculations, see Appendix C.

*** Deflection values are from the dead load position. Where failure was due to BM, deflection value is for the plastic load.

**** Failure over one support.

The basic types of failure indicated by the tests are also tabulated in Table 6.2.

Panels RGS 1, 2, and 3 failed suddenly and completely in bond slip along the reinforcing rods followed by shear failure through the masonry. Failure occurred considerably before the reinforcing rods developed their yield strength. (See Fig. 6.4)

Panels MGS 4-1 and 2, which were reinforced with wire mesh, failed suddenly in rupture completely through both the steel and brickwork. No tension cracking occurred in the gunite prior to this failure. The modes of failure showed general characteristics similar to failures observed in unreinforced brick panels. (See Fig. 6.5)

Beam 6RG-1 showed shear failure in the mortar joints of the masonry near one end of the beam.

The hollow concrete block beam failed in shear across the webs of the blocks. (See Fig. 6.6) This occurred when the reinforcing steel was stressed to considerably less than its yield point.

All other reinforced gunited members failed in yielding of the steel due to bending moment at midspan. As the load was applied, cracks opened in the gunite which widened and spread up into the masonry as the load increased and the steel was strained. Steel strains increased as the member yielded.

No separation between the masonry and the gunite layer was observed until excessive plastic deflections were reached.

The deflections at which this occurred varied from 1/32 to 1/40 of the span length for the 8 in. walls which failed due to bending moment.

A photograph illustrating a typical failure is shown in Fig. 6.7.

The results on three 38 in. square panels 4 in. thick tested under dynamic loads in the slab machine are presented below.

RGD4-1

tested at 22 psi - no damage

retested at 23 psi - slight cracking on tension side no measurable permanent deflection.

RGD4-2

tested at 22 psi - no damage

retested at 23 psi - slight cracking on tension side no measurable permanent deflection.

RGD4-3

tested at 25 psi - slight cracking on tension side no measurable permanent deflection.

6.4 DISCUSSION OF RESULTS

The tests described above bring out the resemblance of the behavior of reinforced gunited masonry panels to reinforced brick and reinforced concrete panels. This behavior is particularly similar in the instances of under-reinforced members which failed due to yield to the tensile steel. This material may well be considered a special type of reinforced concrete construction.

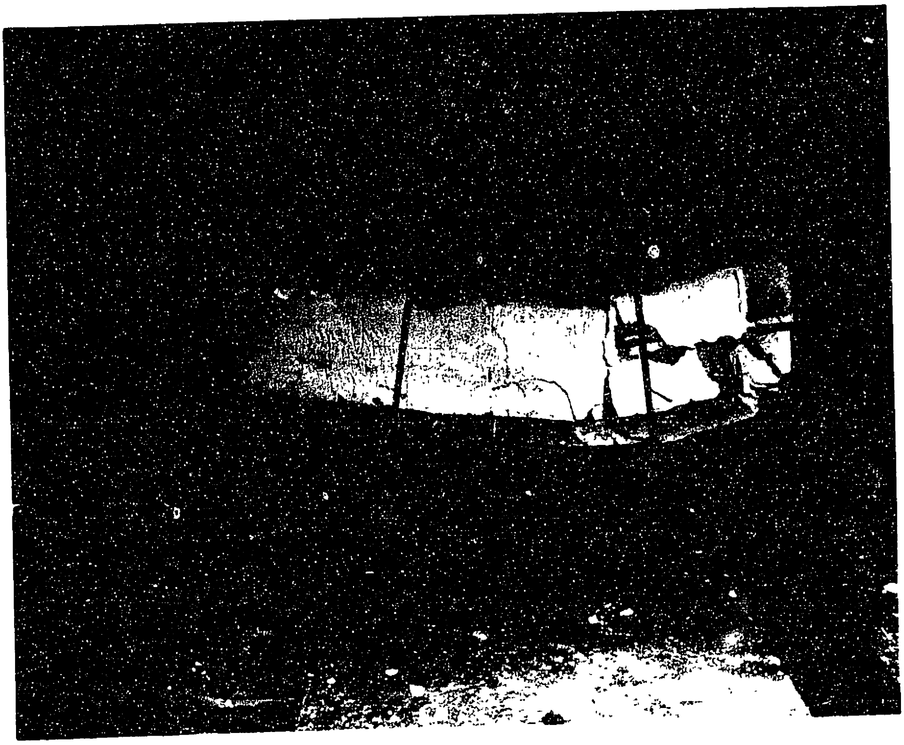


Figure 6.4 Panel RGS-1 After Failure

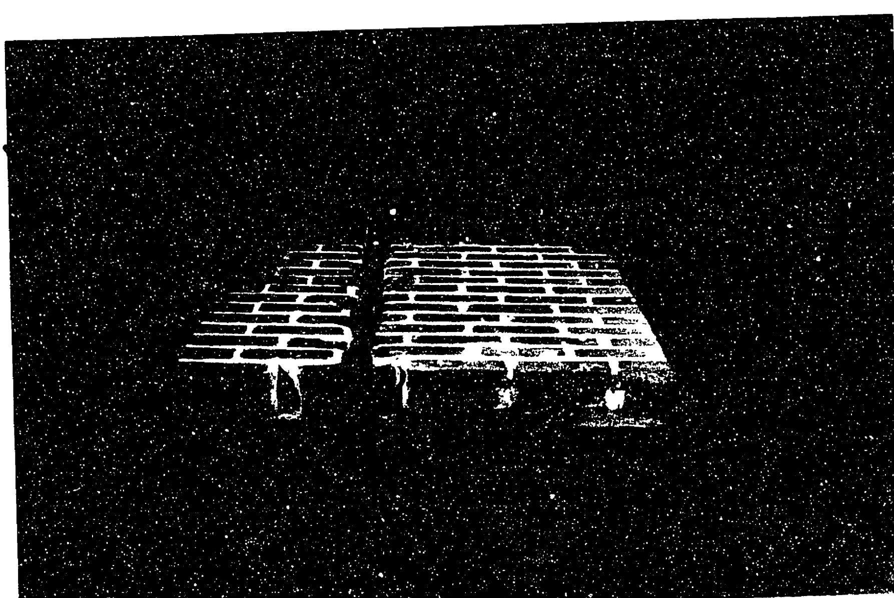


Figure 6.5 Panel MGS4-1 After Failure

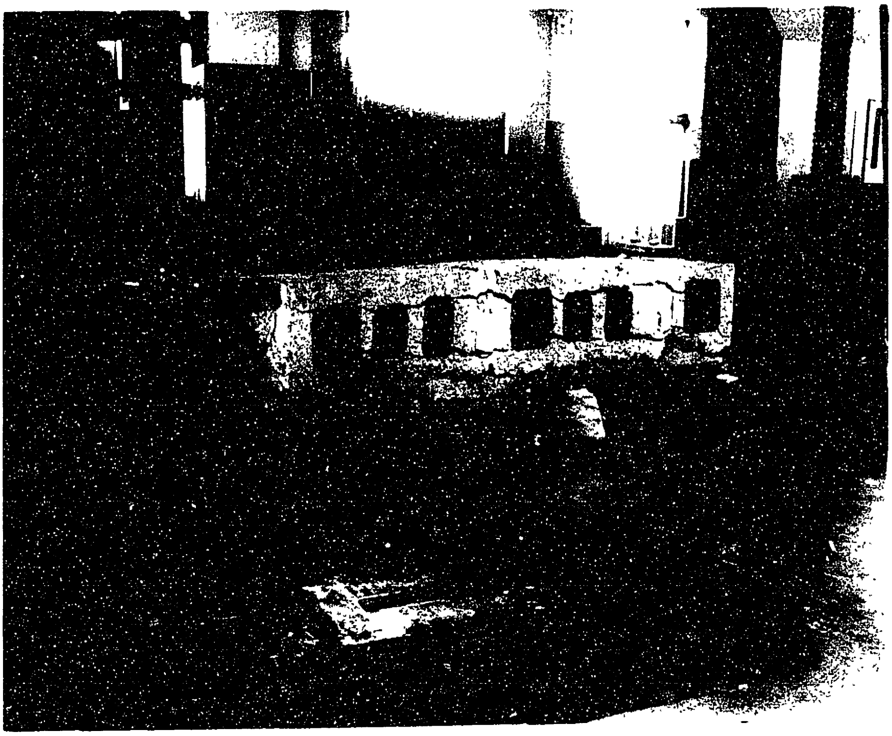


Figure 6.6 Reinforced Gunited Concrete Block Beam After Failure

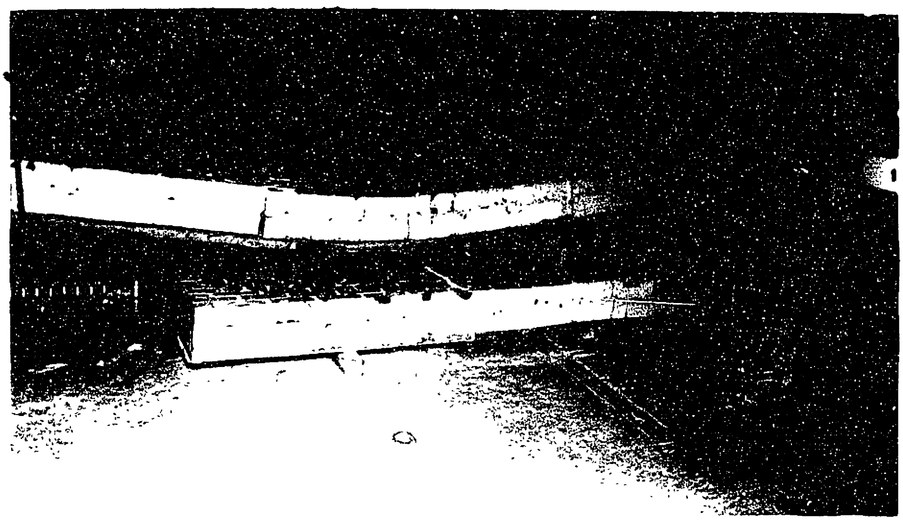


Figure 6.7 Bending Moment Failure of Reinforced Gunited Masonry Beam

A pertinent question in reinforced gunited masonry, as it was in reinforced masonry, is whether or not the principles applied in the design of reinforced concrete can be applied to the design of reinforced gunited masonry construction. The wall panels considered in this section, subjected to transverse loads, encountered the same three modes of failure as those included in Chapter 5, i.e., bond failure along the reinforcing steel, shear failure through the masonry, and bending moment failure. Shear and bond failure stresses were calculated in a manner similar to the one described in Chapter 5. The bending moment failure of the walls was reconciled by use of the plastic design theory (10). As in the preceeding chapter, substituting the observed values into the plastic theory formula for the plastic moment indicated that the resulting value for c, the moment arm, will be very close to the value d, the distance from the compression face to the centroid of the reinforcing steel. (See Fig. 6.8) Consequently, the predicted plastic moment was computed by the formula:

$$M_{PL} = d\ A_s f_y$$

Table 6.3 includes a tabulation of the moment arms c, obtained by substituting the observed values into the plastic theory design formula, and of the predicted ultimate moments, using the formula shown above, for the reinforced gunited members failing in bending moment; and a tabulation of the shear and bond stresses at ultimate load for those members failing in bond and shear.

One of the reasons which may explain the deviations in the computed results appearing in Table 6.3 is the difficulty in knowing the actual value of d because of construction methods.

Panels RGS-1, 2, and 3 which failed due to bond slippage along the reinforcing rods in the gunite, provide ultimate values of bond stress to be expected under the existing conditions. These values may be computed by the formula:

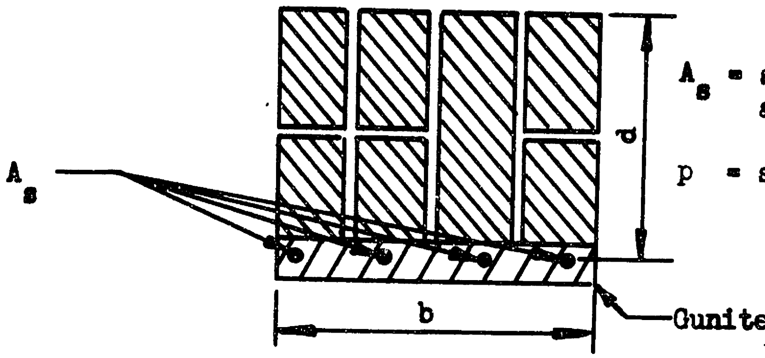
$$u = V / \sum o j d$$

as used before. The values, tabulated in Table 6.3, range from 562 to 700 psi with an average value of 621 psi and a probable error of $\pm$ 28 psi.

The mode of failure encountered in panels reinforced with wire mesh indicated that the panels were grossly under-reinforced. Apparently the bending strength added by virtue of the steel mesh was less than that of plain gunite layer.

The failure of beam 6RG-1 in shear near the supports is not consistent with the shear stress of 106 psi developed at failure. This is considerably lower than shear stresses successfully sustained in other tests. Apparently the failure was due to a poor batch of mortar incapable of developing consistent shear strength.

The tests serve to substantiate the belief that no extraordinary bending methods need to be used to attach the gunite layer to the masonry. Results showed that the adhesive strength of the gunite mixture was sufficient to cause the two materials to act integrally until excessive plastic deflections were reached. Therefore, the use of anchorages



A_s = steel cross-sectional area.

p = steel ratio = A_s/bd

Figure 6.8 Cross-Sectional View of Typical Reinforced Gunited Masonry Section

TABLE 6.3

Test No.	Observed Pl. Mom.	A_s (in.^{2}/ft)	c (in.)	Pred. Ult. Mom. = dA_sf_y (in.-kips/ft)	Sh. Stress at Ult. Ld. (psi)	Bd. Stress at Ult. Ld. (psi)
RGS-1	207.5*	0.44			274.0	700.0
RGS-2	166.7*	0.44			220.0	562.0
RGS-3	180.0*	0.44			238.0	600.8
RGS4-1	48.0	0.22	3.70	51.9		
RGS4-2	48.0	0.22	3.70	51.9		
RGS4-3	55.2	0.22	4.25	51.9		
MGS4-1	30.0*	0.052				
MGS4-2	20.4*	0.052				
RGD4-1	44.5	0.22				
RGD4-2	49.2	0.22				
RGD4-3	52.8	0.22				
6RG-1	213.5	0.44	8.20		106.0	271.0
6RG-2	217.0	0.44	8.35	208.0		
6RG-3	217.0	0.44	8.35	208.0		
12RG-1	193.0	0.44	7.43	208.0		
12RG-2	187.0	0.44	7.20	208.0		
12RG-3	216.5	0.44	8.35	208.0		
12RGCo-1	87.5*	0.41			37.5**	

*This is an ultimate moment rather than a plastic moment in that failure was sudden and complete.

**Computed assuming that only web area resisted shear.

or set mortar joints in unnecessary.

Observations on the under-reinforced specimens tested in the plastic range showed that such reinforced gunited masonry walls remain stable and integral when deflected as much as 1/40 of the span length for 12 ft spans, and approximately 1/250 of the span length for 6 ft spans.

It appears that the under-reinforced gunited masonry panels are successful in developing more strength and absorbing more energy than unreinforced brick members. Comparison of moment vs deflection curves emphatically brings this out.

The single test on reinforced gunited concrete block indicate that while the scheme is successful in strengthening the concrete block masonry, the web of the concrete block does not provide enough area for shear strengths to develop the potential bending strength of the reinforced gunite. By using a smaller percentage of reinforcing steel, a balanced design could be evolved. Another possibility might be to fill the hollow portions of the concrete block panel with grout. However, under the conditions tested, the scheme is less successful on hollow concrete block panels than ordinary solid brick masonry panels.

The three panels tested dynamically cracked slightly under dynamic loads of 23 to 25 psi. Identical panels under static loads failed completely under loads between 24 to 29 psi. No absolute conclusions may be drawn from these results. The limited results do indicate that if the dynamic load factor is between 1.5 and 1.9 there is a substantial increase in the yield strength of the steel under dynamic rates of straining. This is logical, considering the likeness of the material to reinforced concrete, particularly since failure was due to the steel yielding. Of particular interest in these dynamic tests was the observation that the gunite adhesive bond to the masonry behaved no different than under static loads.

Considering the nature of the material and the limited test results, it seems logical to expect that under-reinforced gunited masonry would behave in the plastic range as does reinforced concrete. As in the case of RBM construction, it would appear that the dynamic design principles developed for reinforced concrete can also be applied to reinforced gunited masonry.

A brief note should be made concerning the economics of this reinforced gunited masonry construction. The cost of gunite application is high, as much as 3 to 5 times that for a same amount of concrete. Also, the guniting of panel interiors may be impractical or even impossible because of the necessary disturbance with respect to occupancy or because of the physical characteristics of the building.

No economic comparison can be made from this test program because of the specialized form of the test specimens. It may be stated, that although the scheme is highly successful in physically strengthening existing masonry walls, it is conceivable that in many instances the scheme may not be economically sound.

6.5 CONCLUSIONS

Based on the limited number of transverse bending tests on reinforced gunited masonry, it may be concluded that:

(a) Reinforced gunite as used herein is physically satisfactory in strengthening plain brick masonry panels against transverse bending, where economically justified.

(b) The behavior of reinforced gunited masonry is quite similar to reinforced concrete members, particularly with respect to: deflection, plastic-action, ultimate load, bending action, and bond.

(c) The plastic moments for reinforced gunited masonry members having steel ratios of 0.46% or less may be predicted with reasonable accuracy using the plastic theory design method neglecting compression in the masonry and assuming the moment arm c in the formula:

$$M_{PL} = c\, A_s f_y$$

is equal to d, the distance from the compression face of the member to the steel centroid.

(d) Ultimate bond stresses of 560 to 700 psi may be developed using standard deformed bars in gunite developing an average 28 day compressive strength of 4140 psi.

(e) Deflections up to approximately 1/40 of the span length may be sustained without complete collapse of separation of the gunite-masonry bond, for steel ratios up to 0.46%, and for depth-span ratios of 1/18. For similar steel ratios and a depth ratio of 1/9, however, the deflection which can be sustained without collapse decreases to 1/250 of the span length.

CHAPTER 7

MISCELLANEOUS WALL PANELS

7.1 CONCRETE BLOCK PANELS

The following tests were conducted in order to evaluate the strength and mode of failure of hollow concrete block masonry walls under uniformly distributed transverse load.

7.1.1 Description of Tests

Six concrete block wall panels were tested statically in this series. All the panels were constructed of standard 8" x 8" x 15" hollow concrete blocks, manufactured locally. The average compressive strength of the block was 2910 psi based on net area, and the average absorption was 5.6% (both by A.S.T.M. standards). The wall panels were 4 courses (34") high, 37" wide, and 7 11/16" thick (See Fig. 7.1).

All wall panels in this series were laid in a 1:1:5 mortar mix, (cement, lime, sand, by volume). This mortar, which provides an optimum cohesive mix, is standard practice for concrete block masonry. Standard construction methods were employed and good to excellent workmanship was maintained.

All wall panels were tested to destruction in the slab testing machine, loaded with a uniformly-distributed transverse load applied statically. All wall panels were supported in the strong direction on a 36 in. span using support condition (2).

Ultimate loads were recorded for all panels tested. The midspan deflection was recorded during the test, and the behavior and mode of failure noted for each wall.

7.1.2 Test Results

The mortar properties, the ultimate load, and the modulus of rupture for each panel are tabulated in Table 7.1. The average ultimate modulus of rupture is 197.5 psi with a probable error of ± 17 psi. The average curve of midspan moment per unit width vs midspan deflection for the panels is presented in Fig. 7.2.

Failure occurred entirely through the mortar joints; in no instance were any concrete block units ruptured or damaged in any manner. Usually, failure was initiated by the opening of a tension crack in a masonry unit-mortar joint near midspan. As deflection increased, these cracks opening wider up to a point where the longitudinal and transverse mortar joints cracked almost instantaneously, and complete failure occurred.

TABLE 7.1

Test Results on Concrete Block Wall Panels
Static Loadings
(36" span - strong direction)

Test No.	Mortar		Ultimate Load (DL incl) (psi)	Midspan Defl. at Ult. Ld. (in.)*	Modulus of Rupture (psi)**
	Avg. Flex. (psi)	Avg. Comp. (psi)			
Co B-S-1			8.3	0.015	137
Co B-S-2			9.3	0.014	149
Co B-S-3	286	1123	11.8	0.029	195
Co B-S-4	275	1108	14.2	0.030	234
Co B-S-5	191	825	10.4	0.025	172
Co B-S-6	148	718	17.4	0.033	297.7

*Deflection values are from the dead load position.
**Based on gross area. For sample computations, see Appendix C.

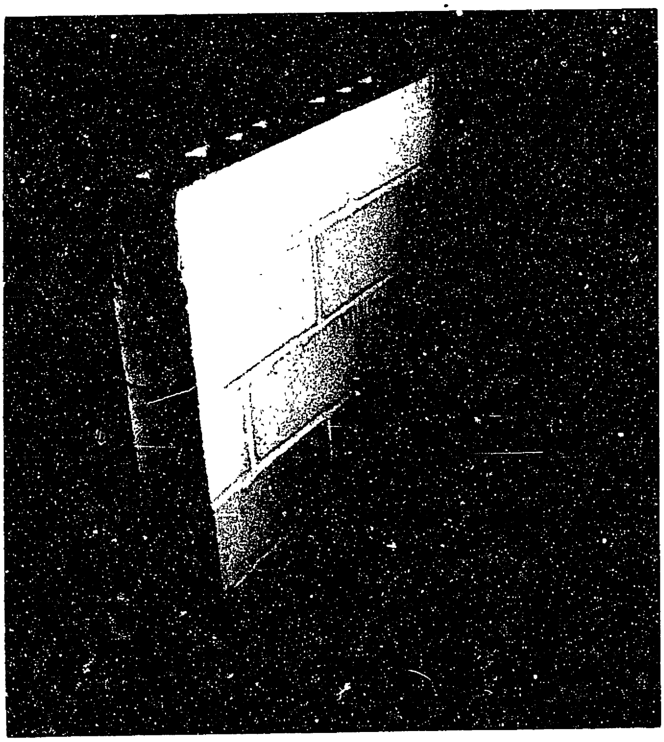


Figure 7.1 Hollow Concrete Block Masonry Wall

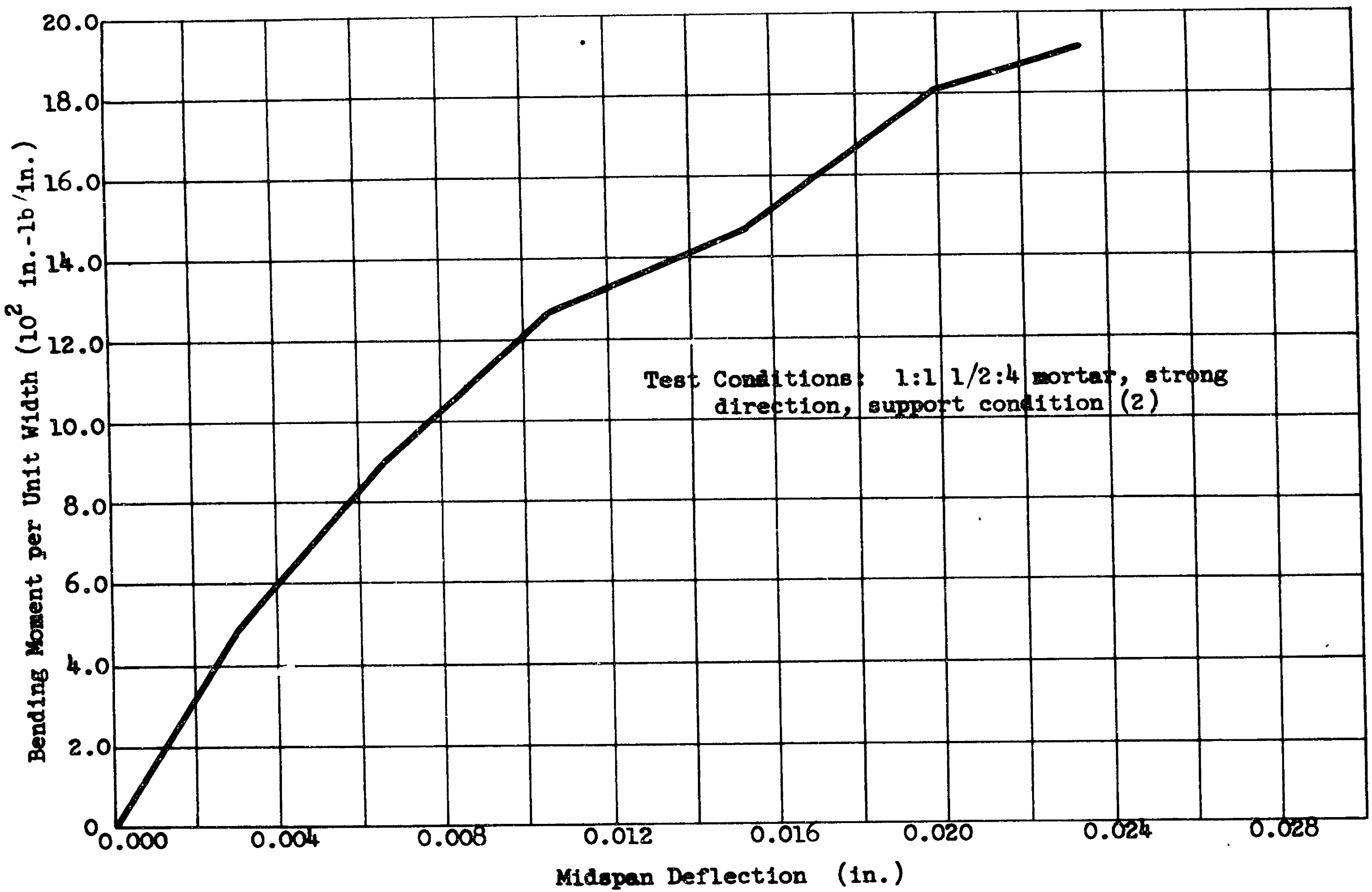


Figure 7.2 Transverse Bending Moment per Unit Width vs Midspan Deflection for Static Uniform Load on 8 in. Hollow Concrete Block Wall Panels of Span Length = 36 in.

7.1.3 Discussion of Results

The results of the six tests show a large variation from the average modulus of rupture. This is probably due to the larger ratio of the size of a masonry unit to the overall size of the wall panel. (This ratio was approximately three times that of the 36 in. square brick walls tested.) The effect of the largeness of this ratio is to increase the detrimental effect of an imperfect joint, since the uninterrupted length of the joint is a relatively large proportion of the total wall height. It is reasonable to expect that if concrete block wall panels with a height of 14 courses (about 120 in.) were tested, results of the consistency of those obtained for brick wall panels would be achieved.

The average modulus of rupture of corresponding 8 in. brick panels, tested under similar conditions, was 247 psi. For the six concrete block wall panels, the average modulus of rupture of 198 psi is about 20% lower. This decrease is due to the combination of a number of inherent factors. It is sufficient to say that, generally, typical wall panels of concrete block masonry are less resistant to transverse loads than corresponding typical brick masonry panels.

Due to large variations in the static test results, a proposed program of dynamic tests on concrete block wall panels was abandoned.

7.1.4 Conclusions

Based upon this limited series of tests on concrete block wall panels, and on a comparison of the results with those from tests on brick wall panels, we may conclude the following facts regarding the behavior of concrete block wall panels in resisting transverse loads:

(a) The modulus of rupture, based on gross sectional area, for bending failure for typical concrete block panels of 8" thickness ranges from 135 to 300 psi. Test results gave an average modulus of rupture of 198 psi, with a probable error of 17 psi.

(b) The ratio of masonry unit size to the overall panel size appreciably affects the consistency of results obtained from a number of tests.

7.2 PARTITION TILE PANELS

The following tests were conducted in order to evaluate the strength and mode of failure of hollow partition clay tile walls under uniformly distributed transverse load.

7.2.1 Description of Tests

Six partition clay tile panels were tested statically in this series. Five of the panels were plain and one was faced with two 3/8 in. layers of plaster of paris on the tension side.

All the panels were constructed of standard 8" x 12" x 12" hollow clay tile, manufactured locally. The average compression strength of the tile, loaded on end, was 5,130 psi, based on net area, and the average absorption was 9.26%.

The panels were all 37 in. square (3 courses high); the plain panels were 8" thick; the plaster coated panel was 8 3/4" thick. The mortar mix used was 1:1½:4 (cement: lime: sand, by volume). The walls were tested transversely in the slab loading machine on a 36" span, supported in the strong direction in support condition (2).

Data recorded were ultimate load, mid-span deflection, and mode of failure.

7.2.2 Test Results

The results of the static tests on the clay partition tile walls are presented in Table 7.2 Since it is believed that wall number PTS-1 suffered some damage prior to testing, the results of this test were not included in the discussion and conclusions for partition tile walls.

TABLE 7.2

Test Results on Clay Partition Tile Walls

Wall No.	Mortar Avg. Flex. (psi)	Mortar Avg. Comp. (psi)	Ult. Load psi (incl DL)	Modulus of Rupture (psi)*	Ultimate Deflection (in.)**	Remarks
PTS-1			1.33	20.2	0.012	Believed to have been damaged prior to test
PTS-2	190	1000	3.45	52.4	0.027	
PTS-3			2.93	44.5	0.035	
PTS-4	225	1270	2.95	44.8	0.035	
PTS-5			2.58	39.2	0.055	
PTP-S1			5.58	71.0	0.025	2-3/8" plaster of paris courses on tension face

*Based on gross area. For sample computations, see Appendix C.

**Deflection values are from the dead load position.

Failure occurred entirely through the mortar joints. Usually failure was initiated by the opening of a tension crack in the mortar joint near midspan. As the deflection increased, the cracks opened up wider to a point where all the mortar joints ruptured almost instantaneously and complete failure occurred. An average curve of midspan bending moment per unit width vs midspan deflection of the clay partition tile walls is presented in Fig. 7.3.

7.2.3 Discussion of Results

Failure of the walls was apparently due to rupture in bending moment. The average modulus of rupture of the plain 8" clay partition tile walls was 45.2 psi based on gross area, with a probable error of ± 1.9 psi.

7.2.4 Conclusions

On the basis of a limited number of tests on clay partition tile walls, it may be concluded that:

(a) The modulus of rupture of plain 8 in. walls is 45 psi with a probable error of ± 2 psi.

(b) Adding a coat of plaster of paris on the tension face of the slab increases its strength appreciably.

7.3 ASBESTOS-CEMENT BOARD

The report of August 1952 covered tests on asbestos-cement board panels with unrestrained ends only. In actual construction, typical attachments tend to restrain the ends of asbestos-cement walls. The purpose of this section, therefore, is to evaluate the effect of restrained ends on the transverse strength of asbestos-cement panels.

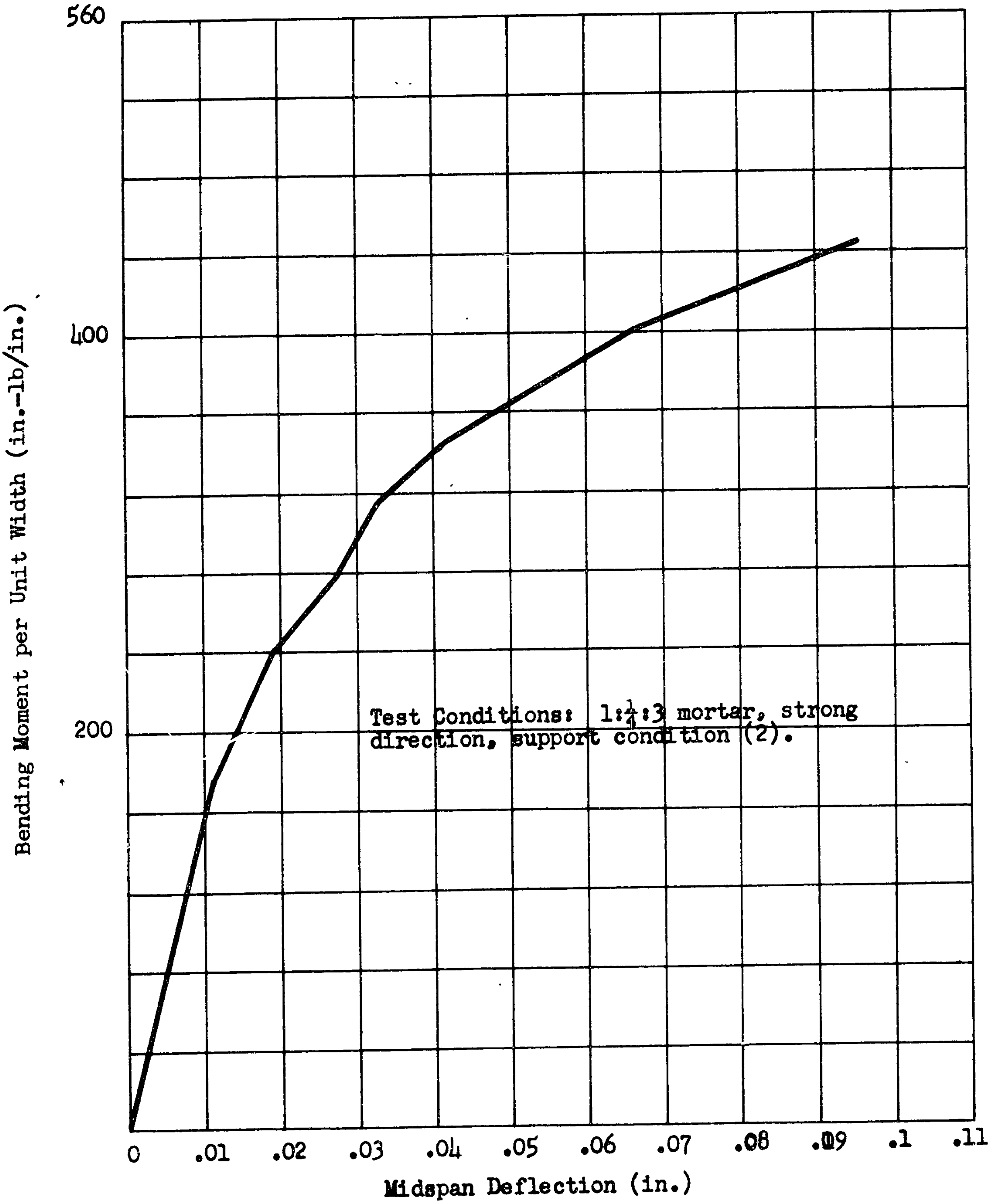


Figure 7.3 Transverse Bending Moment per Unit Width vs Midspan Deflection due to Uniform Load on 8 in. Partition Clay Tile Wall of Span Length = 36 in.

7.3.1 Description of Tests

A series of four tests were conducted on corrugated asbestos-cement board panels. The corrugated asbestos-cement board used in the tests was supplied by the Johns-Manville Company under the trade name of Transite, and is described in detail in Chapter 5 of the August 1952 report.

All of the tests were conducted on the Transite sheets placed in a horizontal position in the slab loading machine with uniformly distributed pressure applied statically. The sheets were supported on a span length of 32 in. with the supports running at right angles to the corrugations. Two of the tests were conducted on sheets which were simply supported on 6 in. 10.5 lb/ft channels providing a 2 in. wide flange for each support, which corresponds closely to the dimensions of the supporting girts that might be encountered in actual construction. The remaining two tests were conducted on Transite sheets restrained on the above-mentioned channels by means of "J" clips and $\frac{1}{4}$ in. lead head bolts through 9/32 in. holes. Three such fasteners were used on each supported end, spaced $8\frac{1}{2}$" O.C. The method of restraint is standard construction procedure, and is described in detail in a Johns-Manville Company publication entitled "Corrugated and Flat Transite".

7.3.2 Test Results

Table 7.3 presents the ultimate loads and deflections resulting from the tests on the asbestos-cement sheets.

Failure was sudden in all cases and occurred in a line approximately at midspan and running parallel to the supports. The tests conducted on Transite sheets with restrained ends also resulted in some failure cracks near the supports.

7.3.3 Discussion of Results

The results show that restraining the ends of corrugated Transite sheets by means of standard fastening procedures will increase their strength by only a small amount. It is expected that using a smaller fastener spacing would increase the end restraint, and thus result in a slightly larger increase in strength.

TABLE 7.3

Results of Tests on Corrugated Asbestos-Cement Board

(Span Length = 32 in.)

Test No.	Width (in.)	Support Condition*	Ultimate Load psi (incl DL)	Midspan Defl. at Ult. Load (in.)
TR-1-A	34	R	3.4	0.150
TR-1-B	34	U	2.9	0.230
TR-2-A	29¼	R	3.7	0.095
TR-2-B	29¼	U	3.5	0.245

*R = restrained
U = unrestrained

7.3.4 Conclusions

On the basis of a limited number of tests on asbestos-cement panels, it may be concluded that the strength of corrugated board is increased only slightly by restraining the ends by means of standard fastening procedures.

7.4 METAL SIDING

The report of August 1952 included results of tests on simply-supported panels consisting of various types of metal siding. In order to have the test conditions correspond more closely to actual conditions encountered in construction, the tests included in this section were conducted on restrained end metal siding panels.

7.4.1 Description of Tests

Three tests were conducted on 18 gauge Galbestos V-beam sheet panels measuring 38 in. by 30 in. This type of metal siding is fully described in Chapter 6 of the first report. The panels, placed in the slab testing machine with the corrugations running at right angles to the supports, were tested by applying uniformly distributed static loads on a span length of 36 in. The ends were fastened to 6 in. 10.5 lb/ft channels using standard steel L-1 drive screws with staggered threads. As recommended by the Robertson Company, three screws per end, spaced 11 in. O.C., were driven into 3/16 in. diameter holes. Data recorded were ultimate load, mode of failure, and deflection at failure.

7.4.2 Test Results

Table 7.4 presents the ultimate loads and midspan deflections obtained when the panels were loaded to total failure.

When the load was applied, panels MS-1 and MS-3 underwent local buckling of the corrugations in a line approximately at midspan and parallel to the supports at loads of 8 psi, while panel MS-2 experienced buckling at a load of 7 psi.

TABLE 7.4

Test Results on 18 Gauge Galbestos V-Beam Panels

(Restrained ends on 36 in. span length)

Test No.	Buckling Load (psi)	Ultimate Load (psi)	Midspan Deflection at Ultimate Load (in.)	Remarks
MS1	8	18	9	
MS2	7	15	7 3/4	Reloaded
MS3	8	15	8	

Complete failure occurred as follows: Specimen MS-1 pulled away from the support at one edge due to two drive screw heads shearing off and the tearing away of the sheet from the remaining drive screw. Specimen MS-2 failed as the edge tore away from two of the drive screws and specimen MS-3 failed due to one end tearing away from all three drive screws restraining that end.

7.4.3 Discussion of Results

The test results indicate that metal siding panels secured at the ends by fasteners used in standard construction procedure will fail, when loaded transversely, due to tension causing either shear failure in the fastening device or tearing of the fastened panel edge. For 18 gauge Galbestos V-beam panels, with standard fastened ends and on a span of 36 in., the ultimate load ranged from 15 psi to 18 psi. The test results of the V-beam panels with fastened ends show that local buckling occurred in these panels at a load value very close to the ultimate load for similar panels which were simply supported (1). It is obvious from the results of the tests on 18 gauge Galbestos V-beam panels that restraining the ends has a large effect on the panel's resistance to transverse loading.

A curve of span length vs load to cause buckling for 18 gauge Galbestos V-beam walls having restrained ends is presented in Fig. 7.4. (Computations used to obtain the curve are presented in Appendix C.4)

7.44 Conclusions

Based on a limited number of tests on metal siding panels, it may be concluded that:

(a) Restraining the ends of metal siding panels by fasteners used in standard fastening procedure greatly increases the panel's resistance to transverse loading.

(b) Eighteen gauge Galbestos V-beam panels having restrained ends on a span length of 36 in. fail, when tested under a uniformly distributed static load, by rupture of the material itself and/or shear failure of the fastening screws at one end of the panel.

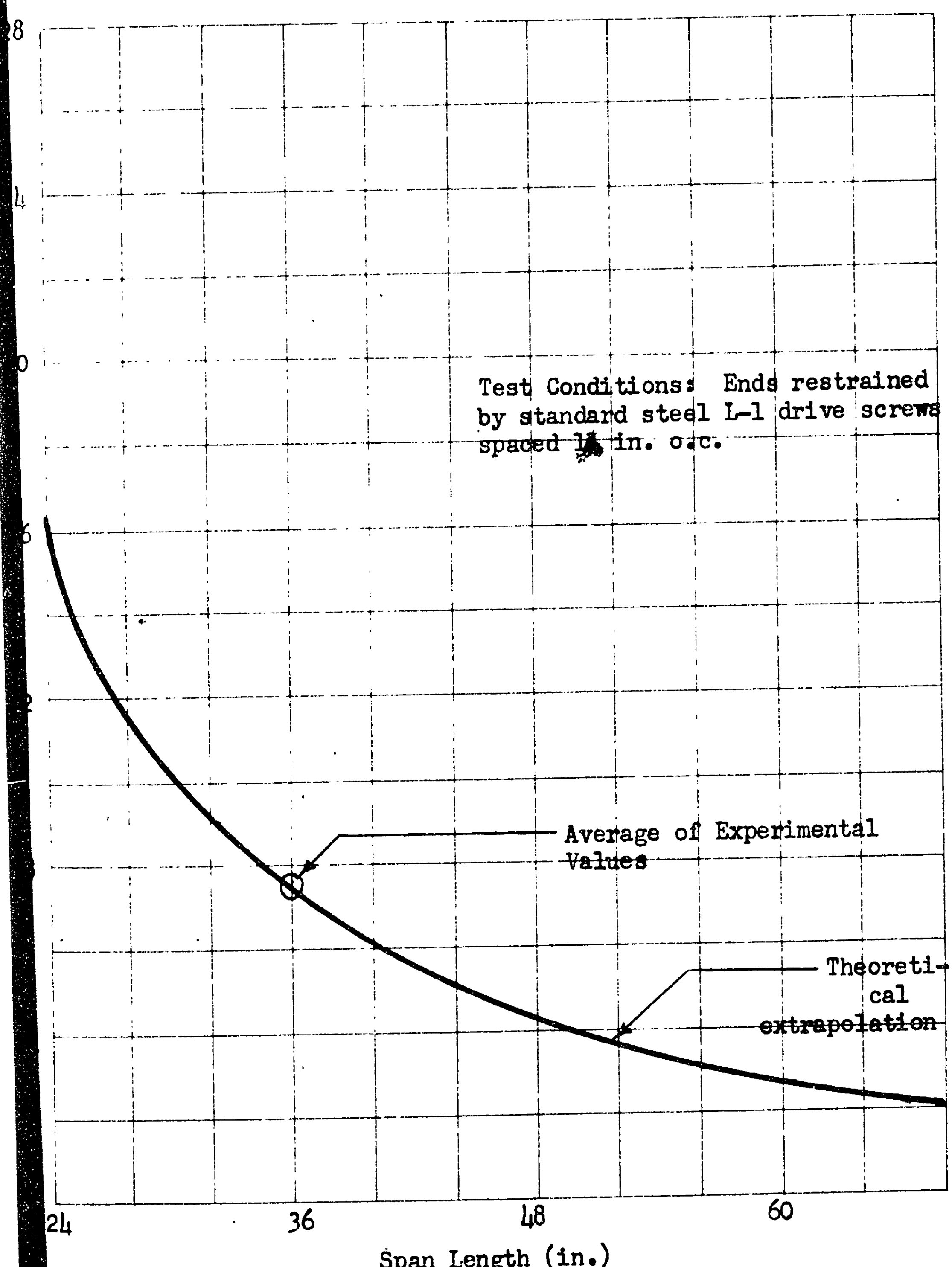


Figure 7.4 Uniformly Distributed Load to Cause Buckling vs Span Length for 18 Gauge Galbestos V-Beam Panels Under Static Loading.

(c) A curve of load to cause buckling vs span length for 18 gauge Galbestos V-beam panels attached to girts by vonventional means is presented in Fig. 7.4.

APPENDIX A

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APPENDIX B

SECONDARY TEST DATA

B.1 BRICK

(a) Rupture Tests were made on random sample bricks from each lot used. The tensile strength in bending (modulus of rupture) tests were made in the conventional manner according to the ASTM specification C 67-44. Modulus of rupture values obtained are tabulated below (only water-struck brick were tested):

Lot 1	Lot 2	Lot 3
1955 psi	1465 psi	990 psi
1900	1376	732
1358	1500	854
1367	1381	1349
1590	1421	1283
1319	1630	1421
1294	1698	1170
1173	1658	1475

(b) Compression tests were made on random sample bricks from each lot used. The compressive strength tests were made in the conventional manner according to the ASTM specification C 67-44. Compression strength values obtained are tabulated below:

Lot 1	Lot 2	Lot 3
10700 psi	9450 psi	6310 psi
12600	10280	6680
7700	9080	7410
5680	12280	7660
13230	10100	8340
9100	12440	9350
7900	11230	10160
6750	10870	7600

B.2 MORTAR

Mortar samples were taken and tested according to

ASTM specification C-91, from most test walls. The tensile strength in bending (modulus of rupture), compressive strength, and modulus of elasticity for mortars corresponding to various walls are given below:

Wall	Mod. Rup. (psi)	Comp. Str. (psi)	Mod. Elas. (psi)
3S1	123		
	148		
3S2	144		
	182		
6S1	89	440	
	152	620	
6S2	147	635	
	138	570	
	153	340	
		670	
6S3	250	1250	
	230	1003	
	215	1290	
	220	995	
6S4	235	1040	
	220	950	
	145	930	
	140		
12S1	256	1017	
	284	805	
	262	475	
	298	741	
	307	840	
	253	820	
12S2	302	1290	
	287	1205	
	251	1256	
	247	1338	
	253	980	
	215	770	
12S3	282	974	
	316	950	
	288	1243	
	336	1265	
	238	850	
	238	798	
LS1	306	1157	
	254	948	
	276	1063	
	284	1009	
	274	648	
	267	837	

Wall	Mod. Rup. (psi)	Comp. Str. (psi)	Mod. Elas. (psi)
LS1	241	549	
	210	320	
	271	450	
	243	773	
	136	759	
	248	821	
3-1	211	975	
	188	1000	
	218	1170	
	188	1170	
3-2	257	1050	
	267	1050	
3-3	211	1050	
	218	1050	
3-4	149	840	
	207	870	
3-5	124	722	
	152	610	
	126	713	
	187		
6-1	196	885	
	214	821	
6-2	141	647	
	210	527	
6-3			
6-4	115	739	
	134	668	
	126	834	
	130	775	
6-5	122	699	
	130	761	
	130	614	
	130	653	
6-6	300	1025	
	268	1060	
	193	613	
	238	740	
8-1	141	785	
	173	665	
	184	680	
	162	565	
	221	595	
	115	600	
	120		
8-2	239	626	
	163	899	
	190	860	
	196	1139	

Wall	Mod. Rup. (psi)	Comp. Str. (psi)	Mod. Elas. (psi)
8-3	274	1272	
	250	1050	
	291		
	248		
12-1	207	785	
	227	788	
	200	1100	
	214	593	
	219	616	
	210	893	
12-2	213	940	
	195	1004	
	115	297	
	176	470	
		382	
		391	
12-3	207	635	
	230	635	
	242	725	
	207	767	
	259	775	
	259	525	
BS-1	153	820	
	145	850	
	100	820	
	111	750	
BS-2	111	590	
	111	470	
	115	610	
	100	570	
BS-3	138	910	
		850	
		1050	
		1060	
BS-4	200	350	
	227	625	
	207	510	
	184	538	
BS-5	188	540	
	184	390	
	198	623	
	200	630	
BS-6	274	1110	
	308	1235	
	312	1180	
	245	1210	
BS-7	168	368	
	192	492	
	168	470	
	190	443	

Wall	Mod. Rup. (psi)	Comp. Str. (psi)	Mod. Elas. (psi)
AS-1	390	2450	
	390	2400	
	470	2050	
	440	2100	
AS-2	360	2000	
	370	2000	
	370	1900	
	400	1800	
AS-3	458	1920	
	377	1690	
	346	1490	
	430	1485	
AD-1	520	1680	
	545	2080	
	384	1620	
	443	1200	
AD-2	407	1920	
	353	1890	
	300	1850	
	352	1730	
AD-3	570	1910	
	445	1980	
	400	1690	
	375	1740	
AD-4	475	1740	
	382	2340	
	360	2310	
		2310	
AD-5	396	2020	
	404	1900	
	433	1630	
	400	1585	
AD-6	505	2600	
	515	2850	
	566	3000	
	490	3100	
RS-1	192	840	
	161	770	
	207	1090	
		1290	
RS-2	256	1040	
	137	937	
	256	585	
	213	730	
RS-3	473	2195	1.4×10^6
	421	2240	1.1×10^6
	430	1615	1.7×10^6
	497	1980	1.3×10^6

Wall	Mod. Rup. (psi)	Comp. Str. (psi)	Mod. Elas. (psi)
RS-4	498	2030	
	453	2060	
	514	2240	
	505	1965	
RS-5	552	1970	
	513	2040	
	582	1880	
	522	2880	
R4S-1	547	1862	
	382	1170	
	312	1870	
	300	1090	
R4S-2	418	842	
	458	900	
	361	873	
	374	1265	
R4S-3	322	914	
	309	1560	
	100	1082	
	173	1560	
R4S-4	520	1680	1.13×10^6
	484	1416	0.85×10^6
	619	880	0.41×10^6
	285	1680	1.42×10^6
R4D-1	399	2250	
	390	2180	
	395	2150	
	375	2100	
R4D-2	546	2100	1.58×10^6
	572	2115	1.42×10^6
	406	1450	1.21×10^6
	510	1471	1.26×10^6
6R-1	397	1670	
	352	1750	
	362	1560	
	377	1520	
6R-2	475	1487	1.42×10^6
	514	1198	1.36×10^6
	500	1680	2.12×10^6
	382		
6R-3	432	1445	1.22×10^6
	442	1855	1.62×10^6
	420	1898	1.41×10^6
	465	1317	1.22×10^6
8R-1	280	1690	
	314	1685	
	528	2120	
	525	2170	

Wall	Mod. Rup. (psi)	Comp. Str. (psi)	Mod. Elas. (psi)
12R-1	528	2100	
	582	2280	
	527	3210	
	544	3250	
	537	2940	
	604	2725	
12R-2	510	1770	2.29×10^6
	635	1830	1.70×10^6
	520	1910	2.29×10^6
	468	2310	1.89×10^6
	537	2170	1.62×10^6
	400	1720	1.70×10^6
LS-2	557	2270	
	529	1990	
	383	1850	
	390	1450	
	446	2010	
	421	2300	
	628	2080	
	552	2125	
	402	1910	
	352	1840	
	582	1880	
	536	1890	
RGS-1	310	1445	
	247	1125	
	135	1120	
	316	1520	
RGS-2	382	1416	
	342	900	
	304	955	
	306	1060	
RGS-3	294	1345	0.85×10^6
	280	1195	1.22×10^6
	286	1450	1.22×10^6
	328	1790	1.13×10^6
RGS4-1	228	685	
	256	650	
	163	926	
	201	930	
RGS4-2	205	658	
	224	586	
	195	636	
	225	644	
RGS4-3	306	628	0.64×10^6
	274	609	0.73×10^6
	208	450	0.71×10^6
	274	438	0.71×10^6

Wall	Mod. Rup. (psi)	Comp. Str. (psi)	Mod. Elas. (psi)
MGS4-1	242	658	
	233	790	
	227	1235	
	239	1260	
MGS4-2	294	1230	
	267	1248	
	304	765	
	270	972	
RGD4-1	160	522	
	201	532	
	216	595	
	212	617	
RGD4-2	175		
	162		
RGD4-3	274	1118	
	253	1090	
	279	910	
	241	883	
6RG-1	207	727	
	247	547	
		650	
		413	
6RG-2	239	666	
	165	560	
	163	386	
	144	296	
6RG-3	230	1072	0.95×10^6
	228	1062	1.13×10^6
	159	1050	0.95×10^6
	225	595	2.83×10^6
12RG-1	247	685	
	208	730	
	219	535	
	168	635	
		656	
		578	
12RG-2	182	942	
	201	920	
	207	808	
	196	785	
	222	1126	
	193	1120	
12RG-3	230	912	
	195	990	
	170	1460	
		1365	
CoBS-3	305	1210	
	282	1053	
	284	1163	
	271	1065	

Wall	Mod. Rup. (psi)	Comp. Str. (psi)	Mod. Elas. (psi)
CoBS-4	276	1090	
	258	1070	
	276	1133	
	290	1140	
CoBS-5	190	705	
	170	821	
	205	879	
	196	895	
CoBS-6	155	718	
	156	870	
	141	582	
	139	700	
PTS-2	190	935	
		1065	
PTS-4	259	1273	
	251	1262	
Pier 1		382	
		340	
		575	
Pier 2		745	
		695	
		565	
Pier 3		550	
		580	
		382	
Pier 4		455	
		358	
		395	
Pier 5		467	
		465	
		360	
Pier 6		405	
		435	
		425	
Pier 7		497	
		526	
Pier 8		716	
		708	
Pier 9		915	
		895	
Pier 10		2410	
		2430	
Pier 11		2240	
		2150	
Pier 12		1560	
		1820	
S_1	192	915	
	138	1120	
	176	910	
	173	650	

Wall	Mod. Rup. (psi)	Comp. Str. (psi)	Mod. Elas. (psi)
S_2	233	1032	
	267	1090	
	230	894	
	222	938	
T_1	252	845	
	245	913	
	334		
	316		
T_2	245	429	
	195	697	
	259	912	
	222	938	

B.3 BRICK-MORTAR BOND

Brick-mortar bond tests were made on random brick samples laid up with mortar. (See Section 4.1.2 of the August 1952 report) The tensile strength in bending (modulus of rupture) and the failure load (P) for the shear test divided by the mortar area (A) are tabulated below. (Note that the stress P/A can be resolved to pure shear by use of Mohr's Circle). 1:1:5 mortar (by volume) of Portland cement, lime, and sand was used.

	Modulus of Rupture	P/A
Lot 1.	101 psi	72 psi
	91	52
	109	100
	101	73
	104	64
	85	88
	122	78
	130	99
	96	89
	85	86
	96	49
	90	74
Lot 2.	162 psi	106 psi
	109	130
	112	87
	106	105
	110	121
	149	95

Modulus of Rupture	P/A
85 psi	98 psi
113	141
96	114
83	101
96	176
	86

SR-4 strain gauges were applied to the mortar joint of some specimens of Lot 2 tested for brick-mortar bond strength. Readings obtained from the gauges during testing were used to obtain an average curve of strain vs bending stress, as shown in Fig. B.1.

B.4 MISCELLANEOUS

(a) The yield strengths of random samples of reinforcing bars used in the reinforced masonry and reinforced gunited masonry specimens were obtained by tension tests conducted in the manner prescribed by ASTM specifications E8-46 and A15. All bars used were of intermediate grade billet steel having standard high-bond deformations and conforming to ASTM specifications A15 and A305. The results of the tension tests for yield strength are tabulated below:

Lot 1	Lot 2
50,500 psi	59,100 psi
51,000	58,100
51,000	60,100

(b) Compression tests were made on random samples of hollow concrete block. Tests were made in the conventional manner according to ASTM specification C140-39. Compressive strengths in psi obtained from the tests are tabulated below (values are based on net area):

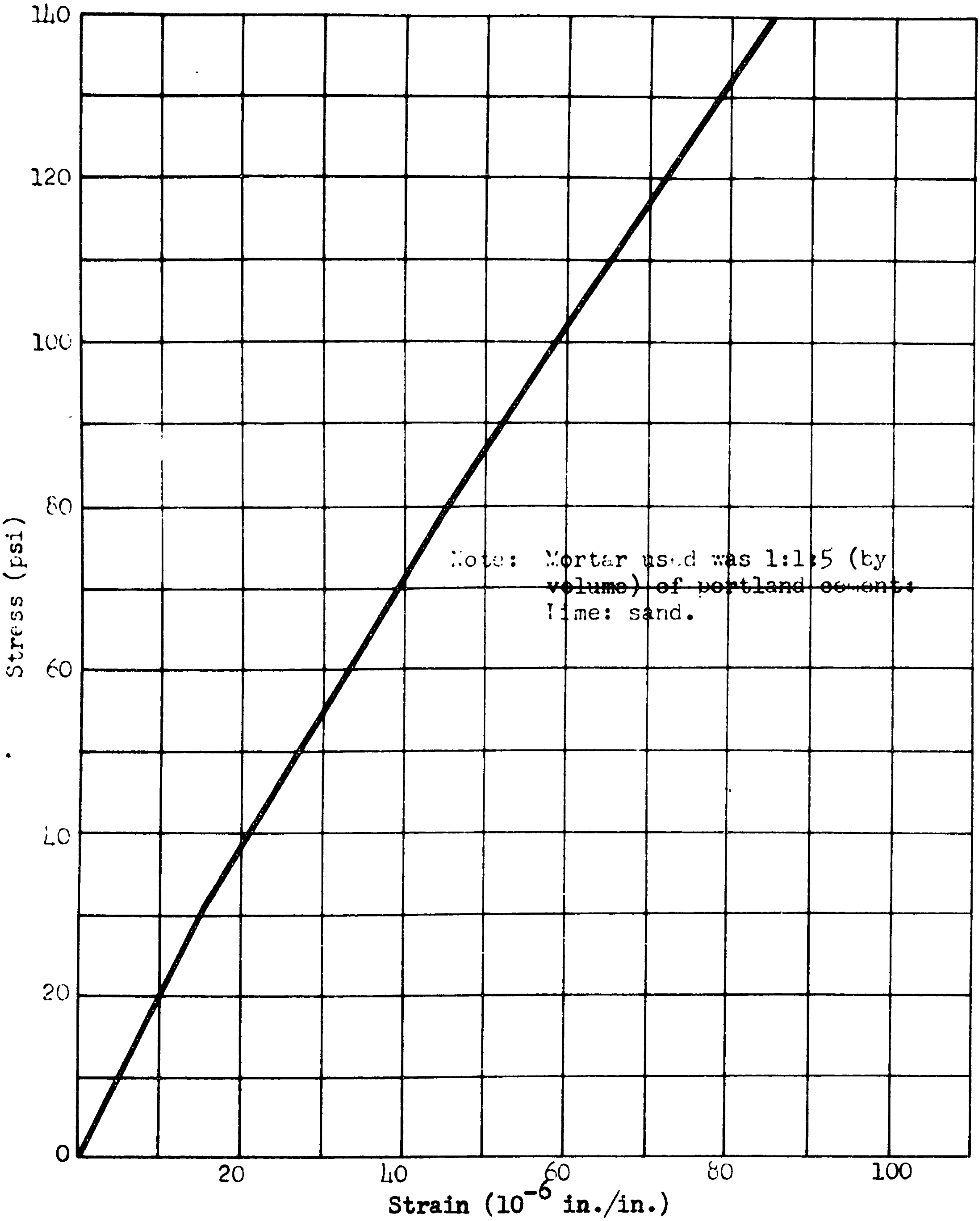


Figure B.1 Average Curve of Bending Stress vs Strain of Mortar Joint in Brick Mortar Bond Specimens.

3,220	psi
2,670	
3,000	
2,760	

(c) Compression tests were made on random samples of hollow clay partition tile block. Tests were made in the usual fashion according to ASTM specification C112-36. Compressive strengths, in psi, based on net area, are tabulated below:

4,420	psi
8,360	
2,450	
5,300	

(d) Gunite samples were taken and tested for tensile strength in bending (modulus of rupture) and compressive strength. The modulus of rupture was obtained by testing, in flexure, gunite beams $2\frac{1}{2}$" x $2\frac{1}{2}$" x 10" on a span length of 8 inches. The compressive strength was obtained by testing, under a compressive load, capped cylinders of gunite 6" x 12" high. Results of the tests are given below:

Modulus of Rupture	Compressive Strength
880 psi	2890 psi
945	5000
805	4460
790	3110
875	5320
875	3600
860	4360
812	4450

B.5 BRICK MASONRY COMPRESSION PIERS

In order to obtain a value for masonry compression strength, a number of brick piers, consisting of several brick and mortar joints were constructed as recommended by previous investigators (7) (see Fig. B.2). The piers were

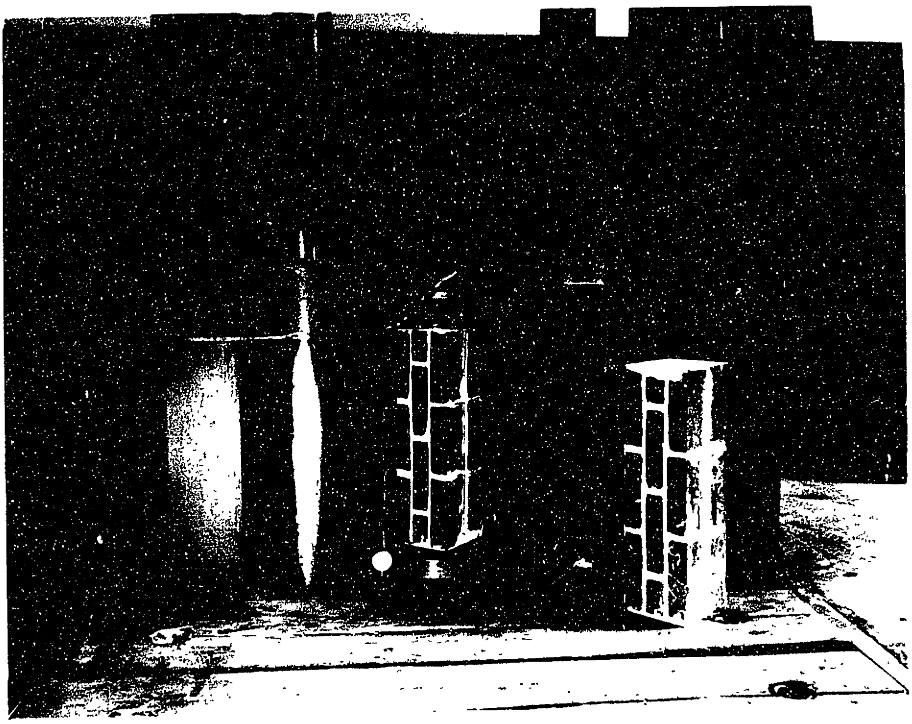


Figure B.2 Brick Masonry Compression Piers capped with plaster at the ends and tested to failure under a compressive load in a Southwark-Emery hydraulic testing machine. By using Ames deflection gauges, deflection readings of the piers were taken during the testing operation. These readings were used to obtain strain values which were average values over the entire pier height. SR-4 strain gauges were applied to the midheight mortar joints of some of the piers, and were used to obtain the strains of the mortar joints as the load was applied. Average curves of both types of strains mentioned above vs applied stress for two types of piers are presented in Figs. B.3 and B.4.

The values tabulated below are the ultimate compressive stresses of the piers:

(a) Piers consisting of medium-burned waterstruck brick and 1:1:5 mortar (by volume) of Portland cement, lime, and sand. Pier dimensions were $7\frac{1}{2}$" x 7" x 25" high.

Lot 1	Lot 2
2380 psi	2800 psi
2230	3800
2430	2860
2420	
2490	
1970	

(b) Piers consisting of medium-burned waterstruck brick and $1:\frac{1}{4}:3$ mortar (by volume) of Portland cement, lime, and sand. Pier dimensions were 7 5/8" x 7 1/8" x 24" high.

5230 psi
4050
3580

The mode of failure of the brick piers was as follows:

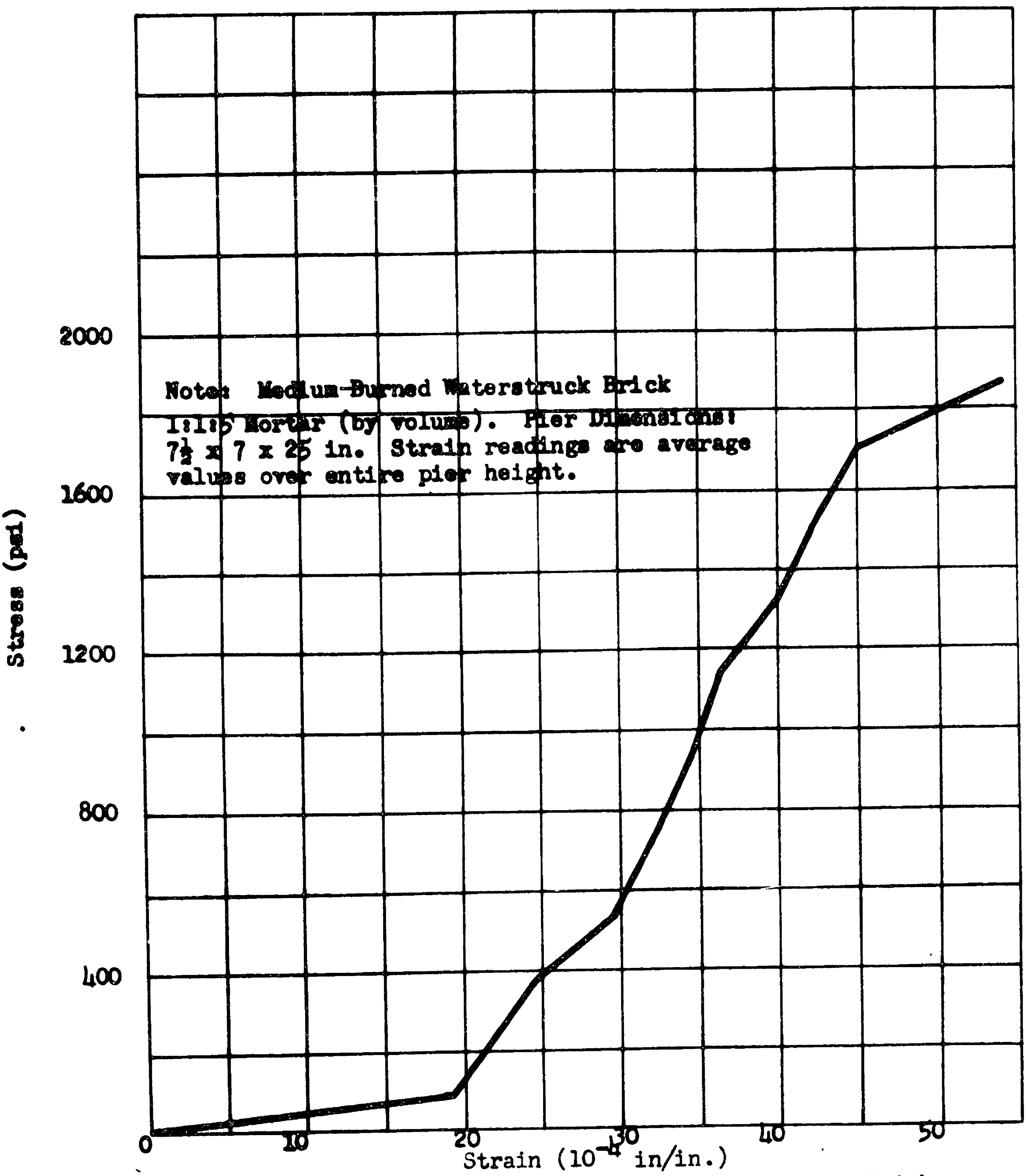


Figure B.3 Average Curve of Stress vs Strain of Brick Masonry Piers under Compression Load.

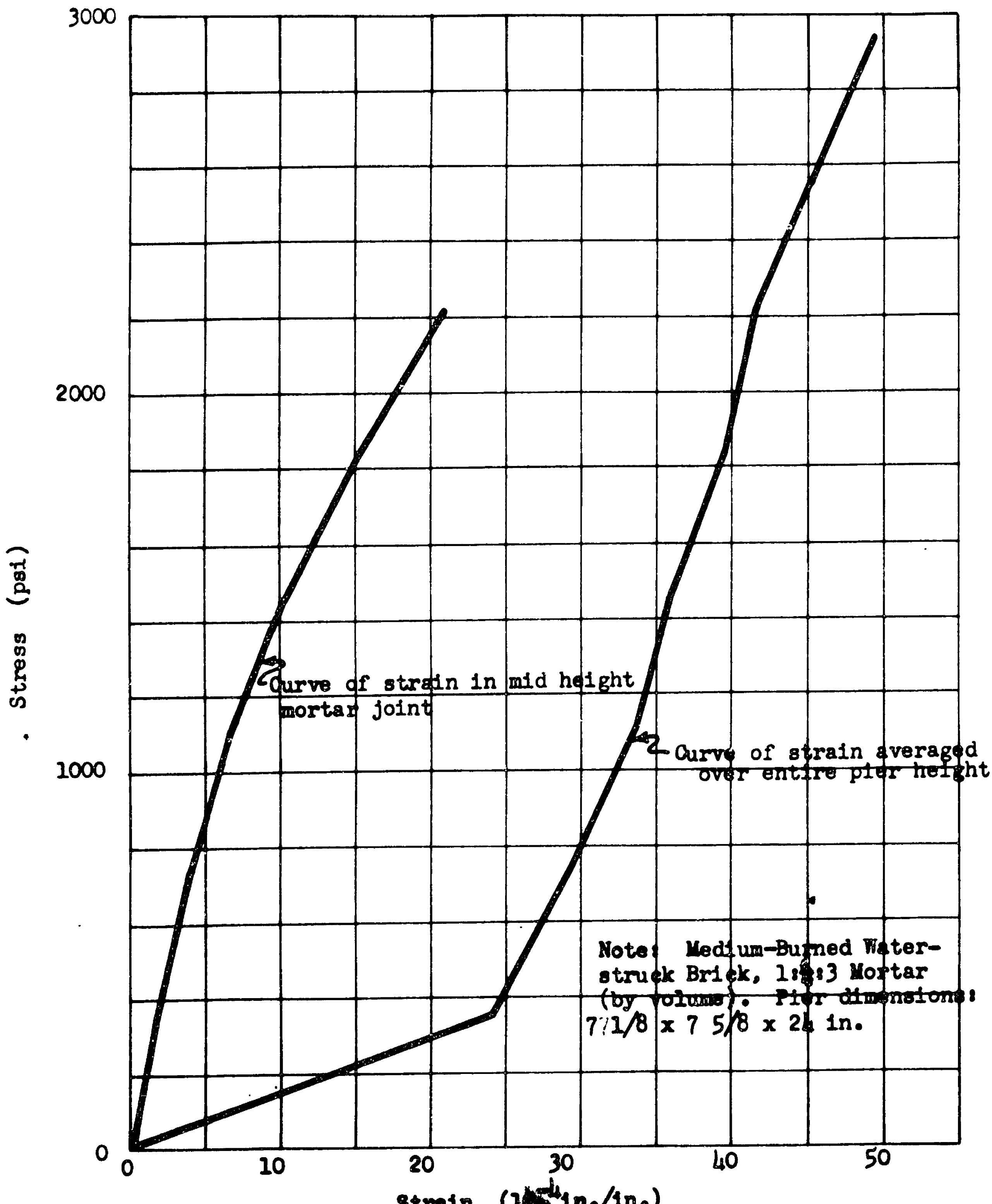


Figure B.4 Average Curves of Stress vs Strain of Brick Masonry Piers Under Compressive Load

As the compressive load approached its ultimate value, pieces of brick broke off the piers, and cracks appeared in the vertical brick masonry joints. Splintering of the brick and vertical cracking increased as the loading continued, until some of the vertical brick courses separated from the pier and buckled out at the ultimate load. Failure was sudden and complete, with cracks appearing in most vertical brick-mortar joints, and with compression failure evident in many of the horizontal joints in the pier.

APPENDIX C

SAMPLE COMPUTATIONS

C.1 UNIFORMLY-DISTRIBUTED ULTIMATE LOAD

The uniformly-distributed load which will cause the same midspan moment as the ultimate third point loading may be computed as follows (computations shown apply to beam 6S4):

Data: Span length = L = 72 in.

Width of beam = 18 in.

Ultimate load = P = 3.3 kips (live load only)

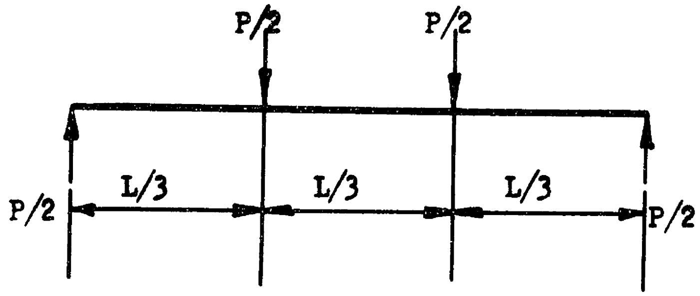


Figure C.1 Third Point Loading

Failure moment (see Fig. C.1) = $(P/2)(L/3) = (3.3/2)24 = 39.6$ in.-kips

Failure moment per unit width = $39.6/18 = 2.2$ in.-kips/in.

Equivalent uniformly-distributed load to cause failure moment = $w = 8M/L^2$

$$= \frac{8(2.2)}{(72)^2}(1000) = 3.4 \text{ psi}$$

Dead weight = <u>0.6 psi</u> (found experimentally)

Uniformly-distributed ultimate load = 4.0 psi

C.2 MODULUS OF RUPTURE

The modulus of rupture as used in this report is actually a modulus of rupture in bending. It is a fictitious stress useful in comparing the ultimate bending strengths of members of varying sizes and materials. The magnitude of this stress is obtained from the fundamental formula:

$$s = \frac{M}{Z}$$

where, s = stress

M = bending moment

Z = section modulus

If M is the ultimate bending moment sustained by the member, then s is the modulus of rupture in bending. The value of s is not a true stress because the equation is valid only when stresses do not exceed the proportional limit; however, it serves as a convenient measure of the bending strengths of members.

Determination of the modulus of rupture under static test conditions is illustrated below for beam 3S1.

Data: Span length = L = 36 in.

Thickness = t = 8 in.

Ultimate load = w = 15.1 psi (incl DL)

Ultimate bending moment $= M = \frac{wL^2}{8} = \frac{15.1(36)^2}{8}$

$= 2450$ in.-lbs/in.

Section modulus $= Z = t^2/6 = 8^2/6 = 10.65$ in.^{2}

Modulus of rupture $= s = MR = M/Z = 2450/10.65$

$= 230$ psi

Under dynamic loads the modulus of rupture is calculated as above, based on the ultimate bending moment sustained by the section, and then multiplied by the dynamic load factor acting.

Determination of the modulus of rupture under dynamic test conditions is illustrated below for panel D1.

Data: Span length = $L = 36$ in.

Thickness = $t = 8$ in.

Ultimate load = $w = 10\frac{1}{2}$ psi (incl DL)

Dynamic Load Factor = 1.6 (determined experimentally)

Ultimate bending moment = $M = wL^2/8 = 10.5(36)^2/8$

$= 1700$ in.-lbs/in.

Section modulus = $Z = t^2/6 = 8^2/6 = 10.65$ in.^{2}

Modulus of rupture = $s = MR = 1.6(M/Z) =$

$$\frac{1.6(1700)}{10.65} = 260 \text{ psi}$$

C.3 DEAD AND LIVE LOAD BEARING ON WALL SLAB OF TYPICAL MASONRY BUILDING

A. One-Story Building

The following computations were made in order to find the maximum dead and live load that bears upon a section of 8 in. masonry wall in a typical one-story building. (See Fig. C.2)

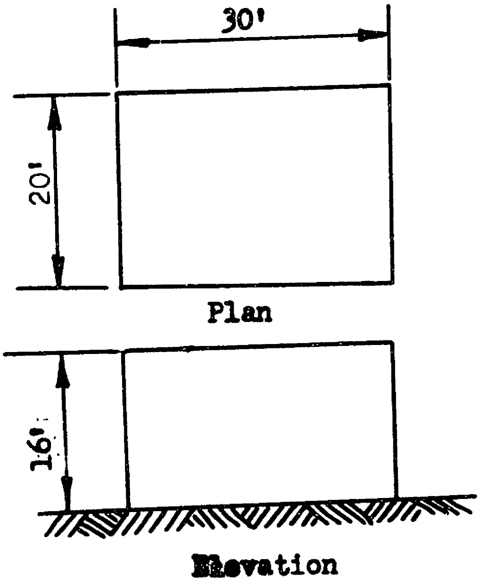


Figure C.2 One-Story Wall Bearing Building

1. Roof Load:

Dead Load: 10 lbs/ft^2

Snow Load: $\underline{30\ lbs/ft^2}$

40 lbs/ft^2

Total Load = 20(30)40 = 24,000 lbs

Load per ft of bearing wall = 24000/100

= 240 lbs/ft

2. Wall Load:

Dead Load = 81 lbs/ft^2 (for 8 in. wall)

Height above section considered = 13 ft

Load per ft of section = 81(13) = 1.052 lbs/ft

Total load per ft of section = 1052 + 240 =

1292/lbs/ft

B. <u>Two-Story Building</u>

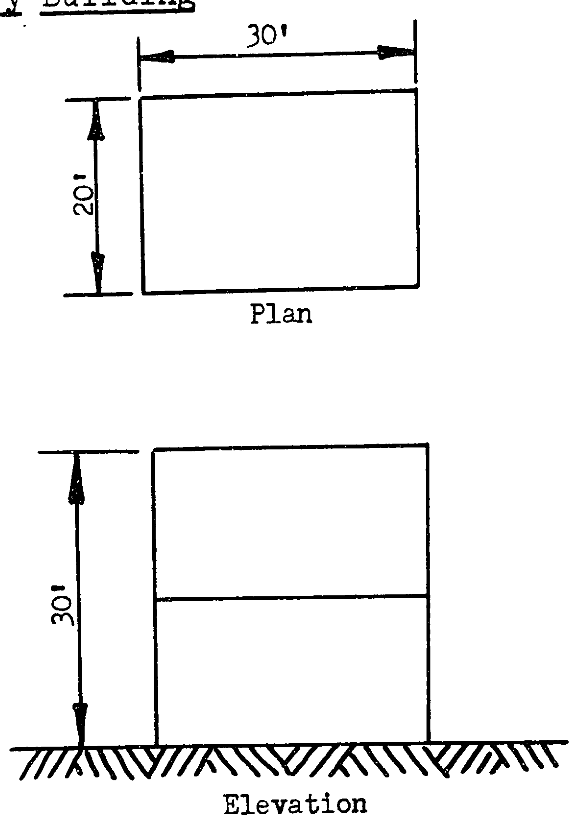


Figure C.3 Two-Story Wall Bearing Building

1. Roof Load:

Same as in A 240 lbs/ft

2. Inter-floor Load:

Dead load (5" thick at 12.5 lbs/ft^2/in. of thickness)

$= 12.5(5)20(30) = 37{,}5000$ lbs

Dead load per ft of bearing wall $= \frac{37{,}500}{100}$

$= 375$ lbs/ft

Live load $= 300$ lbs/ft^2

Total live load $= 300(30)20 = 180{,}000$ lbs

Live load per ft of bearing wall $= \frac{180000}{100} =$ 1800 lbs/ft

3. Wall Load:

Dead load $= 81$ lbs/ft^2

Height above section considered = 27 ft

Wall load per ft of section $= 27(81) = 2{,}185$ lbs/ft

Total load per ft of section $= 240 + 375 + 1800 + 2185$

$= 4{,}600$ lbs/ft

C.4 RELATIONSHIP OF LOAD VS SPAN LENGTH FOR 18 GAUGE GALBESTOS V-BEAM PANELS

A. Buckling Load

Assuming that buckling of the corrugations will occur in corrugated metal siding panels at some critical stress which is a function of corrugation pitch and depth and may be found experimentally, and that this stress corresponds to a critical moment for any one particular type siding, we may find the relationship between span length and theoretical buckling load as follows:

M_b = critical midspan moment at which buckling occurs for a specific metal siding.

w_b = uniformly distributed load at which buckling occurs.

L = span length.

$$M_b = C_1 w_b L^2$$

$$w_b = (1/C_1)(M_b/L^2)$$

Therefore,

$$w_b = C_2/L^2 \text{ ; since } M_b \text{ is constant.}$$

The curve shown in Fig. 7.4 was obtained from this relationship.

APPENDIX D

MISCELLANEOUS

D.1 ANALYSIS OF SLAB FAILURE

Included in the August 1952 report on pages 18-21 is an analysis of the theoretical strength of a brick wall loaded transversely which assumed that: (a) the brick is sufficiently strong so that all failures will occur through the brick-mortar joints at midspan, and (b) that the failure consists of transverse rupture at the joint in each course, and torsional shear failure on the face of one-half of two bricks between each course; that these two effects occur simultaneously and that the addition of the two effects results in the uniform static strength of the wall. Carrying out the analysis based on the assumptions mentioned above, and known strength properties of mortar-bond in rupture and shear, resulted in a value of 9.1 psi for the uniformly distributed load attributable to pure torsional shear, and in a value of 6.3 psi for the uniformly distributed load attributable to rupture in the transverse mortar joints. Adding these two values resulted in a total uniform static strength of 15.4 psi, which seemed to correspond closely to the observed uniform failure load of the wall slabs.

Four slabs were built in order to check the assumptions upon which the analysis in the preceding paragraph was based. These slabs were standard 36 in. span walls 2 wythes (8 in.) thick of American bond, laid in 1:1:5 mortar.

In 2 slabs, all the vertical (rupture) joints at midspan were omitted in order to isolate the effect of torsional shear between courses during failure. The remaining two slabs were constructed with all the horizontal joints at midspan omitted in order to isolate the effects of pure rupture between courses during failure. These slabs were statically tested with a uniformly-distributed load. Results of these tests are tabulated in Table D.1. The test results of the four slabs, presented in this table, show that the uniformly-distributed load required to cause only torsional shear failure averaged 14.0 psi, and that the uniformly-distributed load to cause only rupture failure in the transverse joints averaged 8.1 psi.

As a result, it may be concluded that: (a) the effects of shear and rupture are not additive, as their maximum values are not attained simultaneously, and (b) the two effects, especially that of shear, are larger than predicted. On the basis of these conclusions, the analysis presented in the August 1952 report is not correct.

D.2 END ROTATION OF FIXED-END BEAMS

Two Ames Dial deflection gauges were placed at each end of the beams tested in support condition (3), one near the top face and the other near the bottom face of the beam. (See Fig. D.1) Deflection readings were taken as the transverse static third-point loading was applied. The rotation of the beam ends was then obtained by adding the deflection shown by the top and bottom dials, and dividing this value by the distance between the gauges, which was 8 in.

TABLE D.1

Test No.	Description	Sup-port	Loading	Mortar Avg Flex. (psi)	Mortar Avg Comp. (psi)	Ultimate Load (psi) (incl DL)	Computed Modulus of Rupture* (psi)
S1	3' square, 8" thick slab. Vertical (rupture) joints omitted at midspan.	(2)	Uniform Static	170	900	15.2	231
S2	Same as S1	(2)	Uniform Static	240	990	12.9	196
T1	3' square, 8" thick slab. All horizontal joints omitted at midspan.	(2)	Uniform Static	287	879	7.2	109
T2	Same as T1	(2)	Uniform Static	230	744	9.0	137

*For sample computations, see Appendix C.

Curves of end rotation vs applied midspan moment per unit width are presented in Fig. D.2.

D.3 <u>SCHEME FOR ACHIEVING SUPERIMPOSED DEAD WEIGHT IN 3 FOOT SQUARE PANELS</u>

In order to apply load on a panel which would successfully approximate the effect of dead weight bearing on the wall of an actual structure, the following requirements must be met in the laboratory:

(a) Equal and opposite pressures along the top and bottom of the wall (in a standing position) must be provided.

1. The pressure must be uniformly distributed

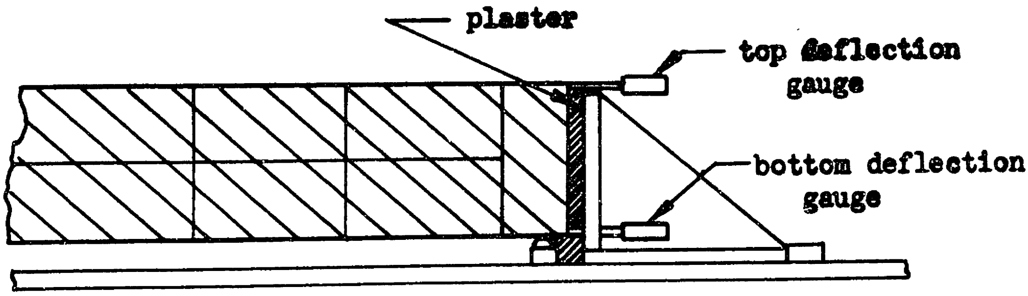


Figure D.1 Scheme Used to Obtain End Rotation Values of Fixed End Beam Under Transverse Static Loading

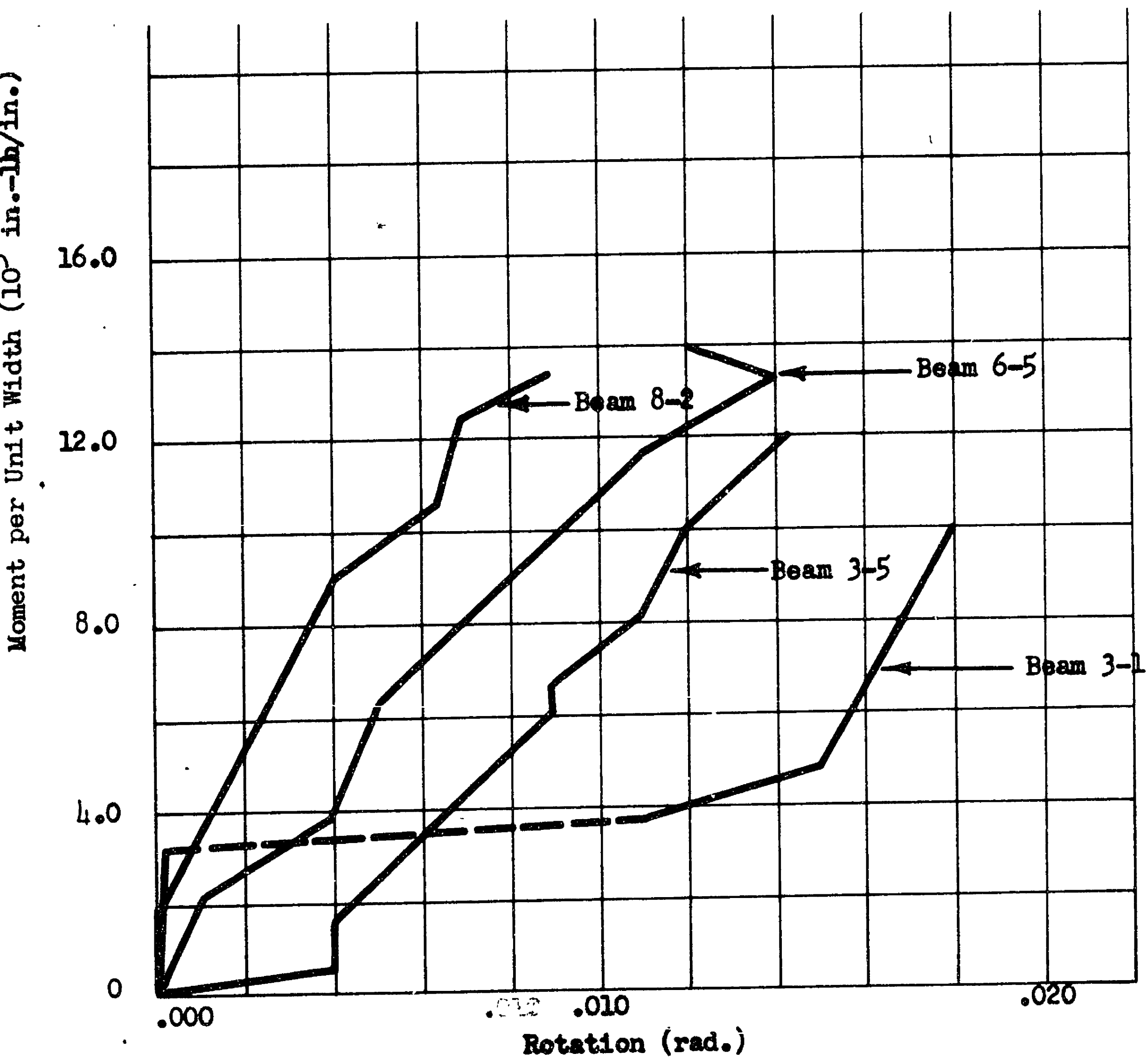


Figure D-2 End Rotation of Fixed-End Beams Under Third Point Loading vs Applied Unit Moment in two directions (width and thickness).

2. The pressure must be controllable and measurable.

3. The pressure must remain constant as the wall is tested to failure.

(b) The method of accomplishing the above listed objectives must not interfere with the testing of the wall panel, that is:

1. Normal deflection of the wall must not be restrained,

2. The method must not effect the magnitude or distribution of the transverse bending load on the wall,

3. The method must not interfere with normal end support conditions.

The method used to obtain the desired results was as follows:

(a) High strength rods of 0.20 in. diameter with ends threaded were lubricated with grease and covered with $\frac{1}{4}$ in. soda straws.

(b) The rods were supported in a vertical plane in a jig with horizontal spacing equal to one brick width (including the joint).

(c) The wall was laid up with one wythe of brick on each side of the plane of rods; the bricks for the two header courses occurring in the 3 ft wall were placed in the horizontal spaces between the rods.

(d) The wall was cured as usual, with the rods in the wall.

(e) After curing, a $\frac{1}{4}$ in. plate with a hole at its center was placed over each rod and plastered to the wall to insure even bearing. These plates were designed to provide sufficient clearance between each other so as to prevent restraint of the wall during deflection.

(f) One end of each rod was provided with a load measuring plug made by mounting a bridge of SR-4 strain gauges on $\frac{1}{2}$ in. tubing one inch long (see Fig. D.3). Each plug was calibrated in an Olsen mechanical testing machine; loads were measured by achieving balance on a type K strain indicator, reading the strain corresponding to a particular load for each plug.

(g) Finally, each plug end was capped with a suitable washer and each rod end was provided with a nut.

(h) The rods were stressed by tightening the nuts at the ends while reading the force by means of the load measuring plugs and the strain indicator.

(i) After the rods were stressed to the proper value and checked, the wall was placed in the slab machine and tested in the conventional manner.

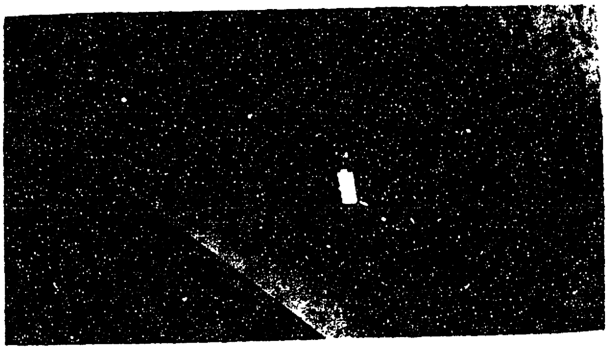
Figure D.4 presents two views of the wall at various stages in the process described above.

D.4 METHOD OF LAYING UP RBM SPECIMENS

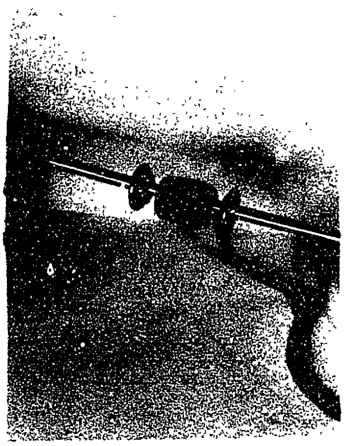
Preliminary preparations consisted of the following:

(a) Where designs required, reinforcing rods were bent by conventional methods.

(b) SR-4 strain gauges were mounted on particular



(a) Showing SR-4 Strain Gauge Attached



(b) Showing Washers

(Figure D.3 Load Measuring Plug

(a) Showing Stages of Preparation

(b) Stressing the Rods

Figure D.4 Wall for Superimposed Load Test rods using ordinary methods. A-3 gauges were used in all instances. The mounted gauges were wired and waterproofed by a 1/8 in. coating of cerese wax.

Laying of brick in courses generally was done in the usual manner, course by course. A bed of mortar was laid the length of the specimen on a completed course which would receive a reinforcing rod in the next joint. The reinforcing rod was set in this mortar bed and pressed in firmly by hand taking particular care to achieve the specified d. (See Fig. D.5) The rod was covered with a thin layer of mortar. The next course of brick was laid directly and gently tapped down to the desired mortar joint thickness. Interior mortar joints were filled in the usual manner and the course completed. The operation was repeated as specified.

D.5 METHOD OF FABRICATION OF REINFORCED GUNITE SPECIMENS

The unreinforced panels were prepared in the usual manner and cured dry for at least seven days. They were placed vertically ready to receive the reinforcing steel.

The reinforcing rods were laid out on the ground, spaced as required and tied to form a stable grid. The grid was then laid up against the brick wall and held securely at the desired distance from the wall face by wiring to nails or "drive-it" pins set in the mortar joints. Where steel wire mesh was used, it was attached in a similar manner.

The four edges of each specimen were provided with

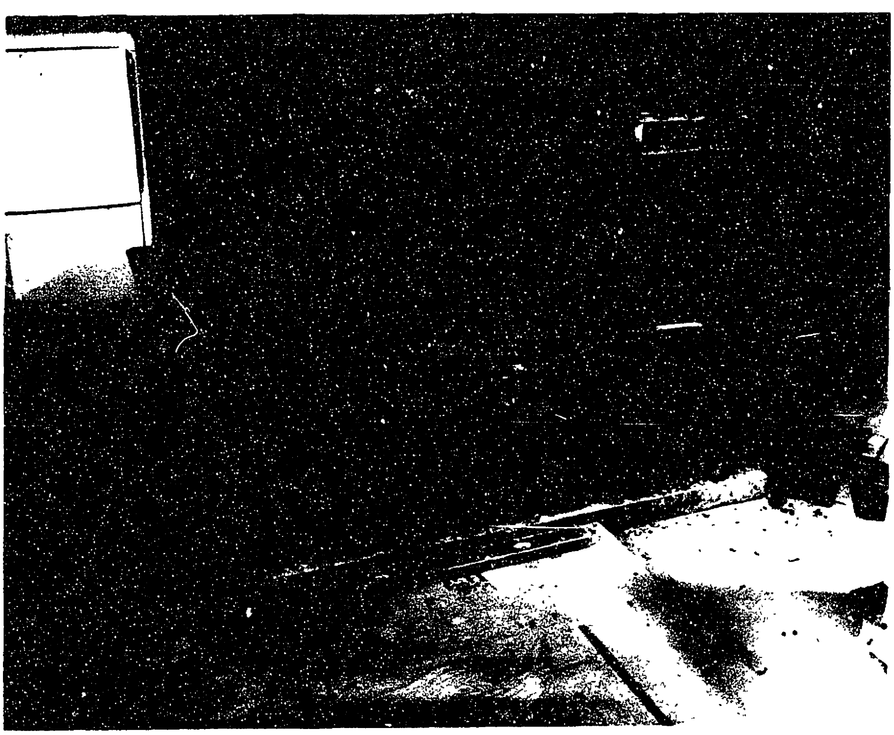


Figure D.5 RBM Beam Under Construction Showing Reinforcing Rod provided with screed boards placed to achieve the desired thickness of gunite. Figs. D.6 and D.7 show specimens at this stage ready for the gunite application.

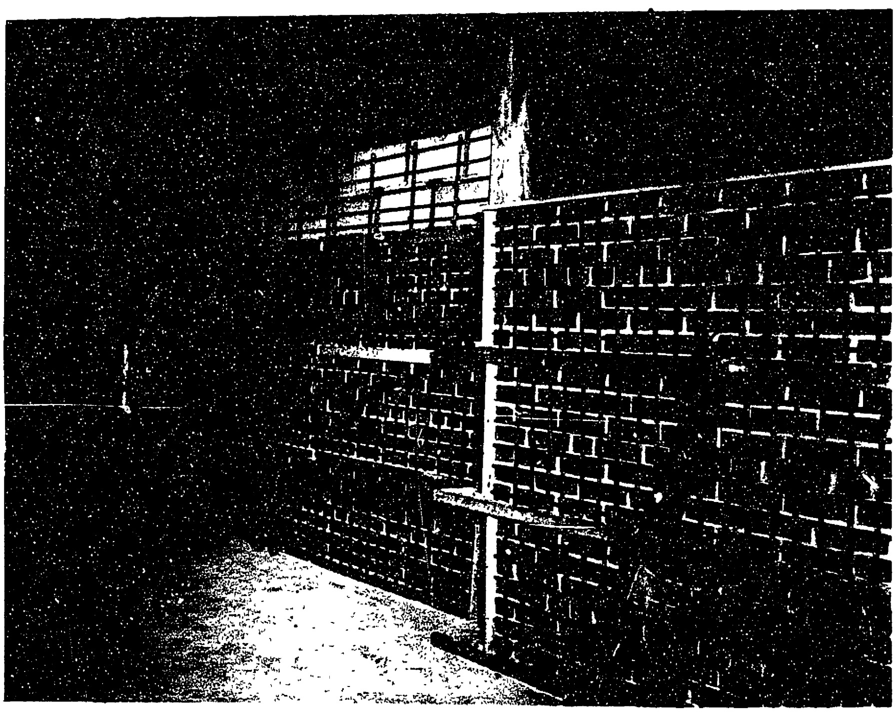
Before application of the gunite mix, the specimens were sprayed with a sand and water blast applied by the guniting apparatus. This sand blast cleaned and removed loose particles in order to help achieve a good masonry-gunite bond. The gunite was applied and screeded to the desired thickness by the contractor. The gunite surface was not faced or floated.

The specimens were cured under wet burlap for seven days. Screed boards were then removed and the specimens cured dry 19 to 23 days before testing.

D.6 REINFORCED GUNITE SLABS

Four reinforced gunite slabs were tested in the slab testing machine by applying uniformly-distributed static load. A description of the slabs and test conditions, a tabulation of test results, and curves of transversely applied unit bending moment vs midspan deflection are presented below. Failure in all cases was due to bending moment. Slab GS-1 after failure is shown in Fig. D.8.

(a) Panels



(b) Beams

Figure D.6 Masonry Specimens Ready for Gunite Application

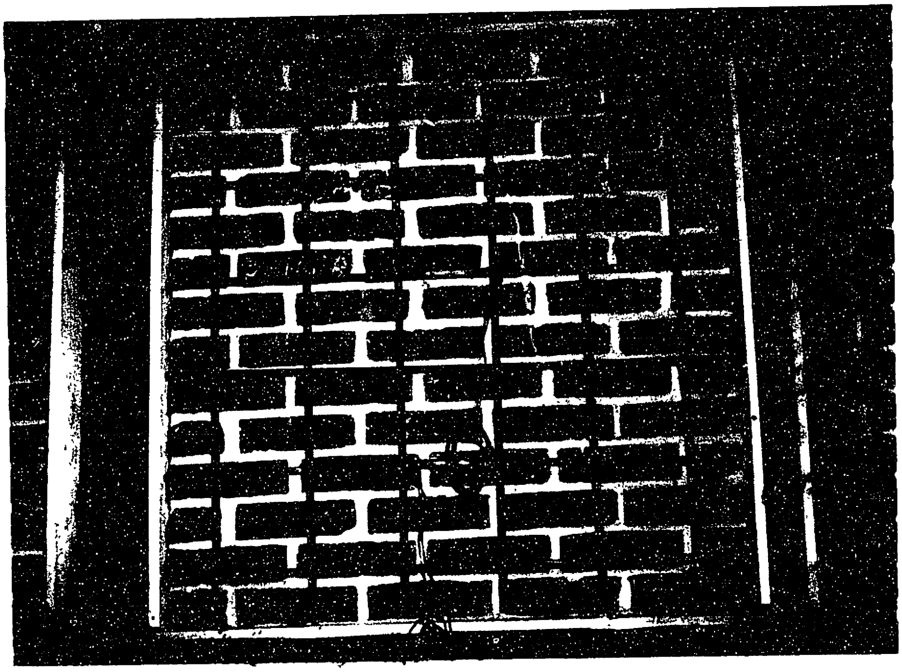


Figure D.7 Close-up of Panel Ready for Gunite Application, Showing SR-4 Strain Gauges on Reinforcing Rods

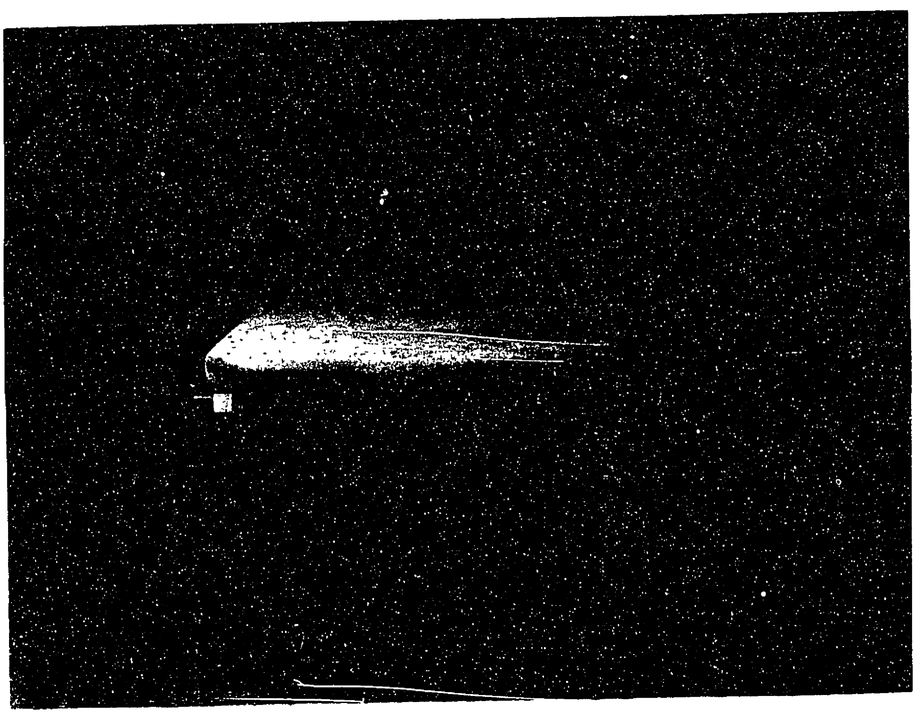


Figure D.8 Reinforced Gunite Slab After Failure

TABLE D.2

Description of Tests on Gunite Slabs

(1:4 Gunite, Int. Grade Standard Deformed* Steel)

Test No.	Gunite Thickness (in.)	Span Length (in.)	d (in.) **	b (in.) **	p (%) **	Steel Arrangement***
GS-1	3	36	$2\frac{1}{2}$	35	1.10	3/8"Ø rods at 4" O.C.
GS-2	3	36	$2\frac{1}{2}$	35	1.10	3/8"Ø rods at 4" O.C.
G2S-1	2	36	$1\frac{1}{2}$	35	1.83	3/8"Ø rods at 4" O.C.
G2S-2	2	36	$1\frac{1}{2}$	35	1.83	3/8"Ø rods at 4" O.C.

*As specified in ASTM Specification A305

**See Figure 6.8

***All steel rods straight, all reinforcement 2-way.

TABLE D.3

Test Results on Gunite Slabs

Test No.	Ultimate Load		Type Failure	Midspan Defl. at Plastic Load**
	psi (incl DL)	Moment* (in.-kips/in.)		
GS-1	20.3	3.3	BM	0.190
GS-2	22.3	3.6	BM	0.165
G2S-1	11.2	1.8	BM	0.320
G2S-2	10.2	1.7	BM	0.390

Steel rod yield stress = 59,000 psi

*For sample calculations, see Appendix C

**Deflection values are from the dead load position.

Curves of transverse bending moment vs midspan deflection are presented in Fig. D.9.

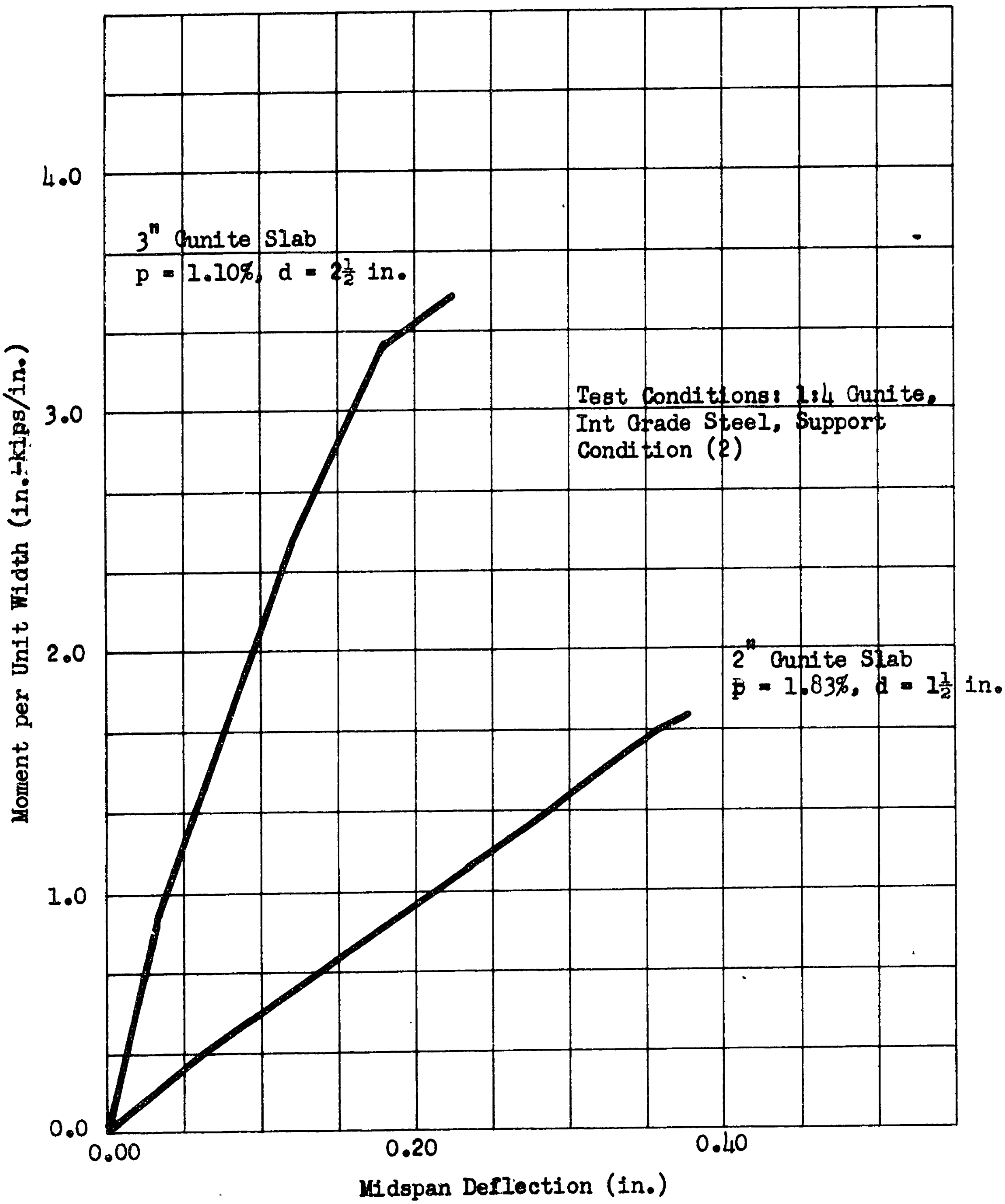


Figure D-9 Bending Moment per Unit Width vs Midspan Deflection for Uniformly Distributed Static Load on Reinforced Gunite Slabs. Span = 36 in.