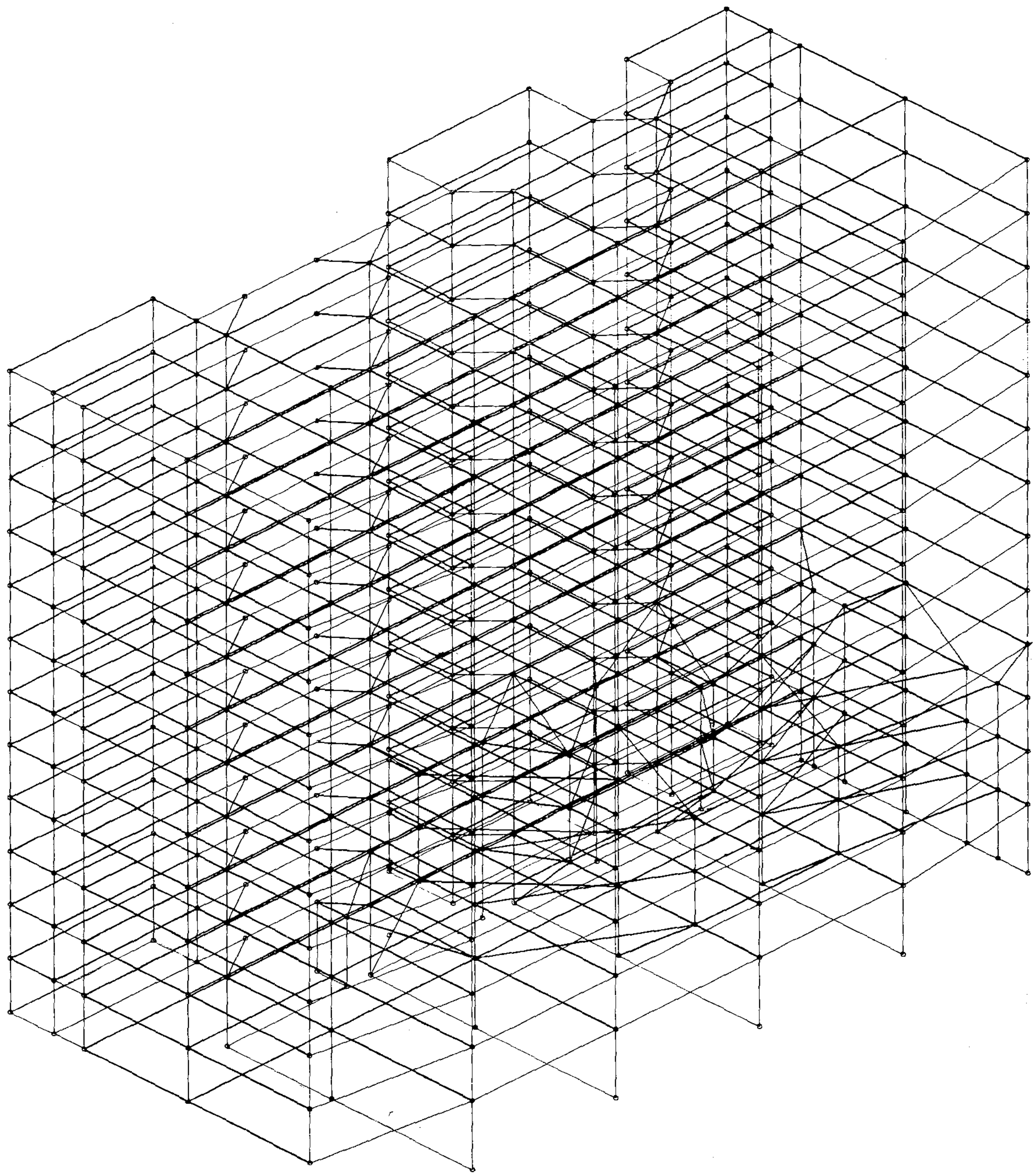
NSF/RA-770880

EARTHQUAKE RESPONSE AND DAMAGE PREDICTION OF REINFORCED CONCRETE MASONRY MULTISTORY BUILDINGS



PRISM TESTS FOR THE COMPRESSIVE STRENGTH OF CONCRETE MASONRY

University of California, San Diego

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/RA-770880	2.	3. Recipient's Accession No. PB 80 127657
4. Title and Subtitle Prism Tests for the Compressive Strength of Concrete Masonry (Earthquake Response and Damage Prediction of Reinforced Concrete Masonry Multistory Buildings)			5. Report Date November 1977
			6.
7. Author(s) G. A. Hegemier, G. Krishnamoorthy, R. O. Nunn, T. V. Moorthy			8. Performing Organization Rept. No. UCSD/AMES/TR-77/001
9. Performing Organization Name and Address University of California at San Diego Department of Applied Mechanics and Engineering Sciences La Jolla, California 92093			10. Project/Task/Work Unit No.
			11. Contract(C) or Grant(G) No. (C) (G) ENV7414818
12. Sponsoring Organization Name and Address Engineering and Applied Science (EAS) National Science Foundation 1800 G Street, N.W. Washington, D.C. 20550			13. Type of Report & Period Covered
			14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

Results of an experimental study on various aspects of concrete masonry prisms testing are reported. The project encompasses the following material: a review of correction factors for prism geometry in the Uniform Building Code; effects of capping materials, height to minimum specimen ratio, bond configuration and mortar bedding on the compressive strength of prisms; influence of ASTM and UBC recommended curing procedures on specimen strength; correlation of prism strength with full-scale wall strength; and recommended changes in prism construction/test procedures and building code modifications based on test results and literature review. In addition to investigating ungrouted concrete masonry for which current test procedures are questionable, attention was devoted to grouted masonry for which more information is required to make sound judgments. Recommendations deal with code correction factors, two-course prisms, proper prism grouting, capping test specimens, curing conditions, and bearing plate thickness. Diagrams, illustrations, graphs, and references are included.

17. Document Analysis a. Descriptors

Concrete construction	Stress
Compression tests	Masonry
Grouting	Tests
Building codes	Earthquake resistant structures

b. Identifiers/Open-Ended Terms

Building design
Prism construction

c. COSATI Field/Group

18. Availability Statement NTIS	19. Security Class (This Report)	21. No. of Pages 78
	20. Security Class (This Page)	22. Price A05-A01

(See ANSI-Z39.18) See Instructions on Reverse

OPTIONAL FORM 272 (4-77)
(Formerly NTIS-35)
Department of Commerce

Report Number
UCSD/AMES/TR-77/001

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La Jolla, California 92093

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November 1977

Report No. AMES-NSF TR-77-1

Prepared for the
NATIONAL SCIENCE FOUNDATION
Grant NSF ENV. 74-14818

PRISM TESTS FOR THE COMPRESSIVE STRENGTH OF CONCRETE MASONRY[1]

by

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ABSTRACT

Results of an experimental program on concrete masonry prisms are presented. Current masonry industry testing procedures and potential problems, and the influence of prism height, capping, bond configuration, mortar strength, mortar thickness, mortar bedding, and bearing-plate thickness are discussed. The original stress-strain curves are included. Modifications of existing codes are recommended.

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1. INTRODUCTION

1.1 The Prism Test

Present working stress design methods are based upon a knowledge of the masonry compressive strength, f'_m. In practice, f'_m is usually determined by prism tests. The importance of proper prism-test procedures and data interpretation is thus evident.

The word "prism" is synonymous with small specimens of masonry; in the case of ungrouted prisms the limiting case is the single block unit. Typical examples of single-wythe prisms are illustrated in Fig. 1. For the determination of compressive strength the prisms are capped at both bottom and top with a capping material (e.g., sulphur, gypsum plaster, mortar, fiberboard, plywood, etc). The failure load in uniaxial compression is divided by the net cross-sectional area of the block for ungrouted prisms, and the gross area for grouted prisms, to obtain the value of f'_m.

1.2 Current Practice

It is standard practice to compute f'_m on the basis of 2-course prisms laid in stack bond and capped with a high-strength sulphur fly-ash compound or a high-strength gypsum plaster ("Hydrostone" or "Hydrocal White") according to ASTM C140. Compression test procedures correspond to ASTM E447.

In the United States current masonry codes [1,2] not only allow the foregoing practice, but encourage the same by adopting universal correction factors for prism geometry (see Sec. 2404. C. 2 of the Uniform Building Code (UBC) [2]). These correction factors, Table 1 and Fig. 2, purport to enable conversion of the strength of a prism of a particular geometry to that of a standard 2-course prism, (more precisely, $h/t = 2.0$ where h, t denote prism height and least lateral dimension, respectively) the correction factor for which is unity. This, and the manner

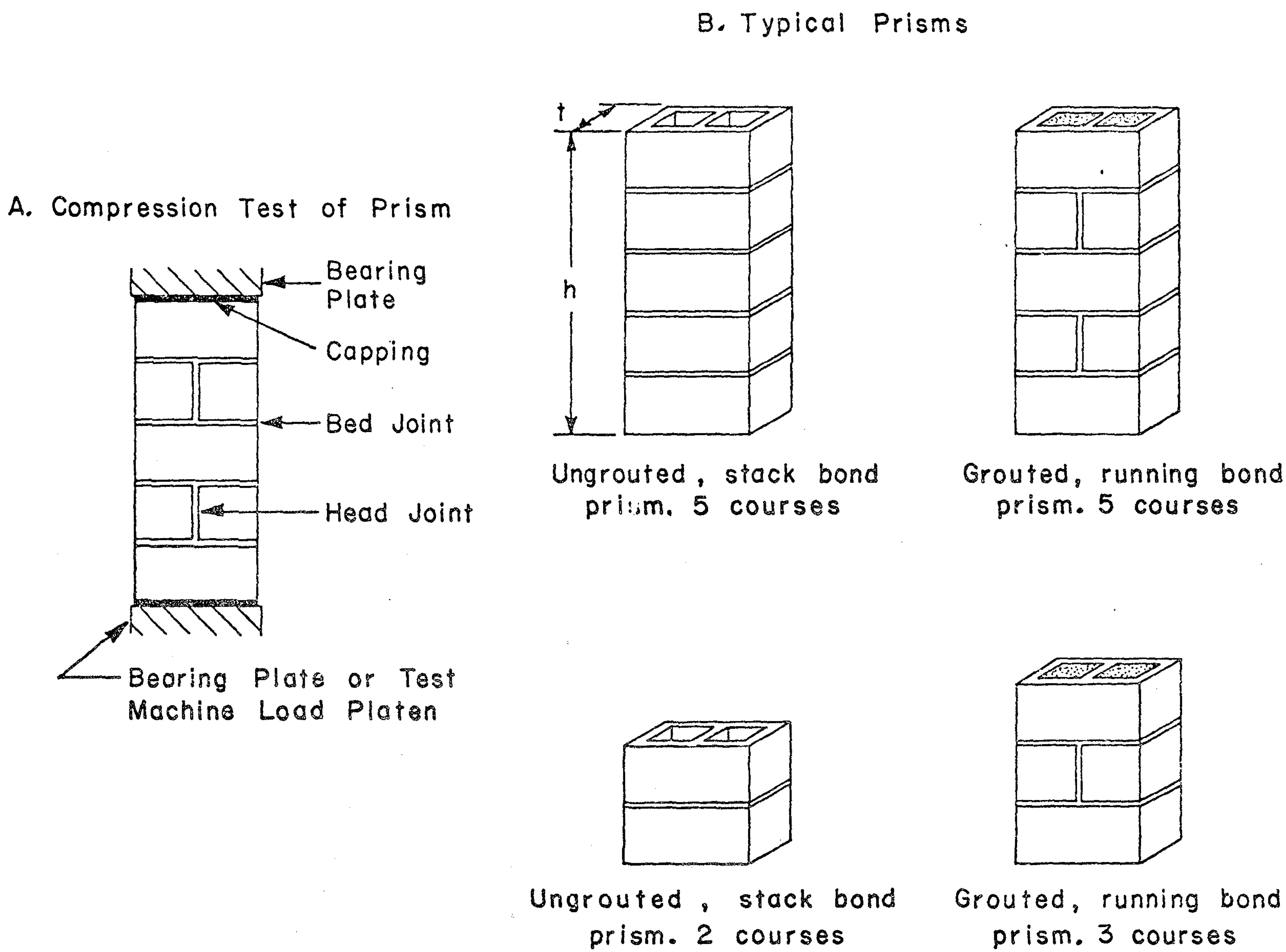


Fig. 1. Typical prism configurations for concrete masonry

Table 1. Code correction factors for prism geometry [2]

Ratio of h/t	1.5	2.0	3.0	4.0	5.0
Correction factor	0.86	1.00	1.20	1.30	1.37

where h = height of specimen

t = minimum dimension of specimen

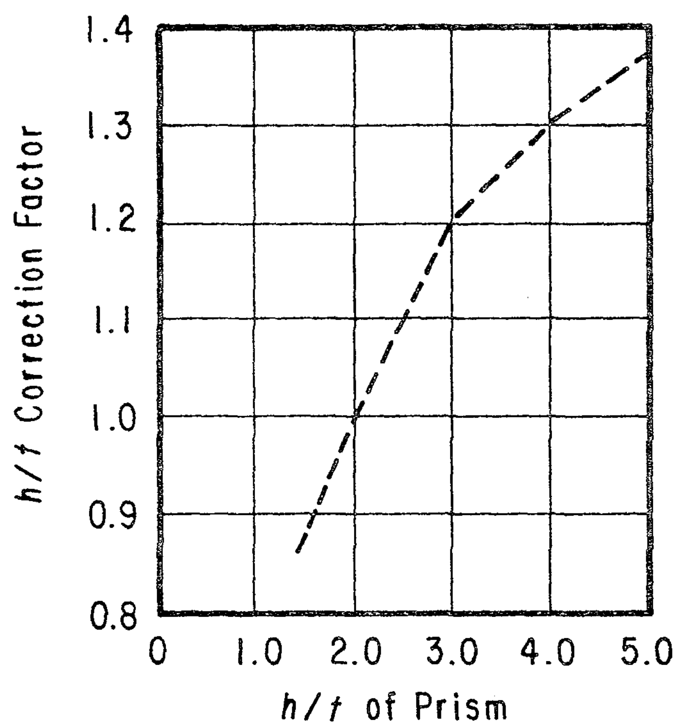


Fig. 2. Code correction factor versus h/t of prism

in which the correction factors are used (f'_m is taken as the compressive strength of the specimen multiplied by the correction factor) implies that a strong correlation exists between h/t = 2.0 and full-scale masonry. In view of the handling problems associated with larger assemblages, as well as the limited clearance in the universal testing machines, it is natural for commercial laboratories to prefer a 2-course prism and apply the correction factors recommended by the UBC.

1.3 Potential Problems and the Present Study

An extensive literature review of prism testing [3] revealed that current test procedures on prisms and the use of prism data in practice are open to serious question in the case of ungrouted concrete masonry. Items of particular concern include: 1) the code(s) correction factors for prism geometry; 2) the influence of prism construction, geometry, bond configuration, curing process, and capping procedures on strength; 3) the influence of bearing-plate thickness; and 4) correlation of prism strength with full-scale wall strength.

As noted, the foregoing literature review concerns ungrouted masonry. Sufficient information to allow judgements on grouted masonry, which is more relevant to multistory, reinforced concrete masonry construction in seismic zones, is not available in the current published literature.

Consequently, an experimental study was initiated to complement the available literature via an investigation of grouted concrete masonry within the context of the foregoing items. The results of this study together with correlation of previous works are presented herein. The significant findings are discussed and recommendations pertinent to general practice, and to building codes, are made. The original stress-strain data is included as an appendix.

2. TEST PROGRAM

2.1 Objectives

The specific objectives and scope of this test program include the following:

1) Determine the source of the correction factors for prism geometry in the Uniform Building Code.

2) Determine the validity of the correction factors.

3) Investigate the effect of capping materials on prism compressive strength.

4) Investigate the effect of the h/t ratio and the number of courses on prism compressive strength for a given capping material.

5) Investigate the influence of bond configuration (running versus stack) on prism compressive strength.

6) Investigate the influence of mortar bedding (face shell versus full) on the compressive strength of prisms.

7) Investigate the influence of ASTM and UBC recommended curing procedures on specimen strength.

8) Correlate prism strength with full-scale wall strength, where possible.

9) Recommend changes, if necessary, in prism construction/ test procedures, and building code modifications, based upon the test results and literature review/evaluation.

2.2 Materials

Prisms were fabricated using 8 x 8 x 16-inch Type N normal-weight two-cell concrete block (ASTM C90), with type S mortar 3/8" thick (ASTM C270), and grouted with a coarse 6-sack grout (ASTM C476) having an 8-10 inch slump (ASTM C143). Grout compaction was accomplished by puddling. One set of specimens was laid in stack bond with full mortar bedding; a second set was laid in stack bond with face shell mortar bedding; a third set was laid in running bond (using a combination of full and half blocks) with face shell mortar bedding. Prisms were constructed by professional masons using conventional field techniques in 2-, 3-, 4-, and 5- course sets*; each set was field cured for at least 28 but not more than 40 days prior to testing.

In addition to prisms, component samples were tested as control variables. These included 3-inch square x 5-inch high grout prisms, 2-inch dia. x 4-inch high mortar cylinders, and 4 inch x $6\frac{1}{2}$ inch high block coupons. Preparation and testing was conducted according to ASTM procedures with the exception that grout and mortar samples were field cured with the prisms. Component properties for the field cured prisms, determined as noted above, are given in Table 2.

2.3 Methods

In the main test series, precision cutting was utilized to obtain the desired h/t ratio and smooth parallel loading surfaces; cutting was conducted with a 30-inch diameter, dynamically balanced diamond-edge saw on an air-driven turbine attached to fixed rails; feed rates were

*Running bond specimens were fabricated only in 3- and 5- course sets to avoid a head joint adjacent to the load platen; the latter was thought to induce premature prism fracture.

Table 2. Component properties for full-block prism tests

	Block*	Mortar	Grout
Compressive Failure Stress (psi)	3705	1974	2828
	4000	1639	3429
	3990	1958	2039
	3148	1592	2299
		1639	
		1241	
		1868	
		2212	
		1353	
mean	3711	1720	2649
std. dev.	399	312	615

*Tests conducted on saw-cut coupons

sufficiently slow to eliminate any specimen degradation. Cutting provided the capability of having one additional bed joint for the same h/t ratio, which permitted an examination of the effect of number of bed joints on the compressive strength of the prisms.

In another test series, specimens were cut and capped with a high strength gypsum plaster (ultracal-30, $f'_c \geq 6,000$ psi) according to ASTM C140. In other test series "soft" capping materials were investigated; these included a polysulfide (PRC-380 M, produced by the Products Research Corporation) and fiberboard, each of 1/4-inch thickness.

The test set-up is shown in Fig. 3. The bearing plates in each test consisted of <u>solid</u> 8 x 8 x 16-inch precisely machined aluminum blocks. A ball and socket joint was used between the top bearing plate and the test machine load platen in order to permit rotation at the top of the prisms and thus eliminate any artificial restraint introducing moments.

Loads were applied by a 300 kip Riehle Machine and measured accurately by a 300 kip MTS load cell. All tests were conducted under displacement control at a rate of .012 in/sec.

The displacement was measured with a ± 0.50 inch LVDT (Linear Variable Differential Transformer), together with a ± .050 inch LVDT for a more accurate record of the elastic portion of the curve. The load versus relative displacement curves were recorded on seperate MFE x-y recorders. Prism failure or compressive strength was defined as the first peak in the load-displacement record.

After the prism tests were completed, it was found that displacement of the Riehle Maching had occured. The displacement

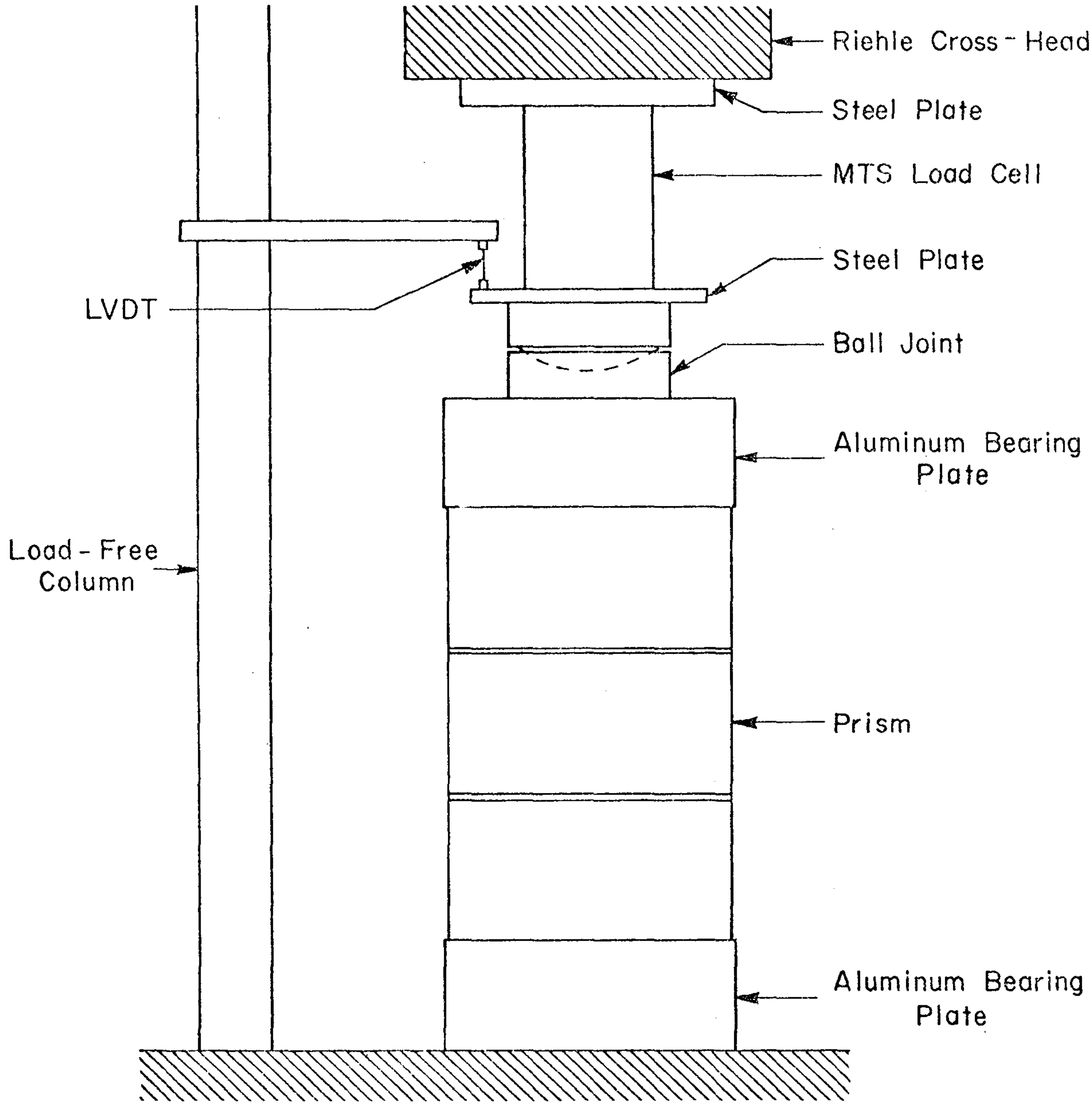


Fig. 3. Compression test set-up for concrete masonry prism

recorded was larger than the actual displacement of the prism by as much as 50 percent. To correct the recorded displacements, a new prism was loaded, with LVDTs attached to the prism itself. The difference between the original LVDT (as shown in Fig. 3) reading and the reading of the LVDT on the prism was noted for each load, and this difference was subtracted from the recorded displacements. This correction was determined for each prism height. The plots in the appendix are the corrected curves.

3. RESULTS AND DISCUSSION

3.1 Literature

A representative cross section of the available literature on prism testing, and the correlation of prism data with wall data, is provided by references [3-14].

The first reported research on representative specimens for concrete masonry wall strength was conducted on walletts, not on prisms, by Richart [12] in 1932. The prism test concept evolved from an industrial need for simpler and more economical methods for estimating the compressive strength f'_m.

Since the original work by Richart on walletts, an enormous number of compression tests on prisms have been conducted. One might suppose, therefore, that the obvious questions concerning a proper prism configuration (e. g., number of courses, stack or running bond, etc.), and a proper test procedure (e. g., capping material) for a quantitative measure of wall compressive strength have been answered with some degree of finality. Unfortunately, this is not the case.

The vast majority of prism tests have served as construction and manufacturing quality controls and the test results are not in the published literature. A substantial quantity of other prism data is evidently buried in the files of private laboratories, institutes, and associations. Consequently the published literature, in particular information pertaining to concrete masonry, is sparse and not well documented. It is sufficient, however, to reveal that considerable precautions are necessary to achieve a reliable estimate of the compressive strength of full-scale masonry.

3.2 Genesis of the Code Correction Factors

Code correction factors for prism geometry were noted previously. It is natural to question the origin of such universal factors. Foster and Bridgeman [5] addressed this question and uncovered an amazing fact: while different masonry codes may have a different "standard shape", i.e., a different value of h/t for which the correction factor is unity, the ratio of the conversion factors is constant - which suggests a common source. This source is almost certainly the preliminary and exploratory investigation by Krefeld [13] in 1936 on brick - as demonstrated by Table 3, which was reproduced from [5]. Each set of correction factors has been divided by an appropriate "code factor" to yield a common value of 0.80 for h/t = 3.0, as was obtained experimentally by Krefeld. Krefeld fully delineated the limitations of his work which involved only one brick and one mortar type; and he concluded that other factors such as brick and mortar strength, bond configuration, and prism cross-sectional dimensions also require investigation. Table 3, however, shows that his results have been accepted as being of general validity, not only for brick, but for concrete masonry as well. This, as Foster and Bridgeman have emphasized, is patently unjustified.

3.3 Platen Restraint and Geometry

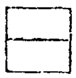
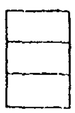
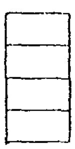
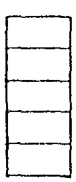
3.3.1 Specimens with "Hard" Caps

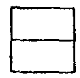
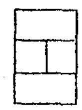
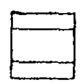
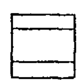
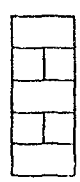
Test results on grouted prisms clearly indicate that high-strength capping materials, as used here and as specified in ASTM C140, lead to lateral restraint of the specimen at the bearing plates. Similar restraint was observed in the case of precision saw-cut specimens with no capping material. In particular, saw-cut surfaces yielded an estimate of compressive strength 10 percent greater than that for high-strength capped surfaces (see Table 4).

Table 3. Comparison between correction factors for prism shape after "Code Factor" modification [5]

Source	"Code factor"	h/t = 1.5	2.0	2.5	3.0	4.0	5.0	6.0
Krefeld	—	0.59	0.67	0.75	0.80	0.89	0.96	1.00
New Zealand Standard	1.50	0.58	0.67	0.74	0.80	0.89	0.95	1.00
Australian Standard	1.25	—	0.68	0.74	0.80	0.88	0.93	0.93
Canadian Code (concrete)	1.50	0.57	0.67	0.74	0.80	—	—	—
Canadian Code (brick)	0.93	—	0.68	0.74	0.80	0.89	0.93	—
Uniform Building Code	1.50	0.57	0.67	0.74	0.80	—	—	—
National Bureau of Standards	1.50	0.57	0.67	0.74	0.80	—	—	—
Structural Clay Prods. Inst.	0.93	—	0.68	0.74	0.80	0.89	0.93	—

Table 4. Strength comparison of grouted prisms for different h/t ratios, number of bed-joints, bond configuration, capping method, and mortar bedding

Type of Prism	h/t ratio	Strength* (psi)
 cut, full mortar bed	2	2502 2160 2562 mean 2408 std. dev. 217
 cut, full mortar bed	3	1989 1826 2405 1653 1847 1939 mean 1943 std. dev. 254
 cut, full mortar bed	4	1989 1838 1574 1496 1625 1662 1699 mean 1698 std. dev. 167
 cut, full mortar bed	5	1699 1773 1607 mean 1693 std. dev. 83

Type of Prism	h/t ratio	Strength* (psi)
 capped, full mortar bed	2	1791 2493 2271 mean 2185 std. dev. 359
 cut, face-shell mortar bed	3	1787 1939 1883 mean 1870 std. dev. 77
 cut, full mortar bed	2	1690 2170 1736 mean 1865 std. dev. 265
 cut, capped full mortar bed	2	1616 1994 1791 mean 1800 std. dev. 189
 cut, face-shell mortar bed	5	1426 1450 1371 mean 1416 std. dev. 41

*Stress based on area of 119.1 in^2

In both cases bearing plate, or "platen" restraint is due to friction at the interface between the specimen and the platen.

Platen restraint can be observed by its effect on compressive strength, by its effect on failure mode, and by strain gage data.

1) Compressive Strength. The most sensitive measure of platen restraint is compressive strength. Test data indicates that prism compressive strength is significantly influenced by platen restraint and, in the absence of a soft capping material, is a strong function of the number of courses up to 4 courses, with strength invariance between 4 and 5 courses. A typical example is illustrated in Fig. 4; the data for this case was obtained from saw-cut stack-bond grouted specimens. The curve in Fig. 4 (the data was normalized using 2295 psi) represents the means of repeated tests at integer h/t ratios with interpolation between integer h/t ratios. Similar results were observed for precision saw-cut grouted specimens with running bond, for h/t = 3 and 5. For comparison purposes, results of the running-bond tests are included in Fig. 4; the data was again normalized on the 2-course stack-bond prism strength, 2295 psi. The test data is also presented in tabular form in Table 4 for completeness.

As can be observed from Fig. 4, the 2-course estimate of f'_m in the presence of platen restraint is, based upon the 5-course prism data, approximately 36 percent high for grouted stack-bond masonry and 62 percent high for grouted running-bond masonry.

The prism test data revealed another important point. Based upon data from saw-cut specimens, prism compressive strength was observed to be primarily a function of the number of bed-joints in the

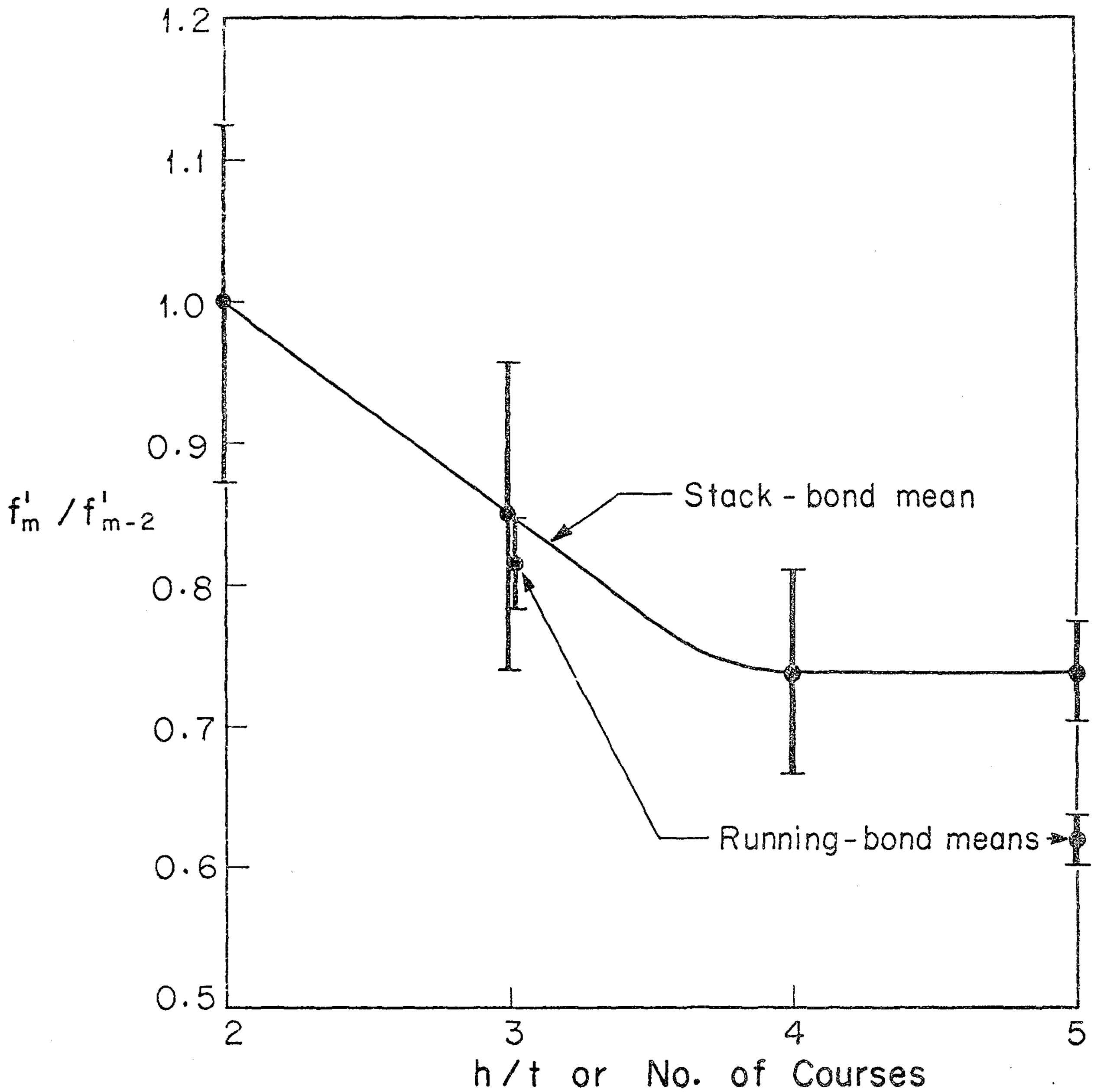


Fig. 4. Prism compressive strength versus h/t

specimen - <u>not the h/t ratio</u>. For example, prisms with 2 bed-joints saw-cut to h/t = 2.0 exhibited strengths similar to specimens with 2 bed-joints and h/t = 3.0 (see Table 4). Thus, <u>interpolation for h/t between integer number of courses (uncut) or bed-joints is not a valid operation</u>.

It must be emphasized at this point that the foregoing trends apply only to the material combination tested. In particular, one should <u>not</u> attempt to construct correction factors based upon the data reported herein. The point, in fact, is just the opposite: since correction factors can be expected to be highly material dependent, they cannot be relied upon to furnish an adequate estimate of f'_m.

2) <u>Failure Modes</u>. Differentiation of failure modes in the case of full-block grouted prisms is difficult; thus, the failure mode(s) is not a good measure of platen restraint. This situation is quite different, however, for half-block prisms, and the latter is worth noting.

In the case of half-block grouted prisms (the component properties for which are given in Table 5), platen restrain in 2-course prisms generally produced shear-type failures, whereas the observed failure mode in walls is vertical tensile splitting; a typical shear failure mode is shown in Fig. 5. In prisms of 3 courses, the failure mode approaches the proper tensile splitting in the central unit; this is illustrated in Fig. 6 a, b. In 4-course and 5-course prisms, the failure mode more closely resembles a wall compression failure.

In the case of 2-course full-block grouted prisms, shear failures (Fig. 7) were usually observed. For prisms of more than 2 courses, the failure mode could frequently be characterized as tensile-splitting of the end face shells, and tensile splitting of the grout cores. A typical failure is shown if Figs. 8 a-c. The phenomenon of face-shell spallation

Table 5. Component properties for half-block prism tests

	Block*	Mortar	Grout
	2080	3780	5380
	2320	4580	5780
	3260	3780	5770
	2570	4260	
Compressive Failure Stress (psi)	3320		
	2450		
	3210		
	3210		
	2680		
	2400		
mean	2750	4100	5640
std. dev.	460	390	230

*Net area strength

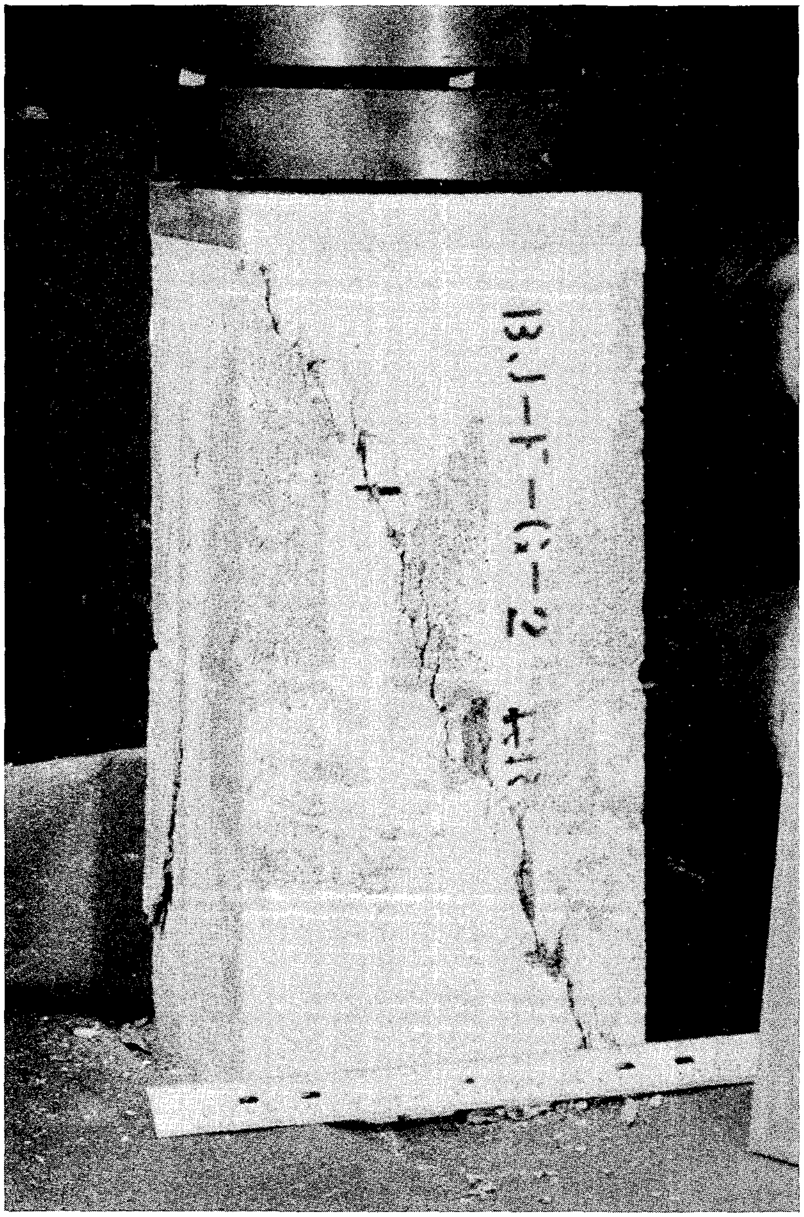


Fig. 5. Typical shear-mode failure in 2-course half-block prism with "hard" cap

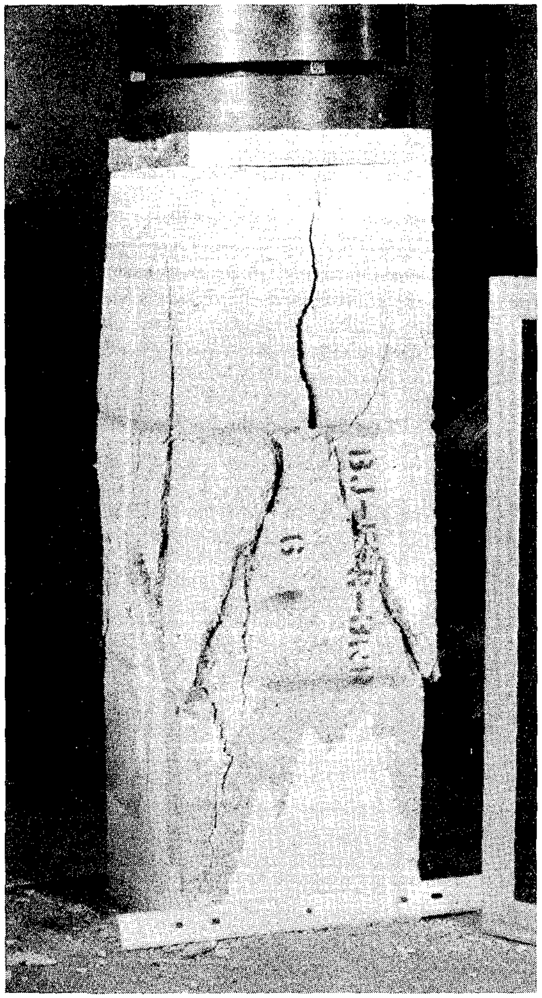


Fig. 6a. Typical face-shell splitting in 3-course half-block prism with "hard" cap

Fig. 6b. Grout core splitting of Fig. 6a

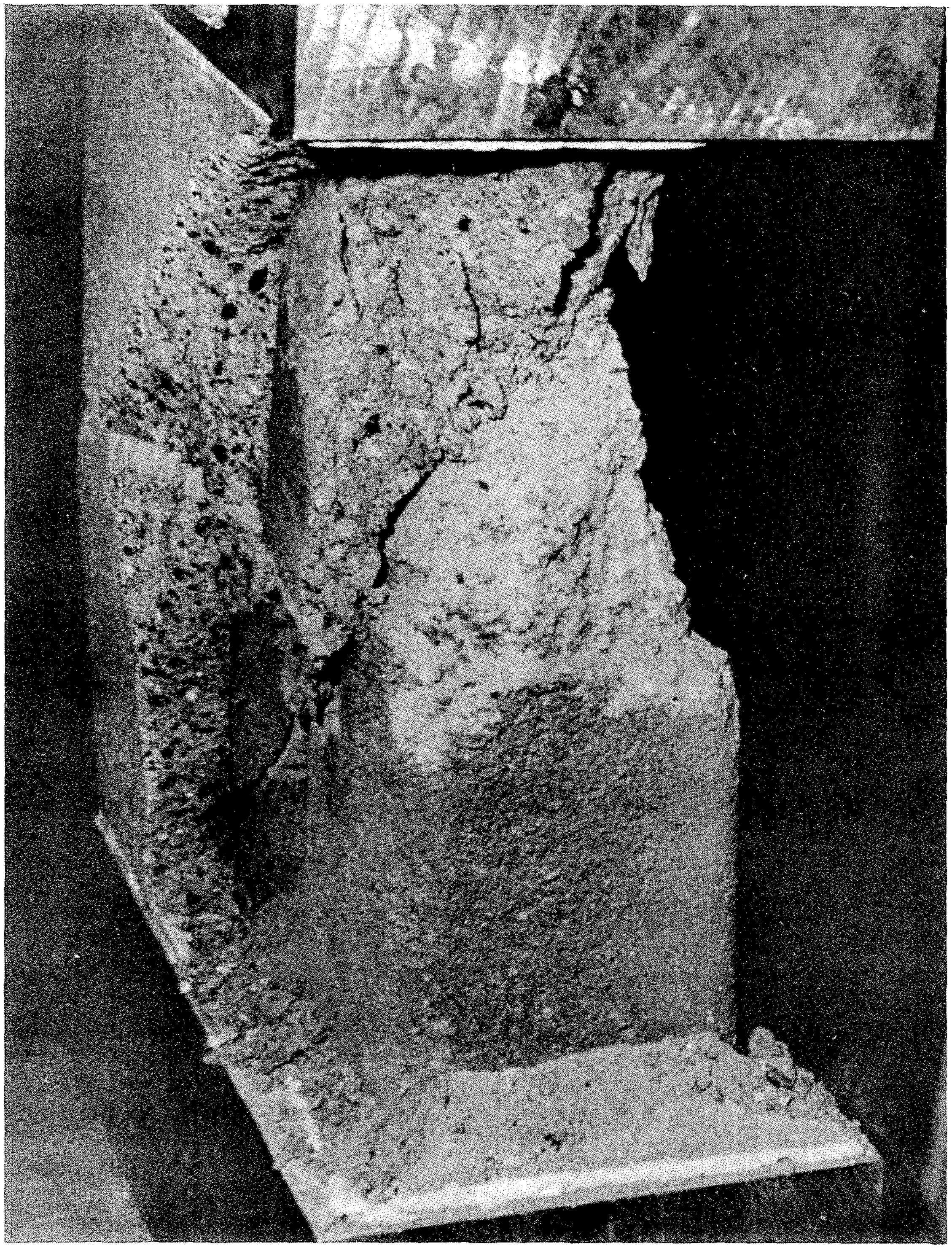


Fig. 7. Typical shear-mode failure in 2-course full-block prism with "hard" cap

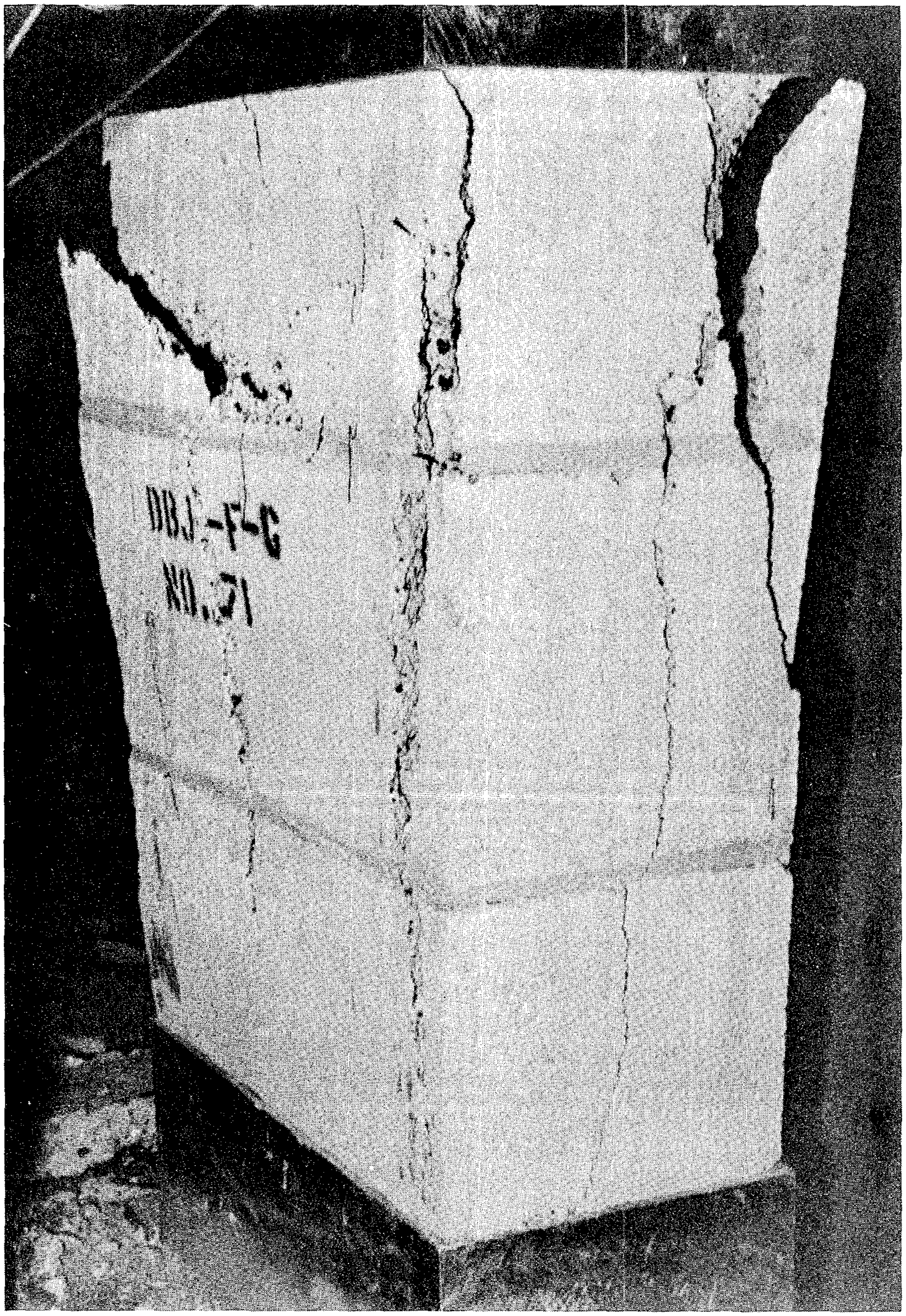


Fig. 8a. Typical face-shell splitting and spallation in 3-course full-block prism with "hard" cap

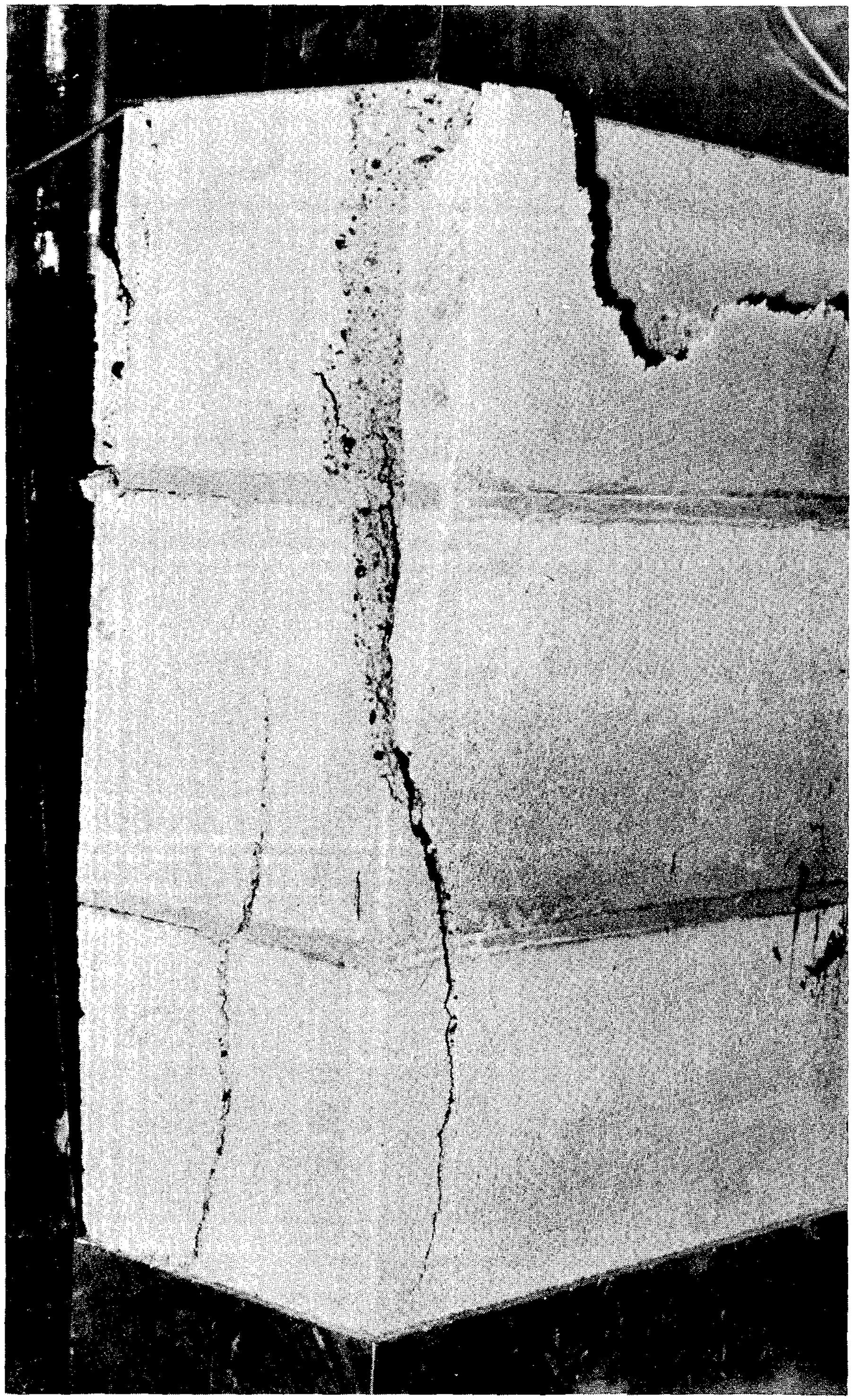


Fig. 8b. Face-shell splitting and spallation of back side of specimen of Fig. 8a

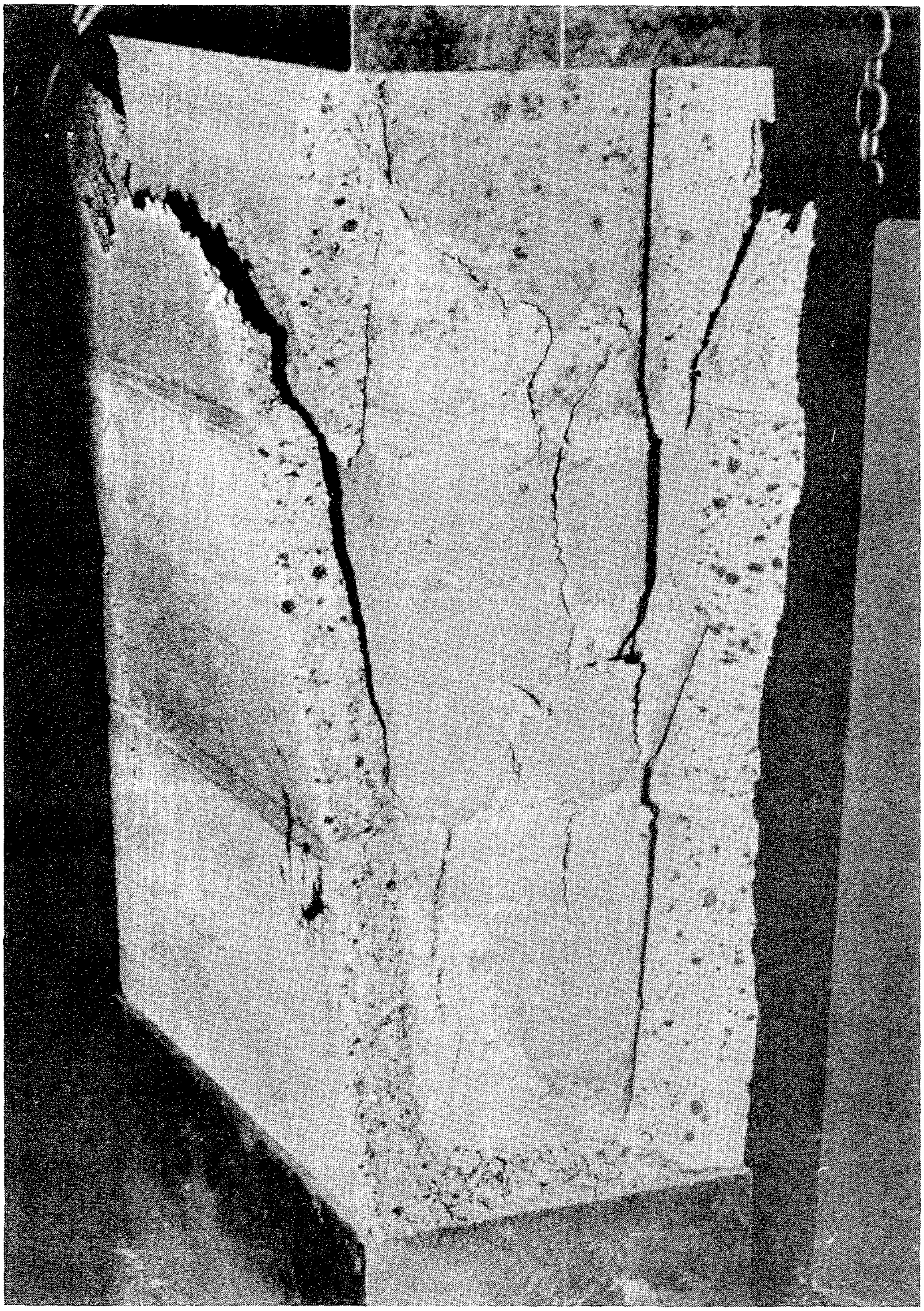


Fig. 8c. Grout core tensile-splitting of specimen of Fig. 8a

away from the grout cores was observed frequently; the block and grout are clearly not functioning as an integral unit in these tests.

3) Strain-Gage Data. The influence of platen restraint can be clearly observed via strain gage measurements. Results of a test on a grouted 3-course prism are shown in Fig. 9.

3.3.2 Specimens with "Soft" Caps

Use of the polysulfide as a capping material yielded proper tensile splitting in 2-course prisms, Fig. 10, and strength invariance between 2 and 5 courses. This is the result of the polysulfide's low shear modulus (150 psi) which lubricates the interface between the specimen and the bearing block and essentially eliminates the platen restraint. Unfortunately, this material (and similar materials) is expensive and difficult to handle; improper use can lead to premature failure. Consequently, the polysulfide capping is judged to be impractical for conventional laboratory or field testing.

In constrast to tests on ungrouted prisms [7], fiberboard capping was observed to produce large data scatter and did not sufficiently relieve load platen restraint in grouted prisms. Further, fiberboard types and grades apparently differ considerably from region to region. Consequently, fiberboard is not regarded as a suitable "standard" capping material for prisms.

3.3.3 Correlation with Available Data

The influence of platen restraint on compressive strength of ungrouted prism specimens is reported in the literature and is worth noting at this point.

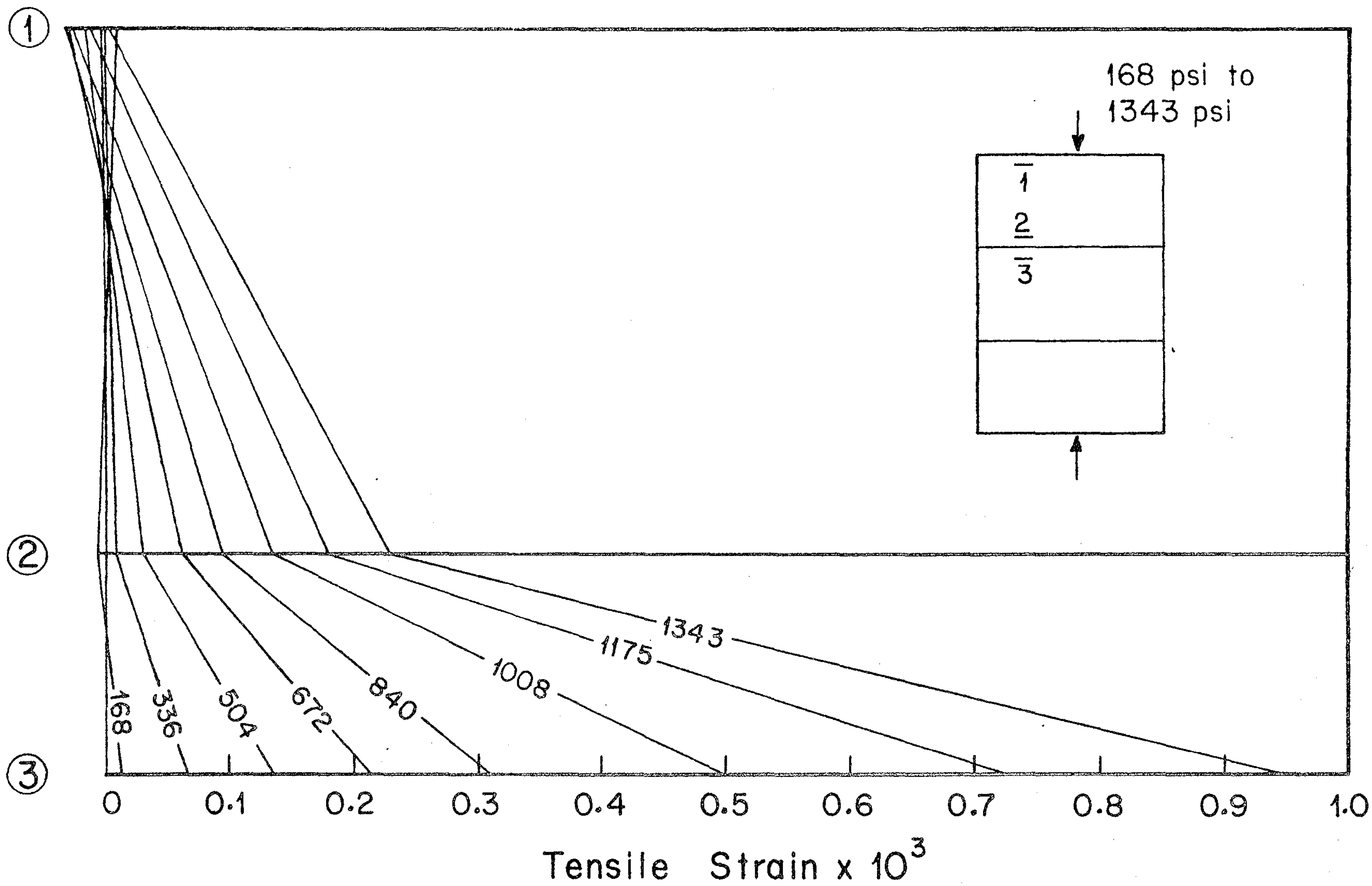


Fig. 9. Variation of lateral strain with height for 3-course prism

Fig. 10. Typical vertical tensile splitting of 2-course half-block prism with soft polymer cap

The decrease in compressive strength with increasing number of courses, associated with the use of high strength capping materials, can be observed in the data of Foster and Bridgeman [5] on 4 × 8 x 16-inch hollow concrete block prisms. The latter is reproduced as Fig. 11. The data shows a decrease in strength at least up to 4 courses and h/t = 8.7.

It is clear that the undesirable effects of platen restraint are alleviated by increasing the number of prism courses. This can be observed via the strain gage data of Self [6] on 2 and 3-course ungrouted prisms, Figs. 12 a-c.

Finally, data on ungrouted prisms supports the premise that reduced platen restraint, and a corresponding decrease in compressive strength, is achieved with soft capping materials. Yokel, Mathey and Dikkers [7], for example, report that the compressive strength of 3-course hollow 8-inch block prisms with high-bond mortar capped with fiberboard was 44 percent less than the same prisms capped with high-strength plaster.

3.3.4 Calculation of f'_m

As noted in Section 1.2, f'_m is taken as the strength of a prism multiplied by the h/t correction factor of Table 1. This procedure seems to imply the true strength of concrete masonry is that of a 2-course prism, while prisms of more than 2 courses are somehow weakened. But in fact 2-course prisms with hard caps are seen to be artificially strengthened, whereas prisms of 4 and 5 courses approach the true strength. While this artificial strengthening may now be compensated for in the safety factor of the allowable working stress, (equal to .2 f'_m for walls) f'_m should be taken to be the strength of a 4 or 5 course prism, in which case the actual safety factor will be clearly evident.

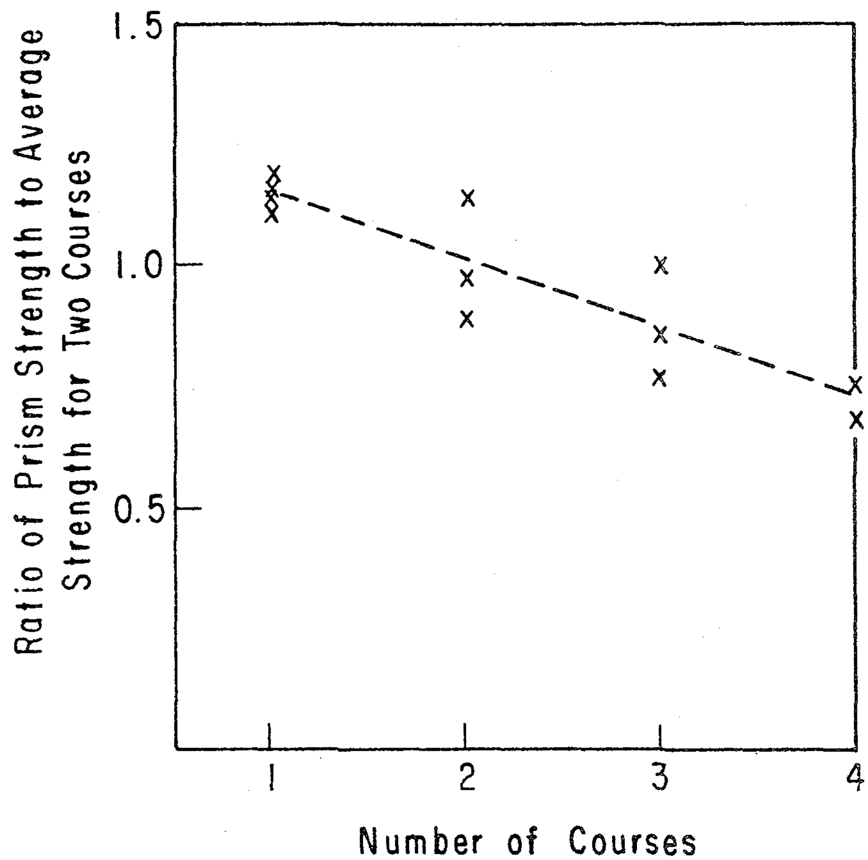


Fig. 11. Correlation between ungrouted concrete block prism strength and number of courses [5]

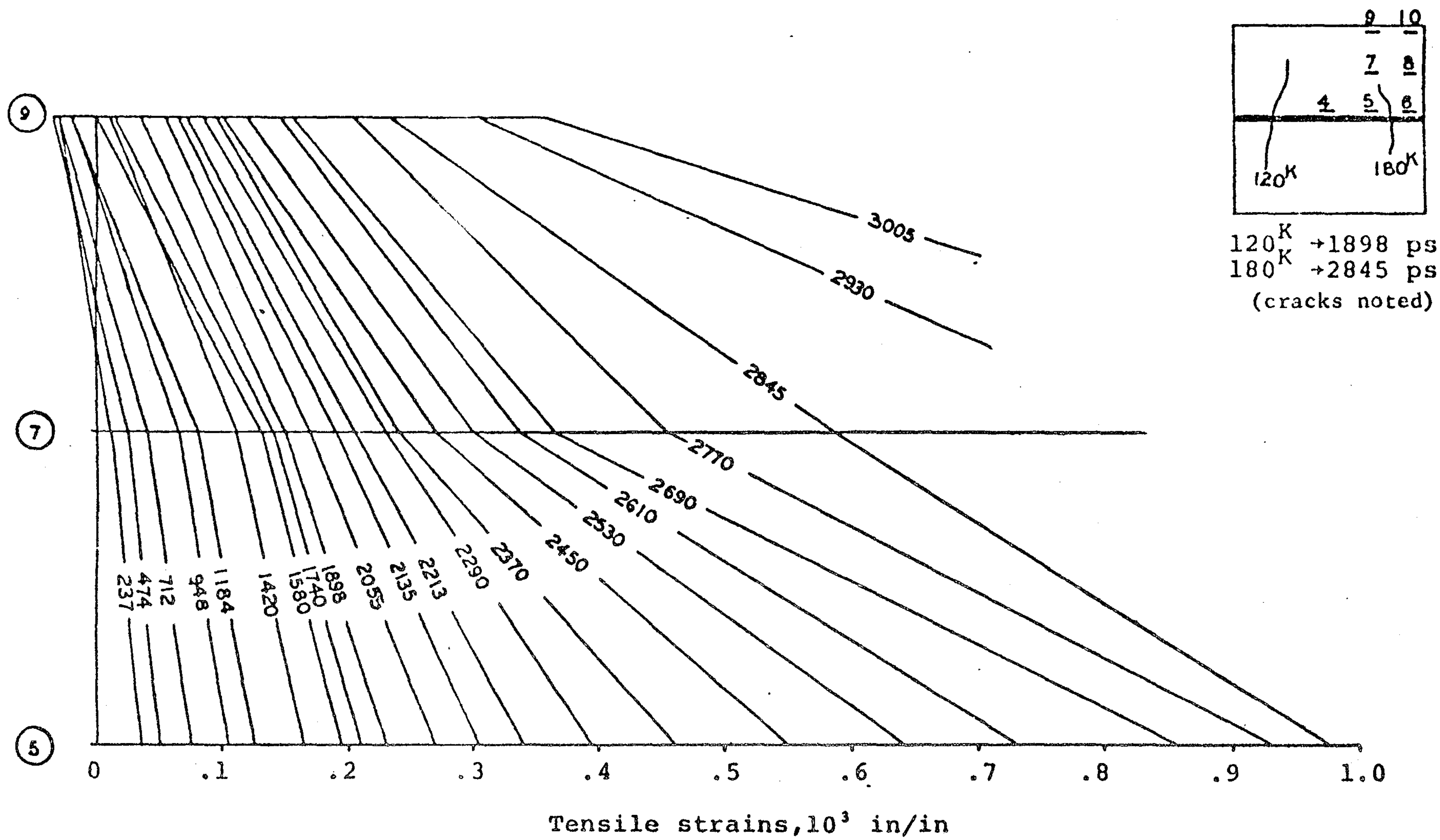


Fig. 12a. Distribution of lateral strains over block height of 2-block prism [6]

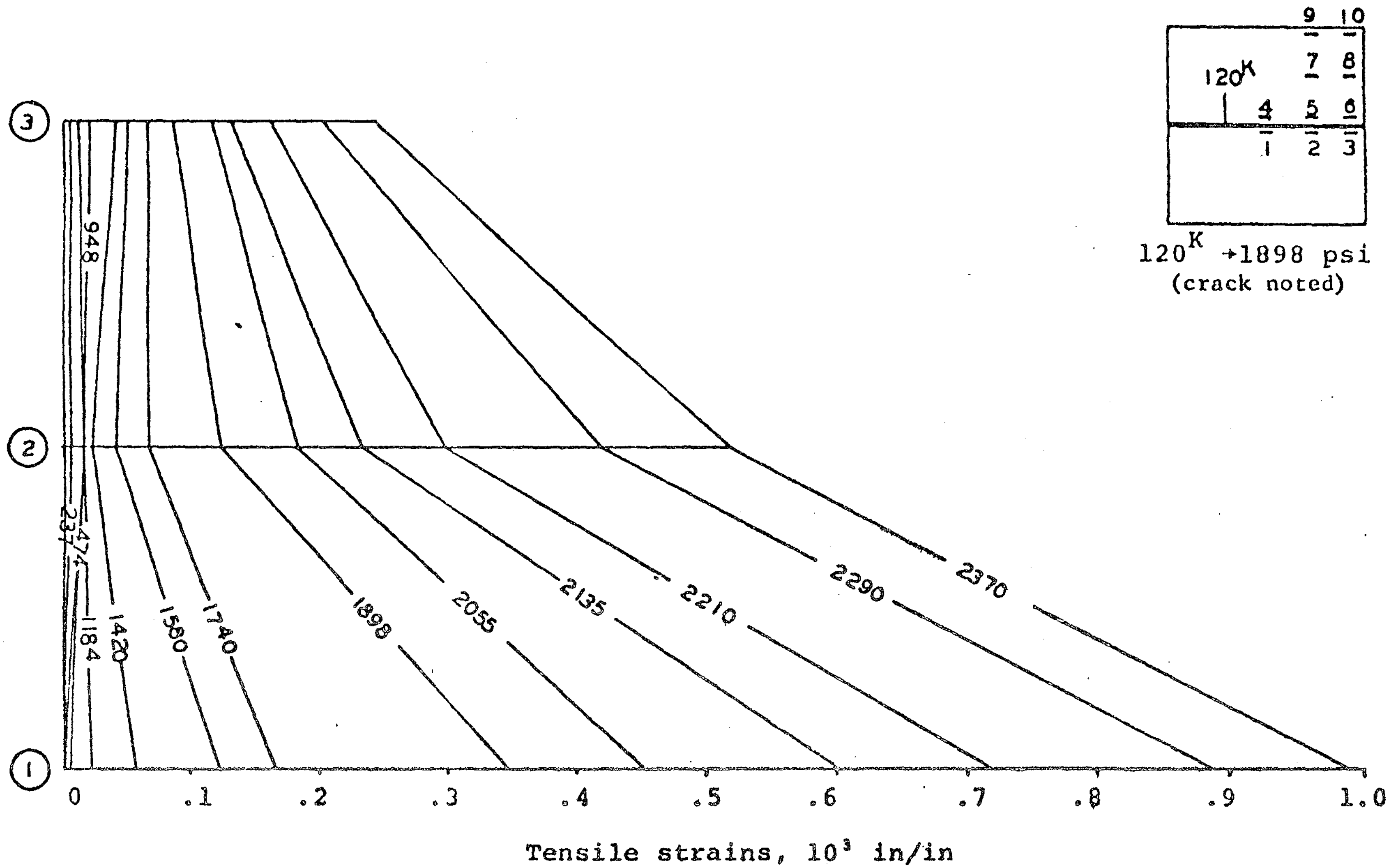


Fig. 12b. Distribution of lateral strains along mortar line of 2-block prism [6]

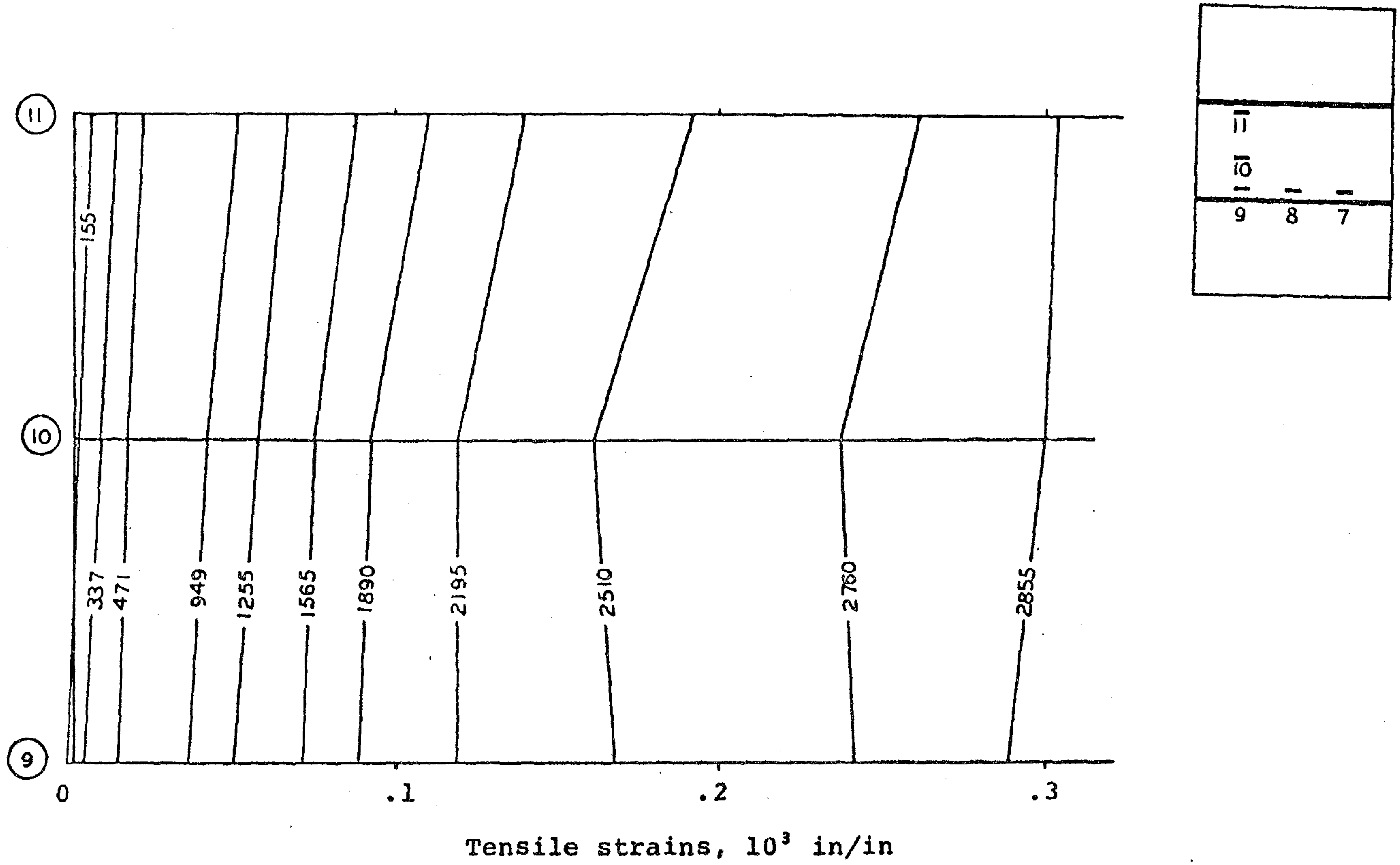


Fig. 12c. Distribution of lateral strains over block height of 3-block prism [6]

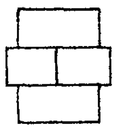
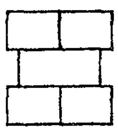
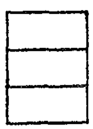
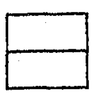
3.4 Mortar Joint Geometry versus Strength

3.4.1 Running Bond versus Stack Bond

Foster and Bridgeman [5] have suggested that prism geometry, in particular mortar joint geometry, may influence prism strength; they in turn have concluded that bond configuration in the prism should simulate the bond configuration in the masonry structure as closely as possible. The experiments by Self [6] on bond pattern (stack or running) in ungrouted prisms appear to support their premise. Table 6, which was reproduced from [6], exhibits considerable differences in compressive strength between stack-bond and running-bond ungrouted prisms. In ungrouted masonry this difference may be attributed to the following: although concrete masonry walls are usually constructed with the block in running bond, test prisms are fabricated in stack bond. The significance is that in running bond the cross webs are not in vertical alignment and even if mortared may not effectively transmit compression through the joint. This is particularly true when stretcher blocks are used. Masons often prefer stretchers because they provide a better hand-hold at the end web. This point has also been noted by Reed and Clements [8]. Self [6] concluded that, consequently, only the face shells (as contrasted to the net cross-sectional area) should be considered as effective bearing area in ungrouted running-bond masonry.

The current test program on grouted prisms has revealed a similar phenomenon: the compressive strength of prisms laid in running bond is significantly less than the compressive strength of prisms laid in stack bond. Table 4 shows typical results for 3- and 5-course prisms, the component properties of which are provided in Table 2. The specimens in this series were, again, precision saw-cut to the desired h/t ratio. In addition to the influence of bond type, Table 4 clearly reveals a decrease of prism compressive strength with increased number of courses for both stack-bond and running-bond masonry.

Table 6. Strength comparison between stack-bond and running-bond ungrouted prisms [6]

Type of Prism	Block Strength psi	Prism Strength psi	Efficiency Percent		Block Strength psi	Prism Strength psi	Efficiency Percent	
	5100	2087	41	41	7400	2796	38	39
		1838	36			3246	44	
		1845	36			2330	32	
		2589	51			2934	40	
	5100	2264	44	46	7400	3246	44	41
		2224	44			2883	39	
		2380	47			3050	41	
		2536	50			3262	44	
	5100	4035	79	69	7400	4420	60	63
		3027	59			4811	65	
		3136	61			4828	65	
		3834	75			4485	61	
	5100	3632	71	73	7400	4680	63	62
		3828	75			4542	61	

Type S mortar with initial flow of 90%, 3300 psi at 28 days.

Capping material: fiberboard.

3.4.2 Influence of Mortar Bedding

Mortar bedding (face-shell versus full) exhibited little influence on the compressive strength of grouted prisms laid in stack bond.

Tests to determine the influence of mortar joint thickness were not conducted. However, information on this item is available in the literature, and is worth noting for completeness. The influence of mortar joint thickness on the prism strength is a function of the ratio of masonry unit height to joint thickness. Because this ratio is high for concrete block, typical variations of joint thickness in commercial construction is expected to produce negligible change in compressive strength. This fact can be inferred from the NCMA data shown in Table 7.

3.4.3 Influence of Mortar Strength

Tests to determine the influence of mortar compressive strength on prism compressive strength were not conducted in the present test series. However, available data on ungrouted prisms reveals little influence. A typical example is provided in Fig. 13; the data on ungrouted prisms laid in stack bond was extracted from [6]. Based upon such tests, it appears safe to conjecture that mortar strength has little influence on the compressive strength of grouted prisms.

3.5 Influence of Bearing Plate Thickness

The bearing plates in the present tests were selected as solid 8 x 8 x 16 aluminum members, as previously noted. The reason for this selection is worth mentioning at this point.

ASTM C140 requires that steel bearing plates employed between the spherically seated head block and the test specimen shall have a thickness equal to at least one third the distance from the edge of the

Table 7. NCMA prism strength research-influence of mortar joint thickness

Joint thickness	Total load lbs. (1)		Compression psi (2)		Percent of unit strength	
	7 day	28 day	7 day	28 day	7 day	28 day
1/4 in.		106,018		2524		95
3/8 in.	99,310	108,918	2365	2593	89	98
1/2 in.		98,712		2350		89
5/8 in.		105,660		2516		95
3/4 in.	81,232		1934		73	

(1) Average of five tests
(2) Based on bedded area

Material properties

Block: 8 x 8 x 16 Two core standard
Unit weight 105 pcf
Net area 65.0 in.^{2}, 54.3 percent
Net area strength 2650 psi
Face-shell bedding area 42.0 in.^{2}

Mortar: Type S, masonry cement
28-day cube strength 1690 psi

Prisms: Three-block, h/t = 3
Face-shell bedding, flush joints

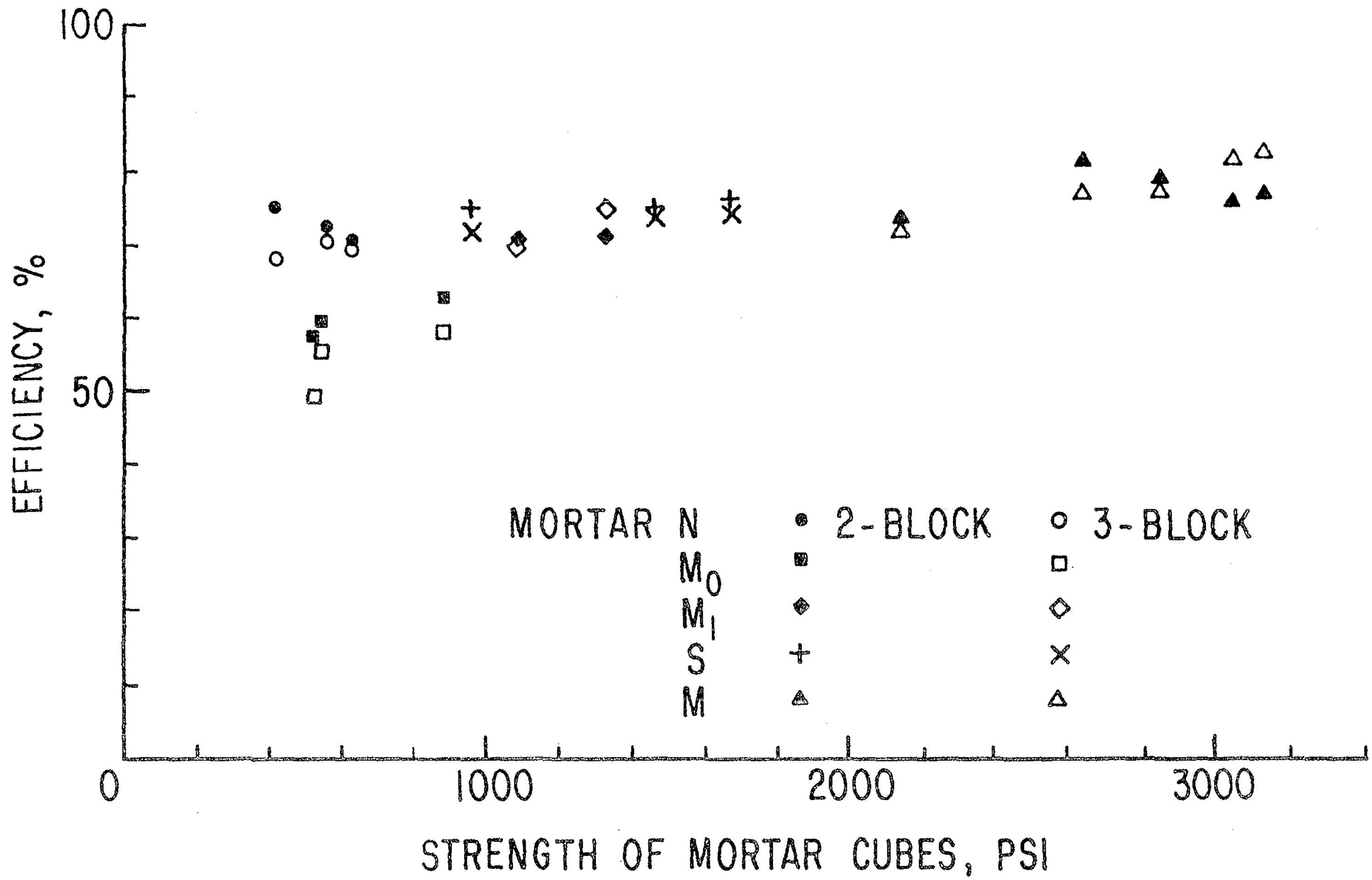


Fig. 13. Prism efficiencies vs. strength of mortar cubes [6]

head block to the most distant corner of the specimen. For a typical 8-inch diameter round head block, and an 8 x 8 x 16-inch concrete block specimen, the required thickness of the bearing plate would be 1-1/2 inches.

Tests conducted by Self [6] on ungrouted prisms and Langpap [14] on grouted prisms reveal that a 1-1/2 inch bearing plate undergoes considerable bending and induces non-uniform strain distributions in blocks and/or prisms. Typical strain variations in single 8 x 8 x 16-inch two-cell hollow blocks versus plate thickness are shown in Fig. 14; this data was excerpted from [6]. Figure 15 on the other hand, shows typical stress (based upon a modulus of 3×10^6 psi) variations in a 2-course grouted prism of 8 x 8 x 16-inch concrete blocks utilizing a 1-1/2-inch bearing plate (with some added ribs); the strain gage layout for the latter test data, which was excerpted from [14] is shown in Fig. 16.

The foregoing tests clearly indicate that ASTM C140 is inadequate and should be modified with respect to bearing plate thickness.

With respect to the present tests, aluminum was judged to be more acceptable than steel due to its low weight and cost, and an 8-inch thickness was, based upon independent calculations, considered a minimum thickness able to provide a resonably uniform strain field.

3.6 Correlation with Wall Data

It was previously emphasized that 2-course prisms (couplets) laid in stack bond and capped according to ASTM C140 can lead to an over-estimate of f'_m for full scale running-bond masonry. The magnitude of the error encountered in some cases can be observed in the data of Read and Clements [8]. The walls tested in uniaxial compression were

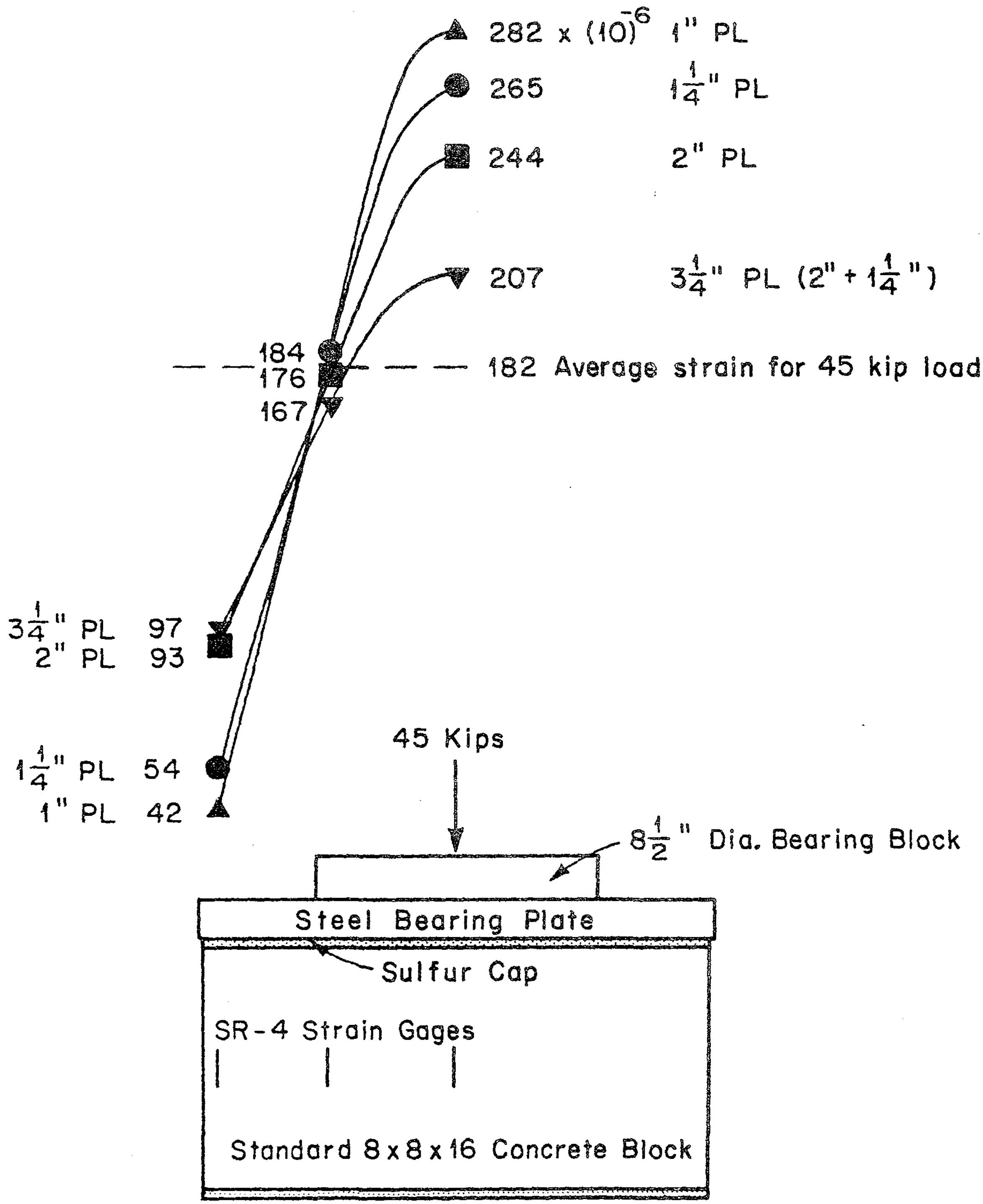


Fig. 14. Influence of bearing-plate thickness upon vertical strain in block face-shell [6]

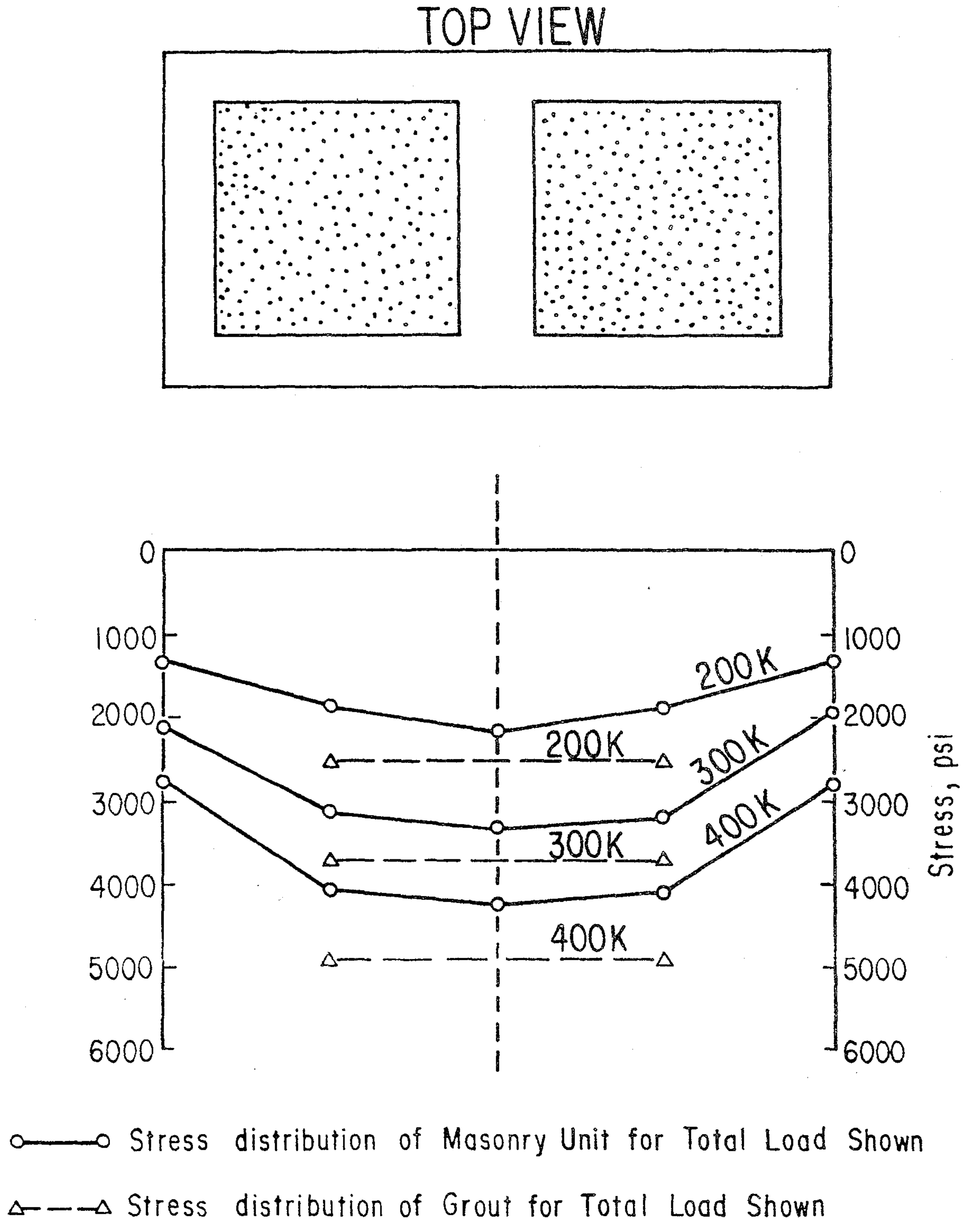


Fig. 15. Stress distribution for various total loads [14].

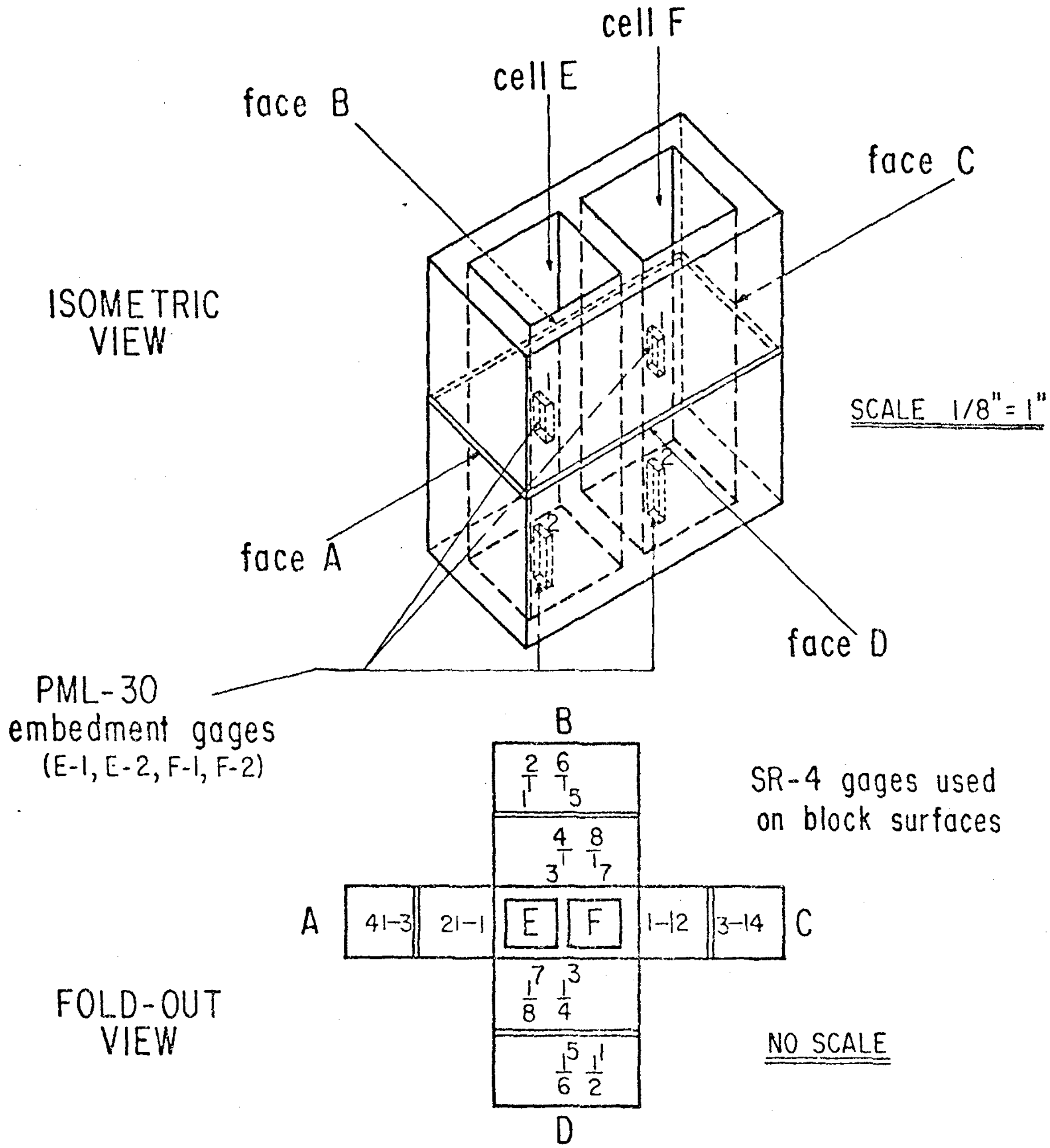


Fig. 16. Masonry prism test specimen, configuration, and gage location plan [14]

2.6m high and 1.8m wide. Correlation between prism and wall data is illustrated in Fig. 17 for ungrouted walls fabricated from the units shown in Fig. 18. A running bond using a 1:4:3 mortar mix was employed.

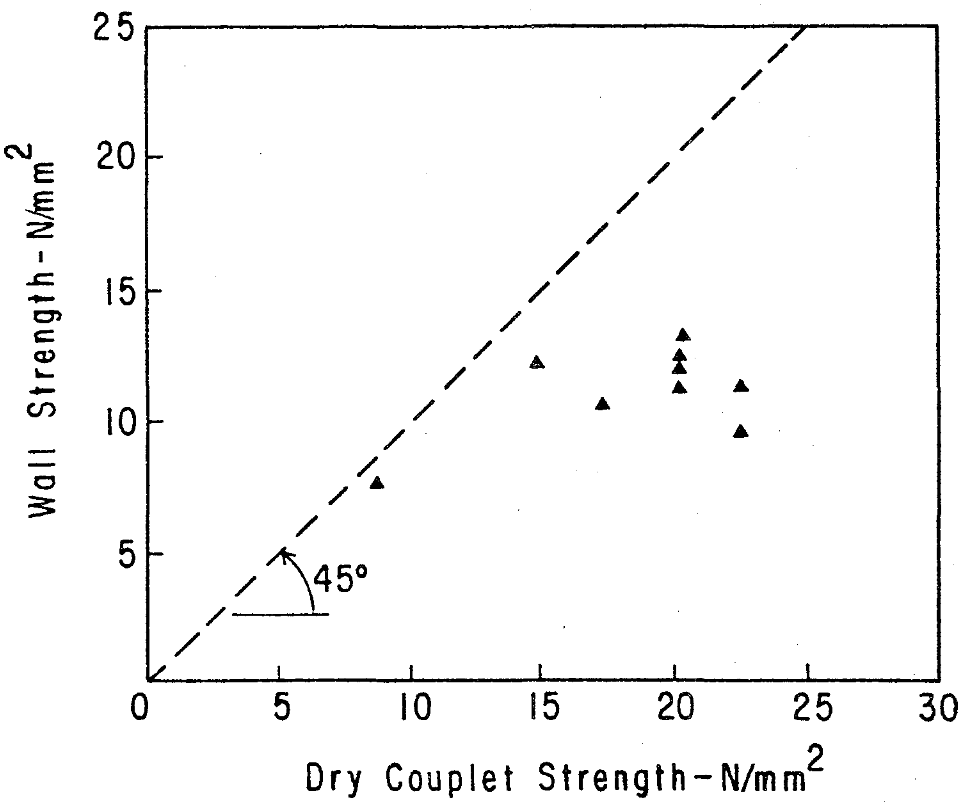


Fig. 17. Comparison of wall and couplet strengths [8] (Units Newtons per square millimeter)

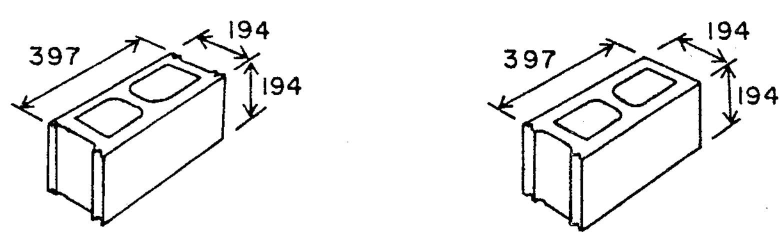


Fig. 18. Types of block used in the investigation [8] (All dimensions in mm.)

4. SUMMARY OF RESULTS

The following information was obtained from the prism test program and/or the associated literature survey.

1) Virtually all masonry code correction factors for prism geometry are based upon a common source: the preliminary and exploratory investigation by Krefeld in 1938 - on brick! The universal use of such data is clearly unjustified.

2) The present widespread practice of computing f'_m from 2-course prisms laid in stack bond and capped according to ASTM C140 is nonconservative. Over-estimates of 62 percent have been observed for grouted, running-bond concrete masonry.

3) High strength capping materials, as specified in ASTM C140, lead to lateral restraint of the specimens at the bearing plates (platens). Platen restraint in 2-course prisms produces shear mode failures (see Fig. 5) whereas the observed failure mode for walls is vertical tensile splitting.

4) In 3-course prisms, the failure mode approaches the proper tensile splitting in the central unit (see Fig. 6). In 4- and 5- course prisms tensile splitting occurs in all units except possibly those adjacent to the platens. For prisms of more than 2 courses, face-shell spallation away from the grout cores was frequently observed. The block and grout appear not to be functioning as an integral unit.

5) Compressive strength of prisms is significantly influenced by load-platen restraint and, in the absence of a soft capping material, is a strong function of the number of courses, up to 4 courses. Typical variations are shown in Fig. 4. Based upon 5-course data, the 2-course results yield in estimate of f'_m which is about 35 percent high for grouted stack-bond masonry and 62 percent high for grouted running-bond masonry.

6) Compressive strength of prisms is primarily a function of the number of bed joints-not the h/t ratio. For example, grouted stack-bond prisms with 2 bed-joints saw-cut to h/t = 2.0 exhibited strengths similar to specimens with 2 bed-joints and h/t = 3.0 (i. e., 3-courses); see Table 4.

7) Bond pattern has a significant effect on prism compressive strength. For example, 5-course grouted prisms laid in running bond with face shell mortar bedding exhibited a strength 16 percent lower than 5-course grouted prisms laid in stack bond with full mortar bedding.

8) Mortar bedding (face shell versus full) showed little influence on the compressive strength of grouted prisms laid in stack bond (see Table 4).

9) Mortar joint thickness variation, within normal commercial construction limits, shows little influence on prism compressive strength.

10) Mortar strength does not appear to significantly influence prism compressive strength (see Fig. 13).

11) Available test data on both ungrouted and grouted prisms indicates that ASTM C140 minimum bearing plate thickness is not sufficient to provide a uniform vertical strain distribution.

12) Platen restraint can be eliminated with use of a capping material having a sufficiently low shear modulus. One such material tested, a polysulfide, provided proper vertical tensile splitting in 2-course prisms. Unfortunately, the polysulfide is expensive, difficult to properly apply, and would be difficult to standardize. Such materials are therefore judged not to be feasible for commercial applications.

13) Limited testing of fiber board as a soft capping material yielded negative results in the sense that: (i) the material did not sufficiently alleviate platen restraint and (ii) the material varies in type from region to region and would appear to constitute a problem from a test standardization viewpoint.

14) A number of lubricants, including oil, were applied to the surfaces of precision-cut specimens in an effort to minimize load-platen restraint. All such tests were negative. Similar tests were conducted on capped surfaces; again the results were negative.

5. RECOMMENDATIONS

5.1 Code Correction Factors

Section 2404. C. 2. b of the 1976 Uniform Building Code, which concerns correction factors for prism geometry, should be deleted. A similar statement applies to all masonry codes where such factors are published.

5.2 Two-Course Prisms

The current wide-spread practice of evaluating f'_m from two-course prisms laid in stack bond should be terminated.

5.3 Proper Prism Geometry

The compressive strength, f'_m, of concrete masonry should be evaluated using prisms with not less than three nor more than four mortar bed-joints. This may be accomplished with prisms of not less than four nor more than five courses. In the case of grouted prisms, the prisms may be precision saw-cut to a lower h/t ratio, commensurate with the above number of bed-joints, in order to alleviate laboratory space problems. Ungrouted prisms, however, should not be cut.

The mortar bond configuration (stack, running, etc.), the mortar bedding (face-shell or full), and the grouting should, in so far as possible, be the same as is used in the structure.

The Uniform Building Code, the ASTM Standrads (ASTM E447-74), and other masonry codes should be rewritten such that f'_m is computed according to, and only according to, the above geometry.

5.4 Capping Test Specimens

The ends of the prisms should be capped as set forth in ASTM C140 with the following exception: grouted prisms may be precision saw-cut, in lieu of capping, to provide smooth parallel surfaces.

At this time it is recommended that estimates of f'_m based upon use of "soft" capping materials, such as fiberboard, not be accepted as valid.

5.5 Curing Conditions

Prisms should be constructed at the job site in a place where they will not be disturbed, and should be subjected to atmospheric conditions at the job site (i.e., cured at the job site) for the entire 28 days prior to transport to a laboratory and subsequent testing. Compressive strength based upon moist-room cured specimens should not be accepted as a measure of f'_m. Revision of ASTM E447-74, the Uniform Building Code, and other masonry codes is recommended to reflect the above.

5.6 Bearing Plate Thickness

Specifications for minimum bearing plate thickness in ASTM C140-75 should be revised to conform to ASTM E447-74.

ACKNOWLEDGEMENT

Publication of this report was facilitated by a contribution from the Unit Masonry Association of San Diego, Inc.

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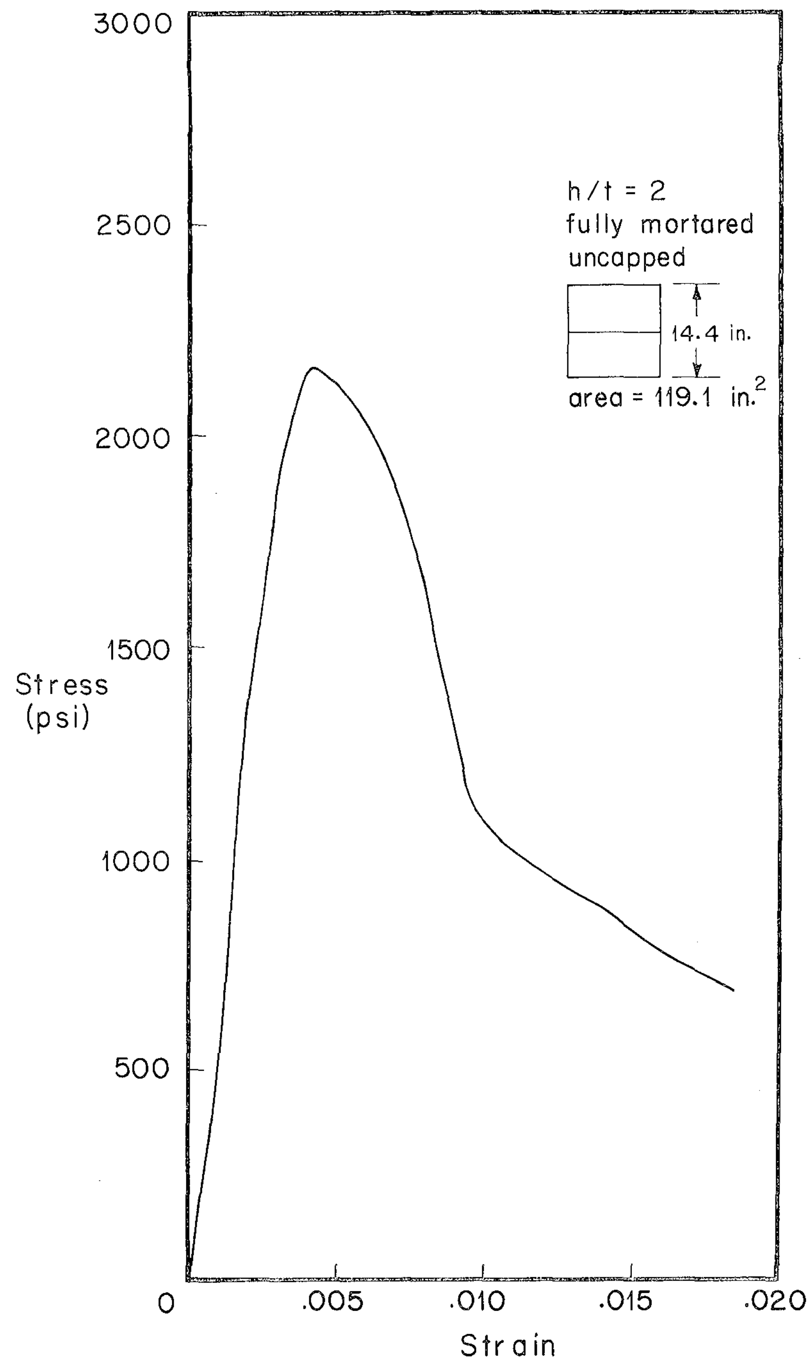
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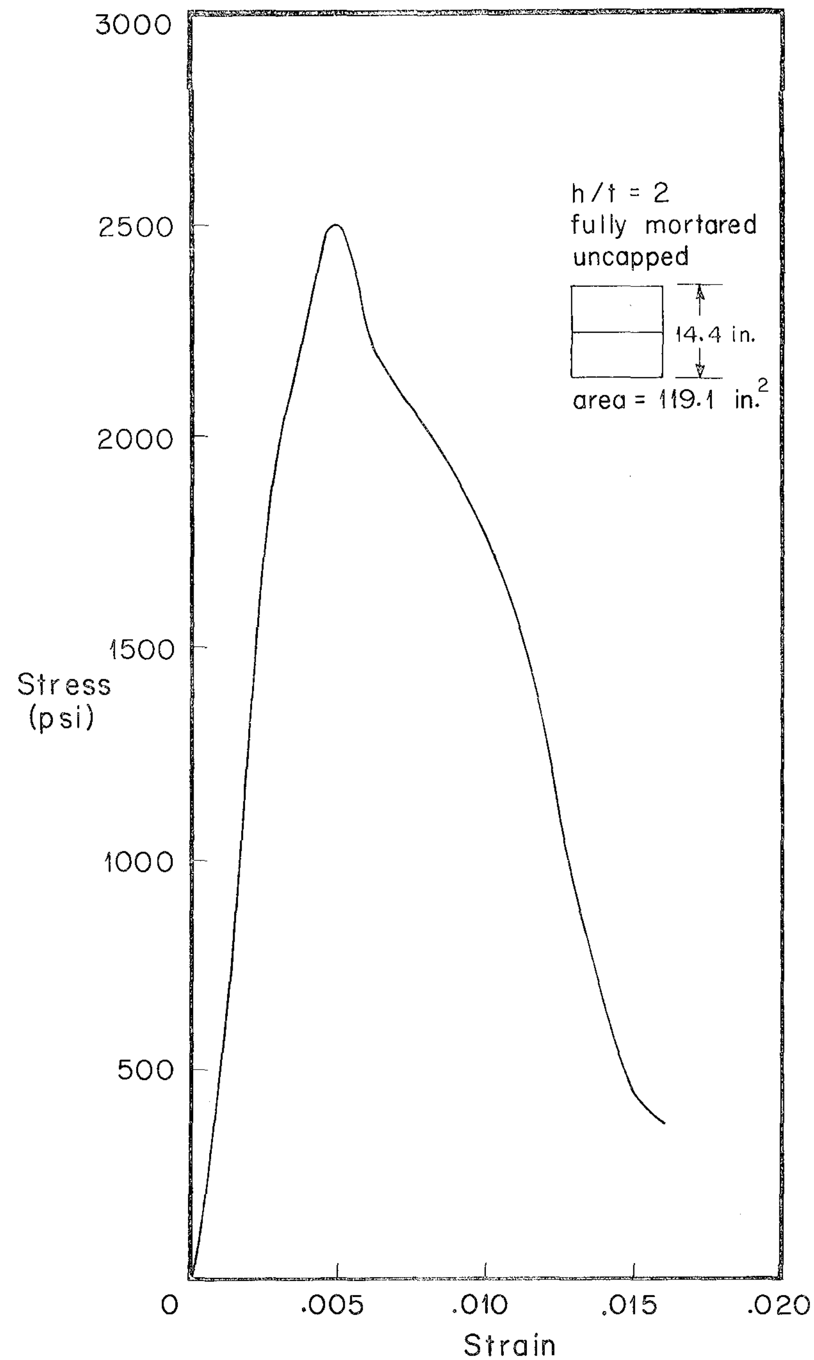
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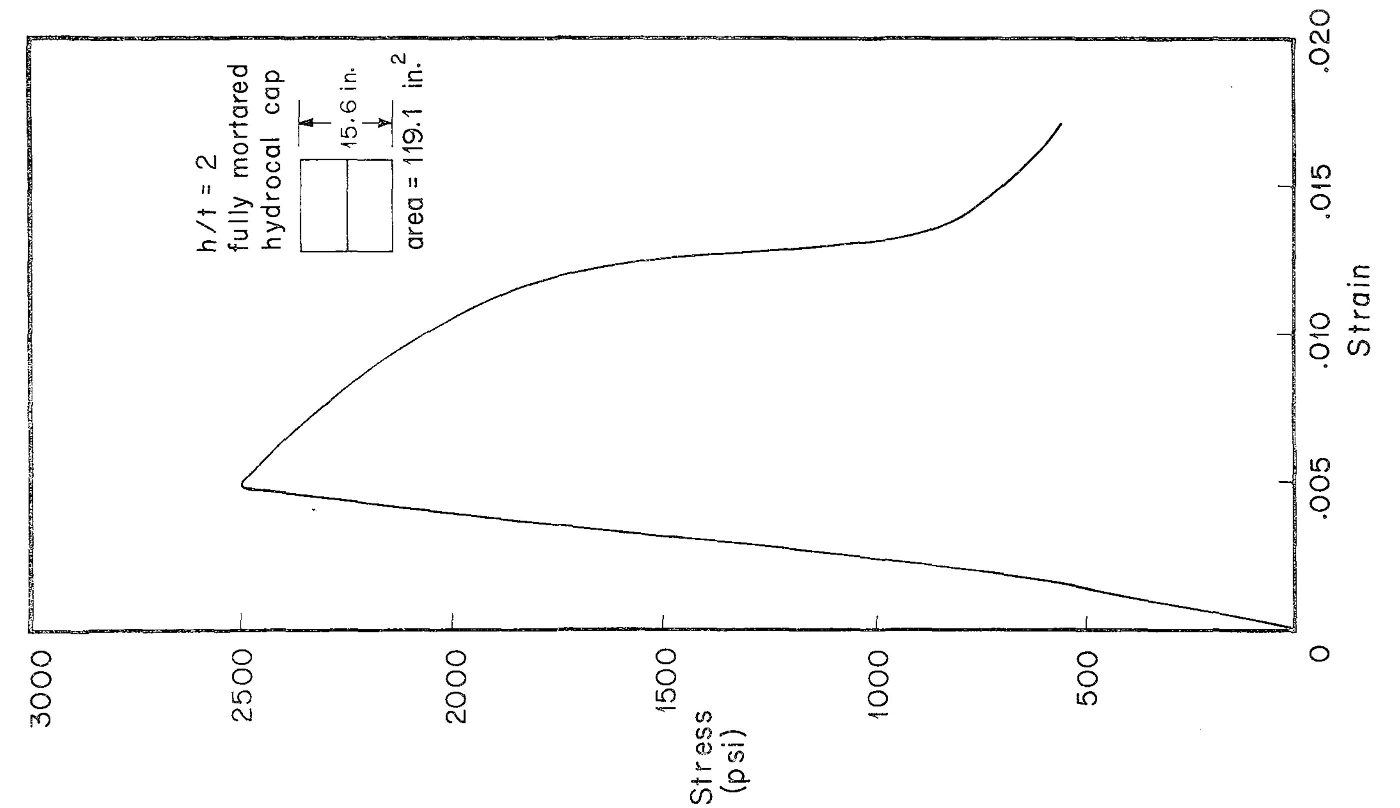
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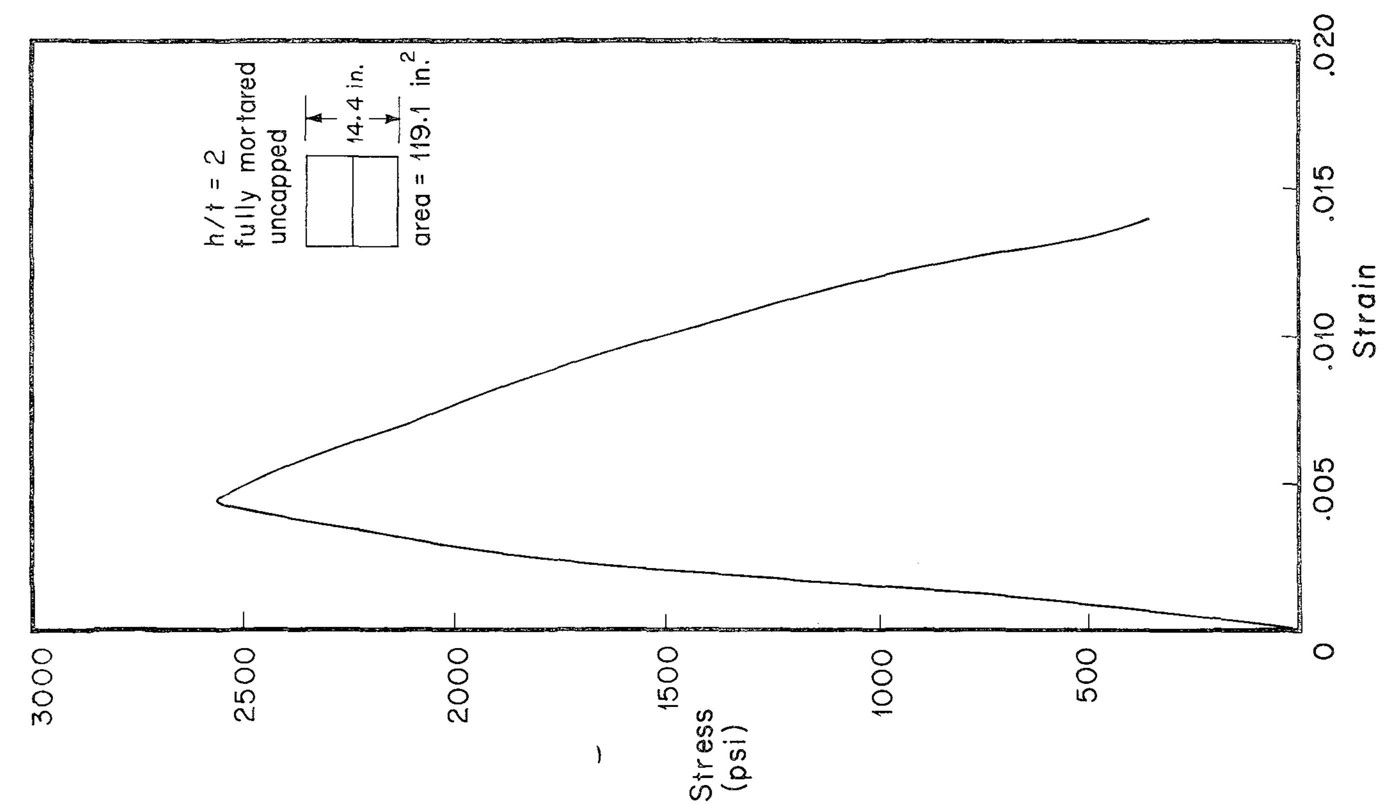
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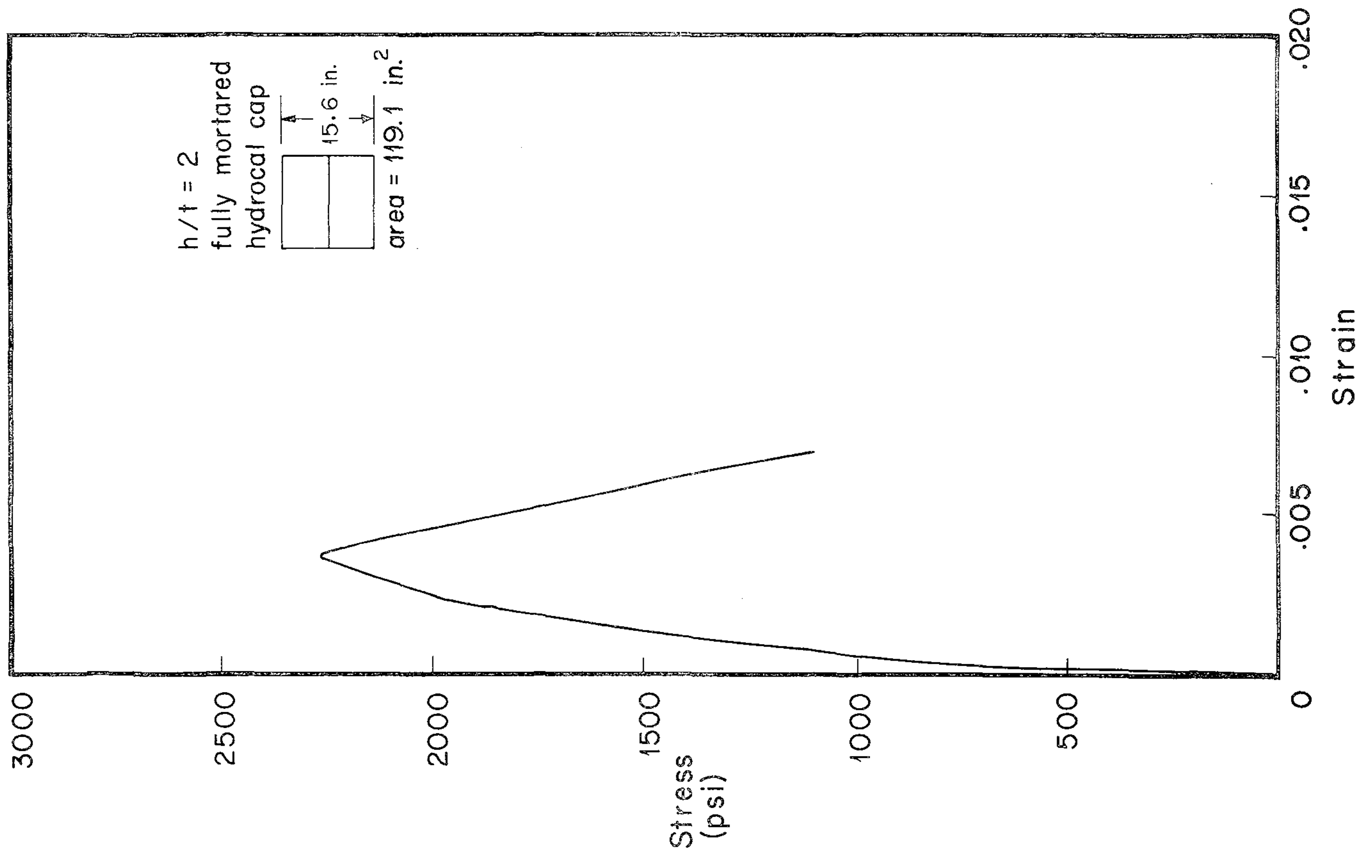
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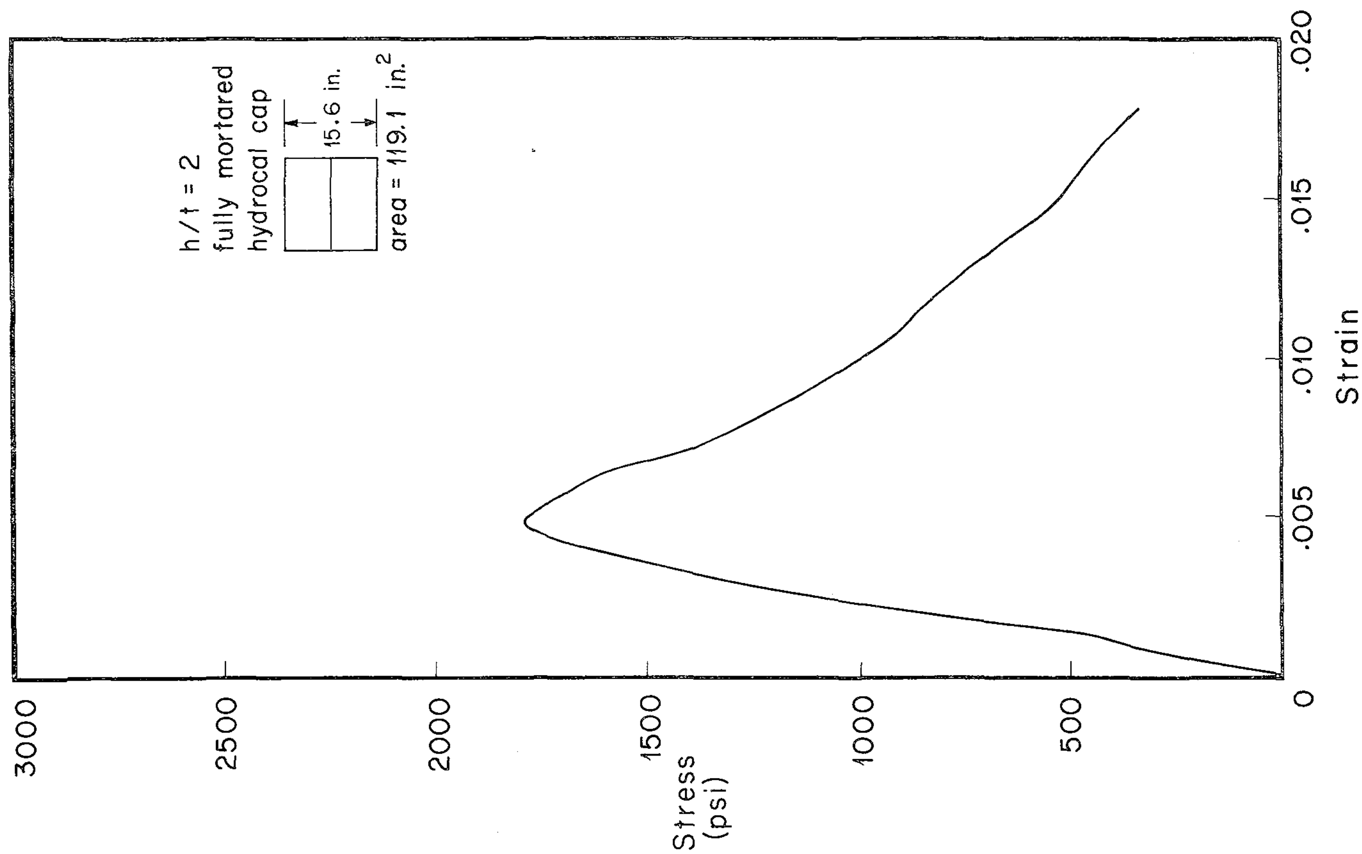
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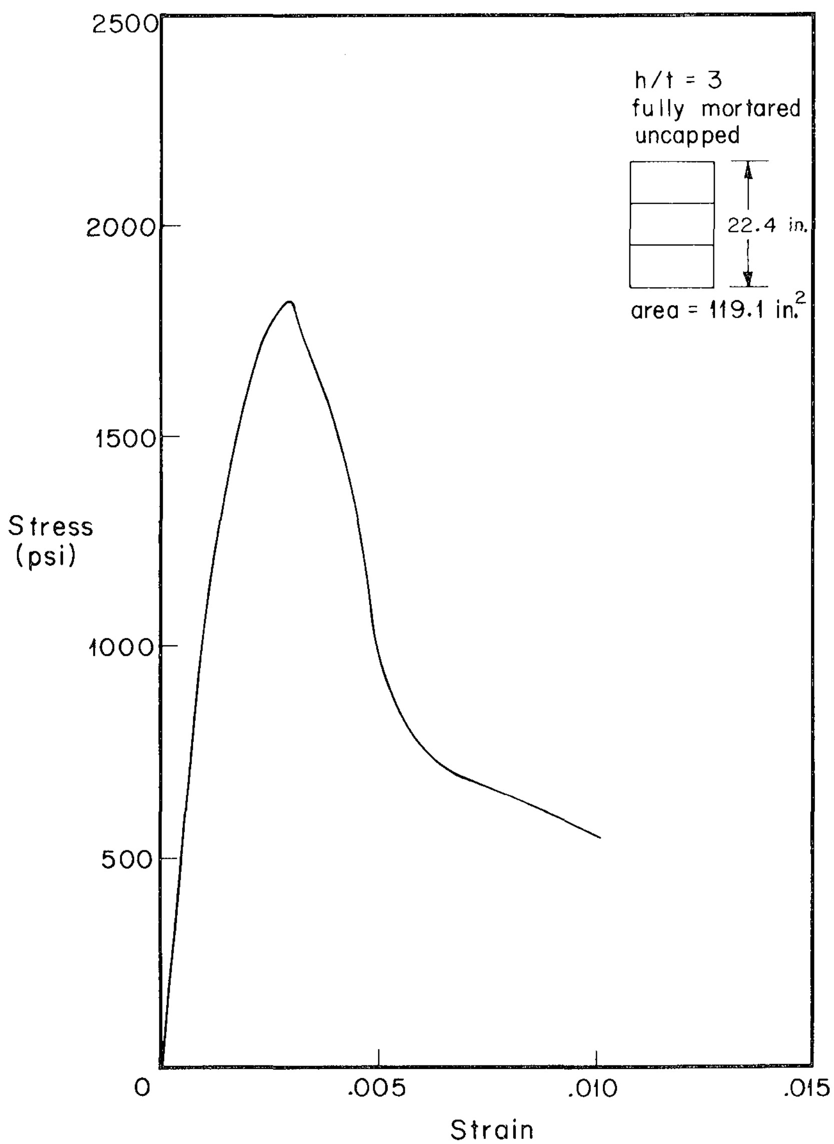
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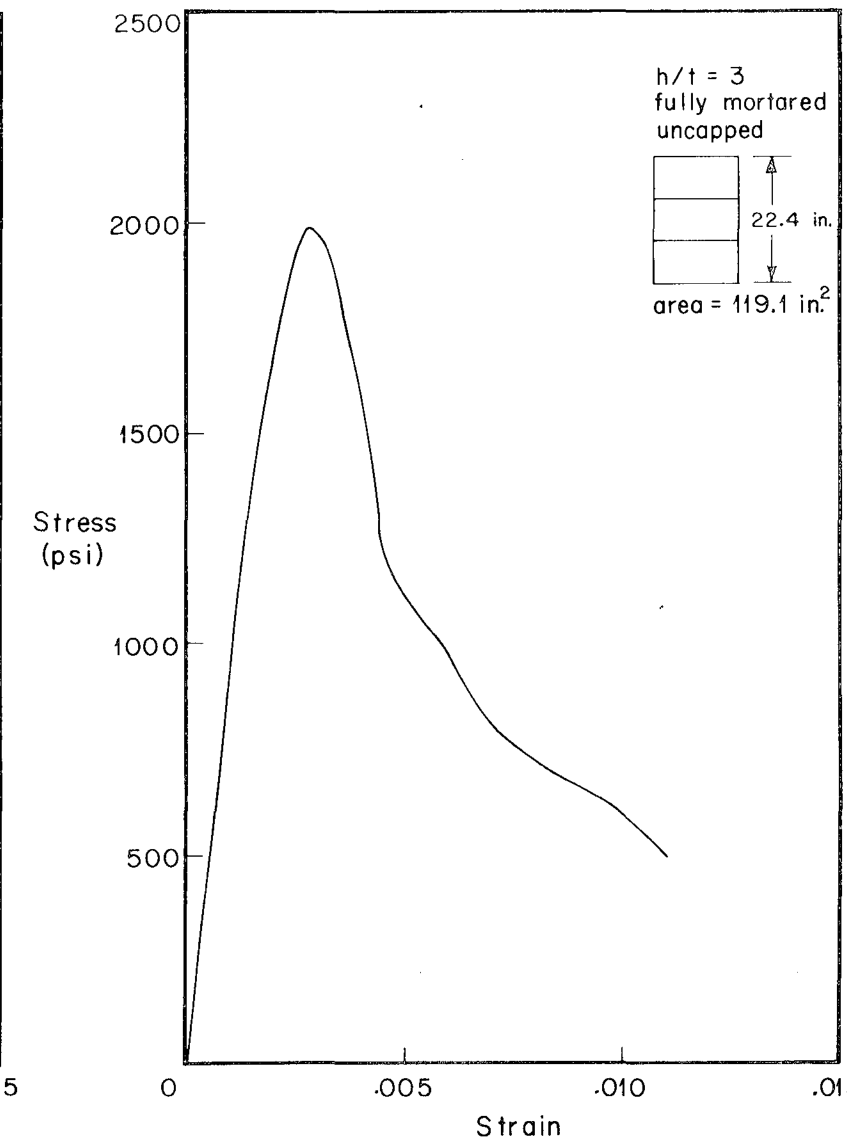
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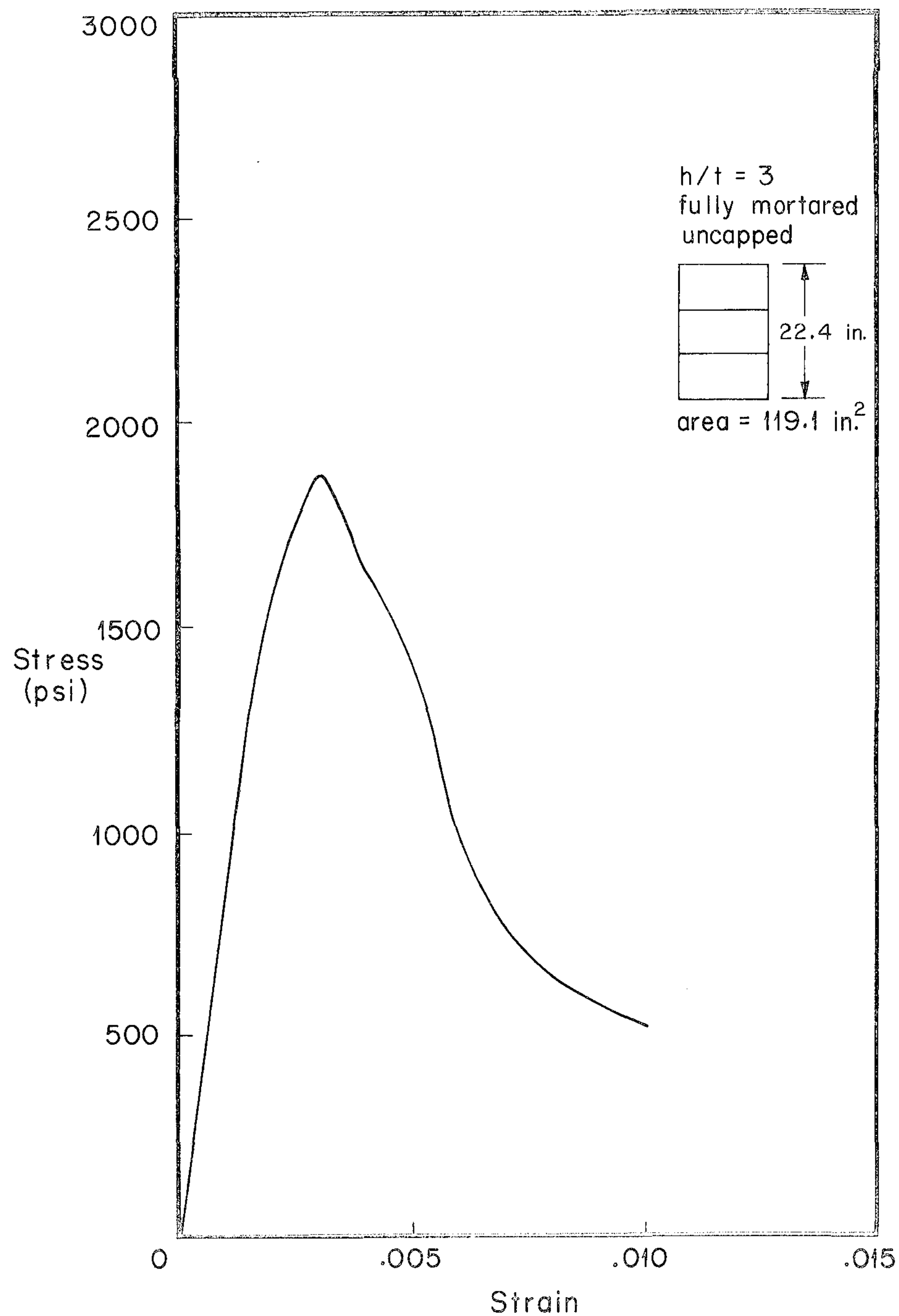
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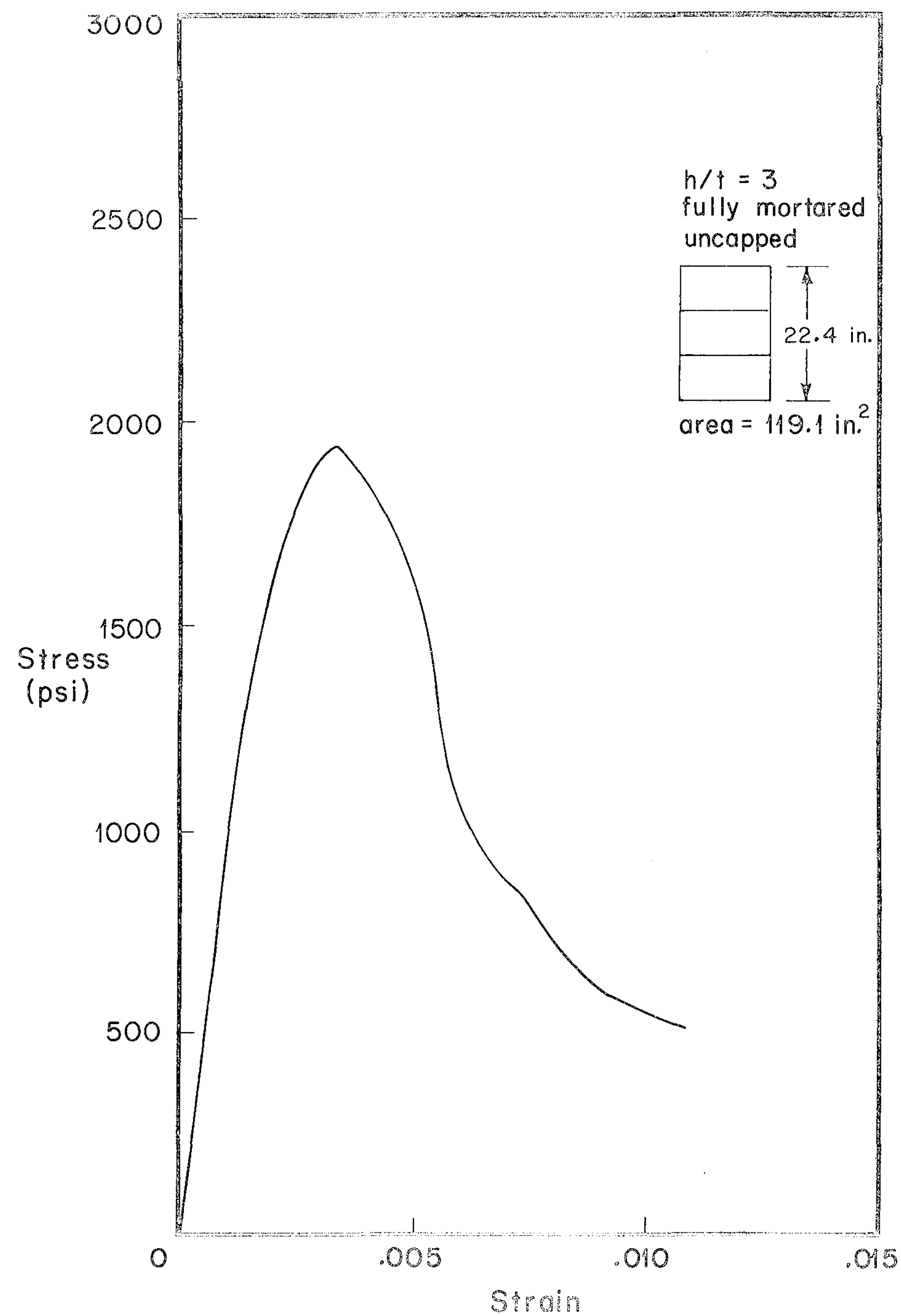
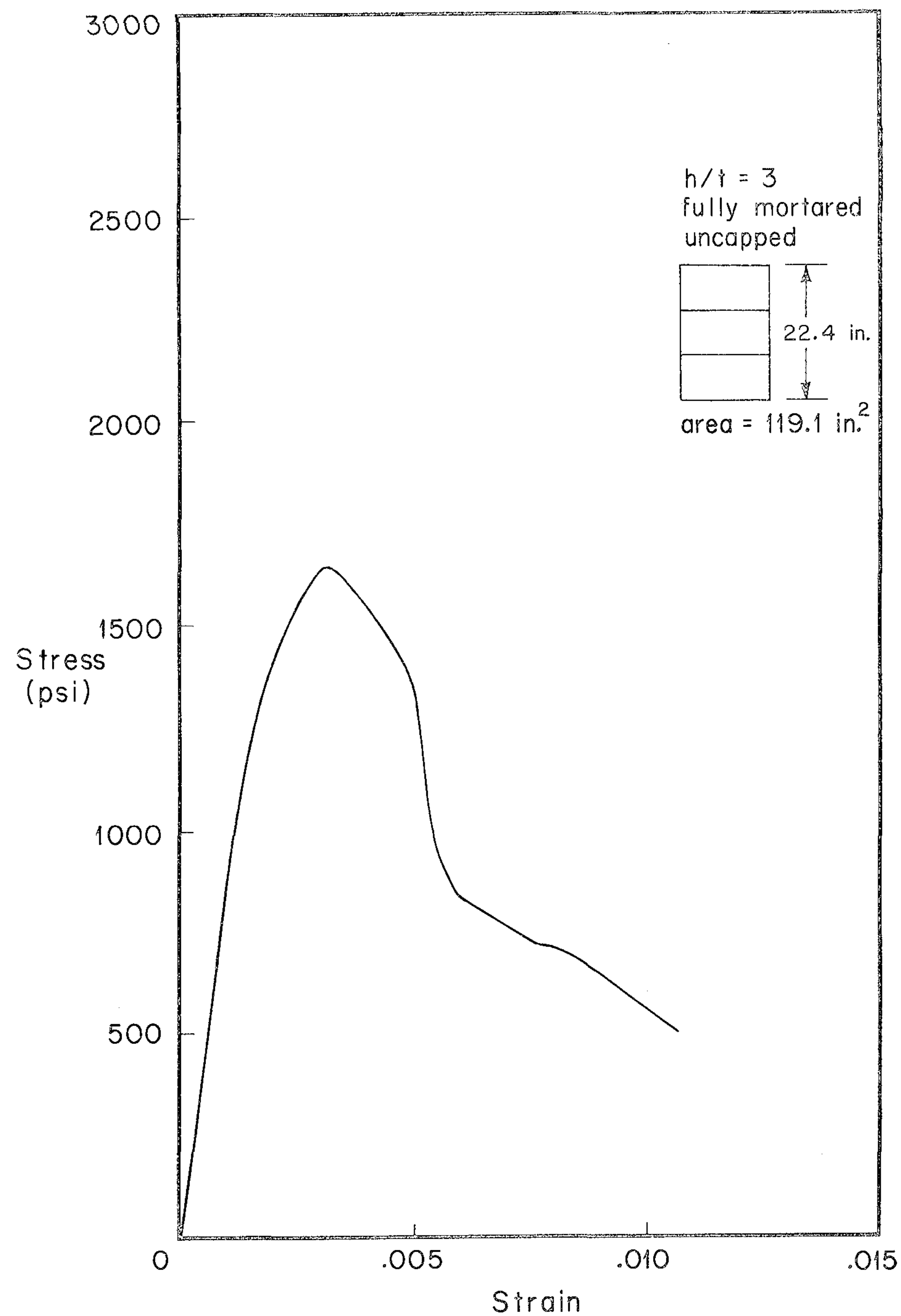
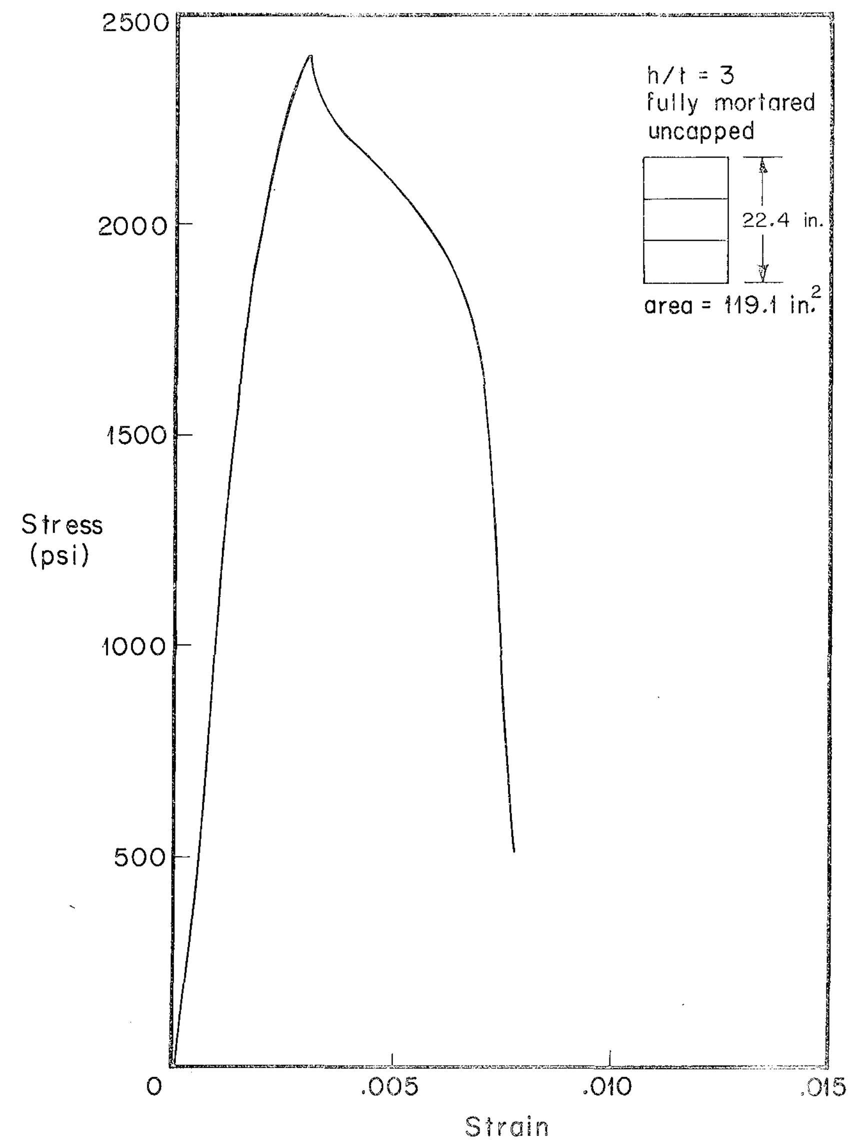
Stress
(psi)
h/t = 3
fully mortared
uncapped
22.4 in.
area = 119.1 in.2
.005
.010
.015
Strain

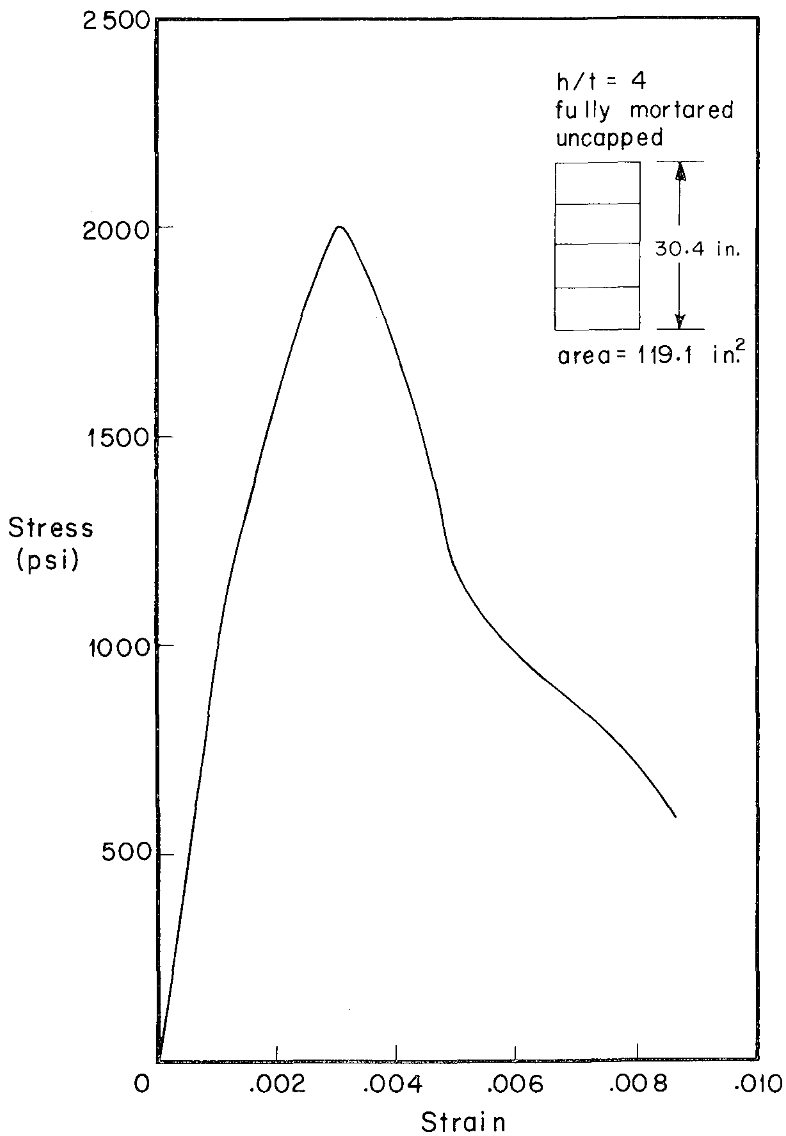

3000
2500
2000
1500
1000
500
0
Stress
(psi)
h/t = 3
fully mortared
uncapped
22.4 in.
area = 119.1 in.2
.005
.010
.015
Strain

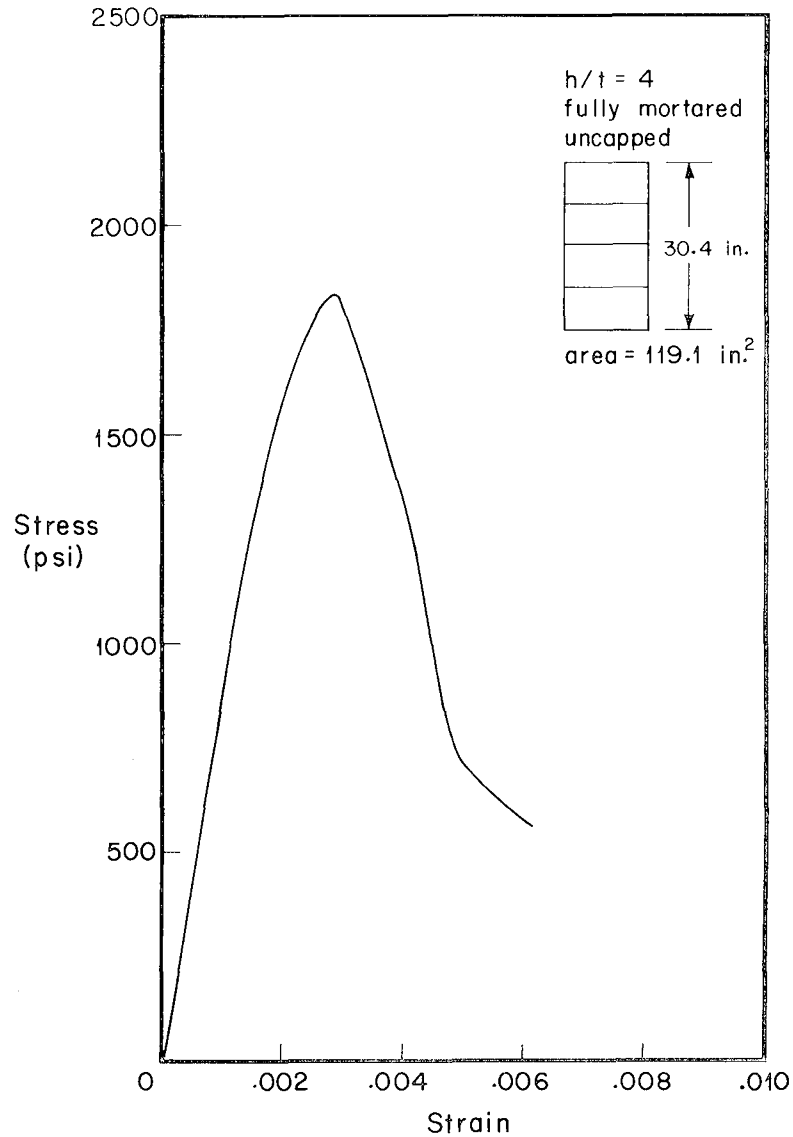


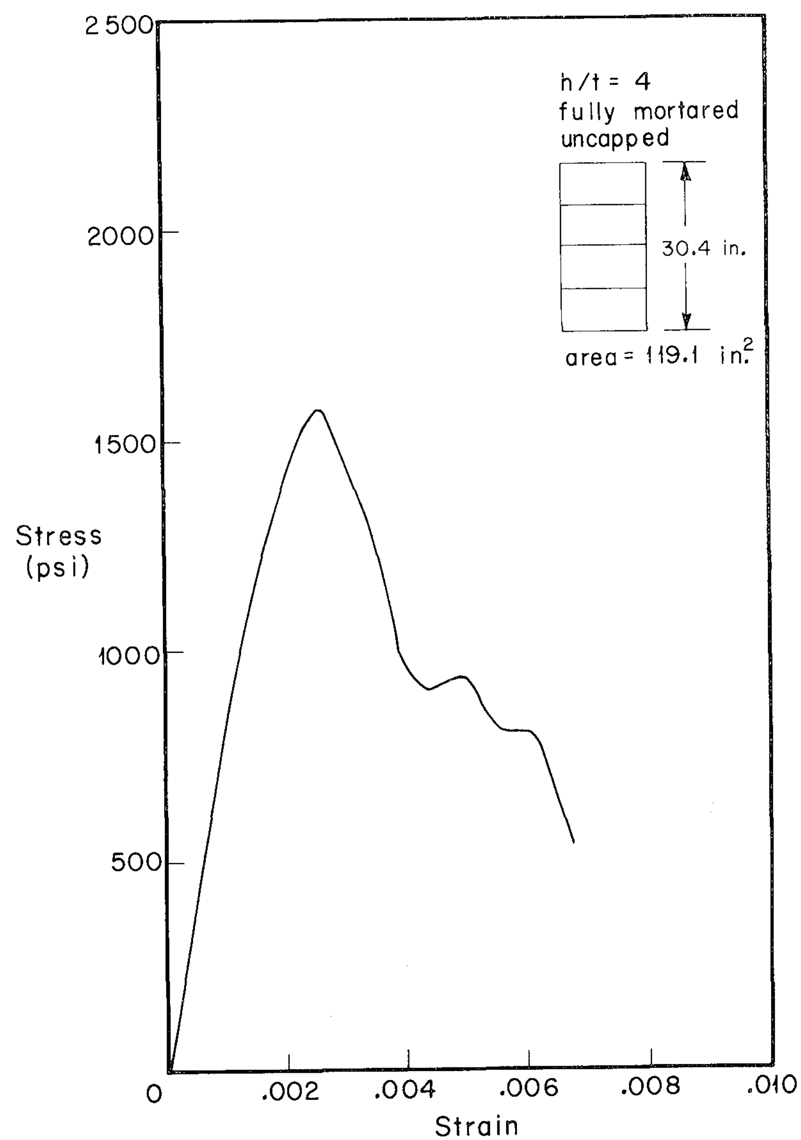
3000
2500
2000
1500
1000
500
0
.005
.010
.015
Stress (psi)
Strain
h/t = 3
fully mortared
uncapped
22.4 in.
area = 119.1 in.^{2}

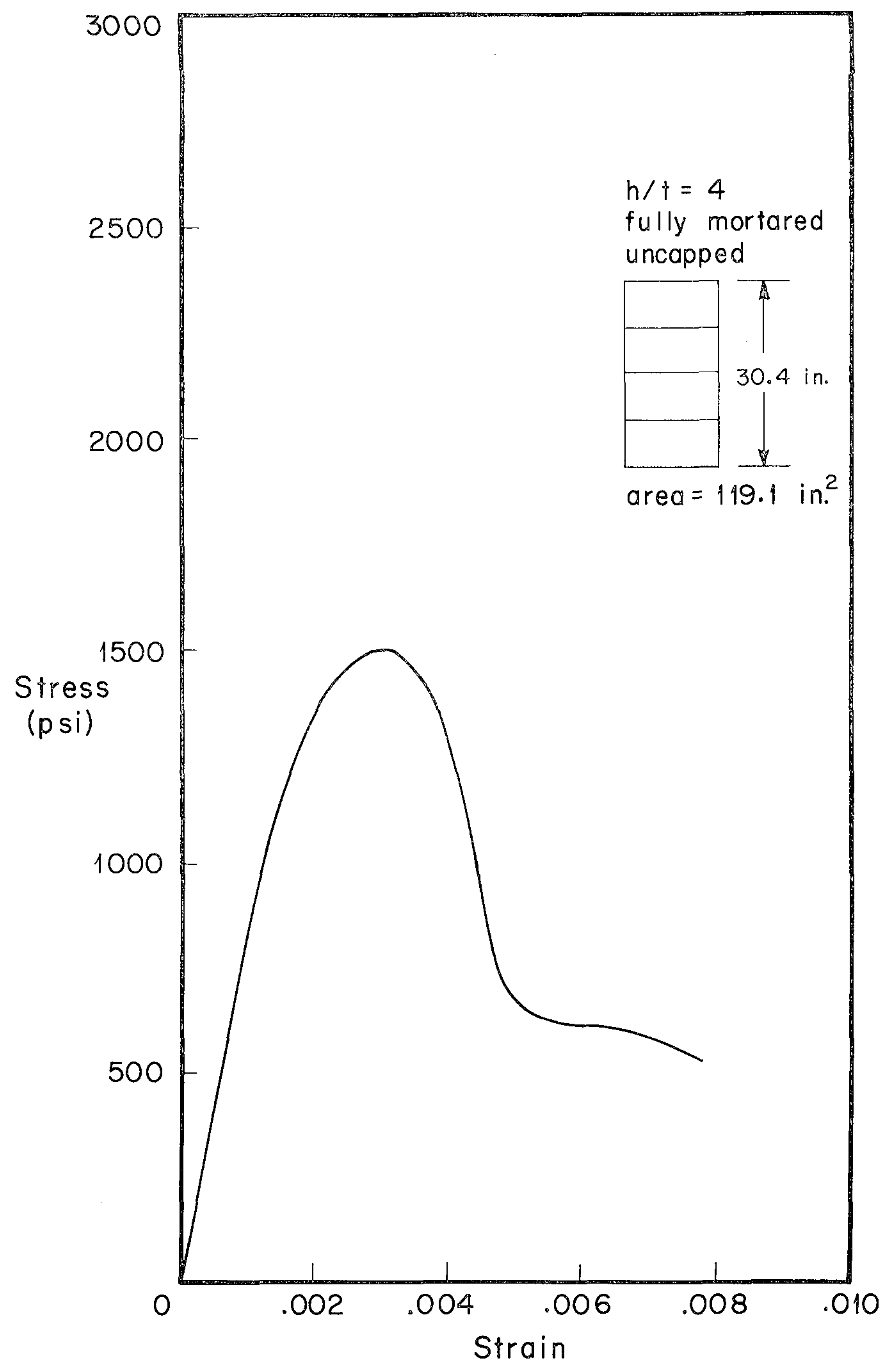


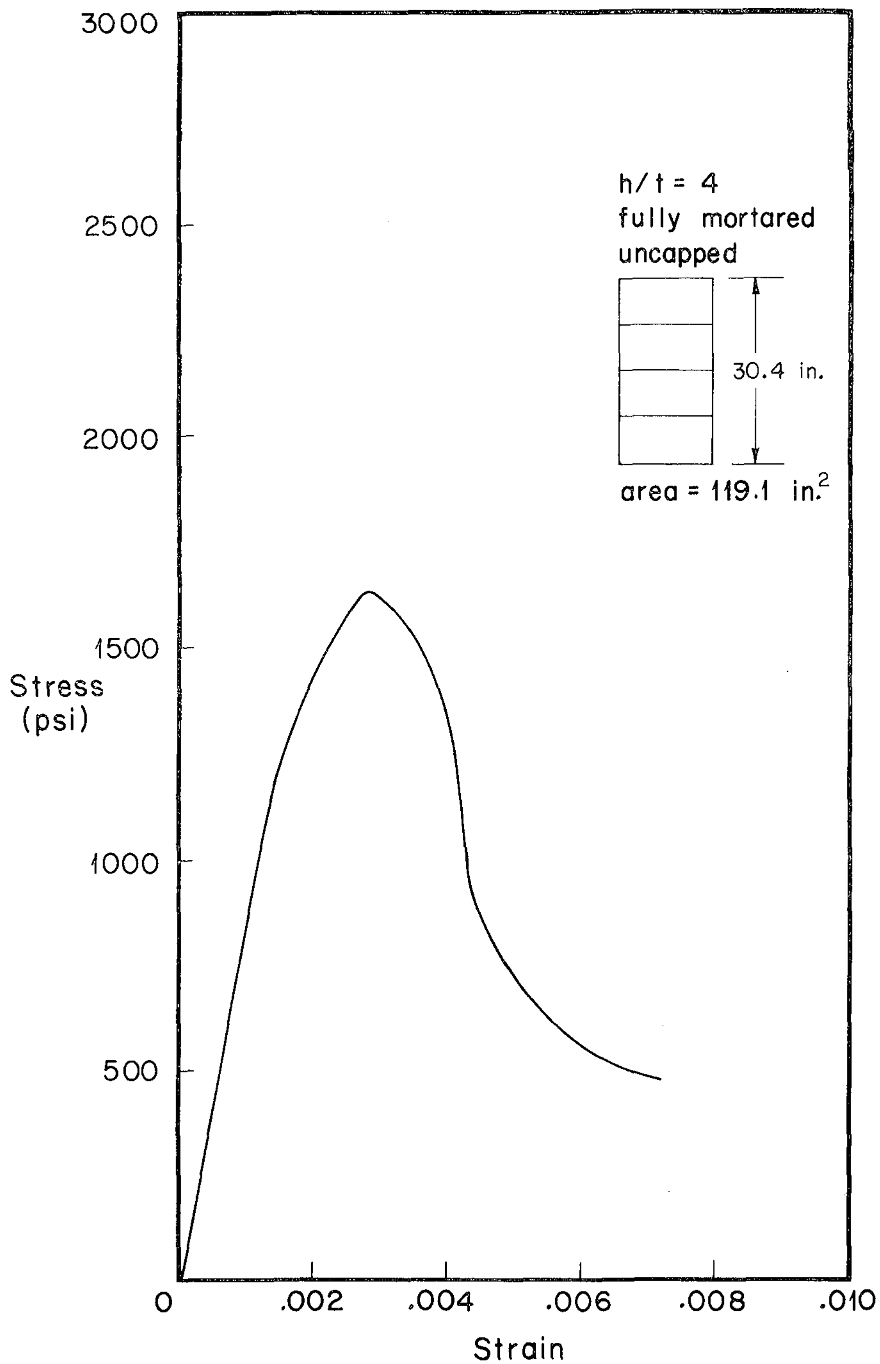
2500
2000
1500
1000
500
0
.005
.010
.015
Stress (psi)
Strain
h/t = 3
fully mortared
uncapped
22.4 in.
area = 119.1 in.^{2}

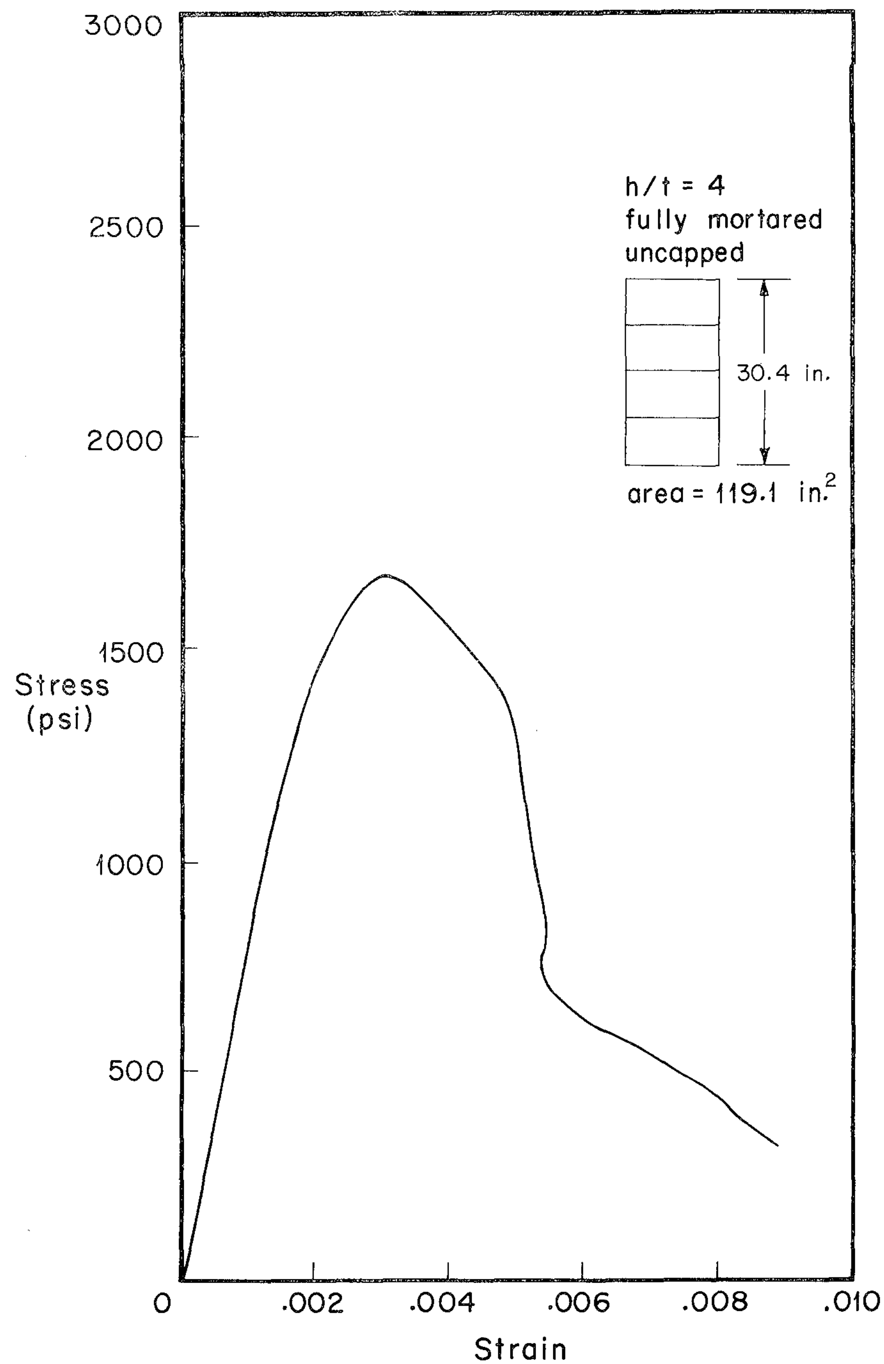

h/t = 4
fully mortared
uncapped
30.4 in.
area = 119.1 in.^{2}
2500
2000
1500
1000
500
0
Stress (psi)
.002
.004
.006
.008
.010
Strain

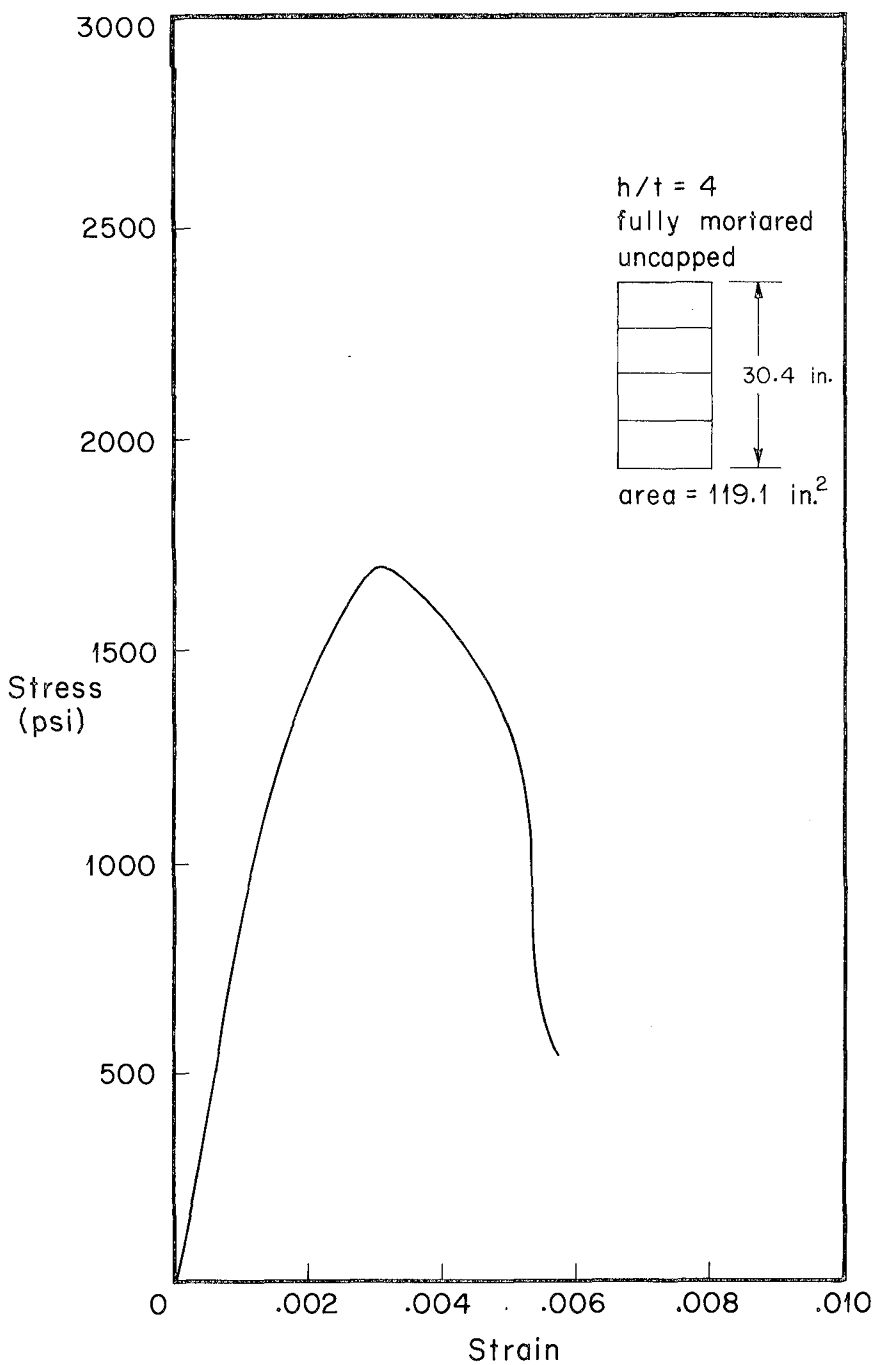

h/t = 4
fully mortared
uncapped
30.4 in.
area = 119.1 in.^{2}
2500
2000
1500
1000
500
0
Stress (psi)
.002
.004
.006
.008
.010
Strain

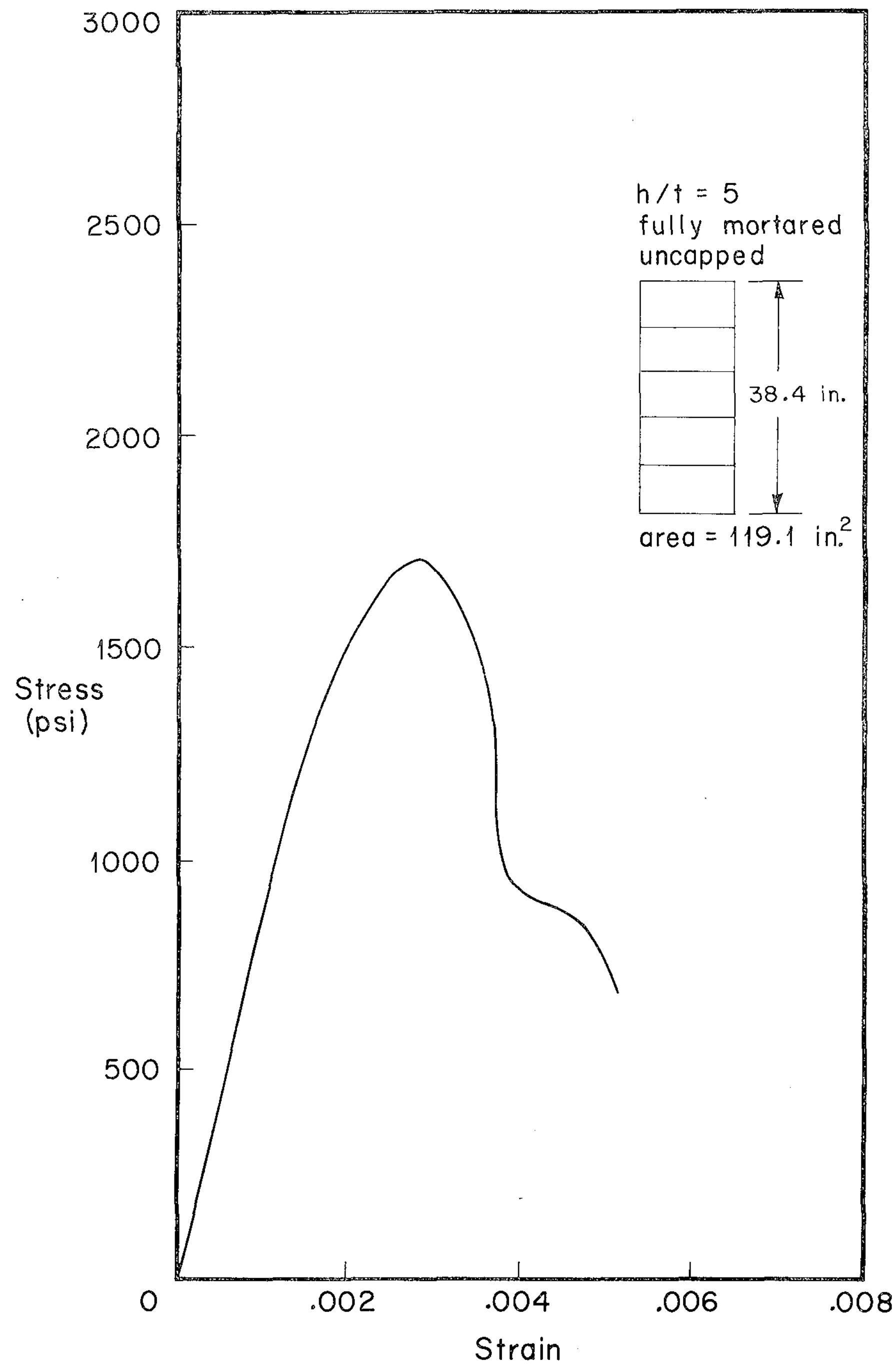

2500
2000
1500
1000
500
0
Stress
(psi)
.002
.004
.006
.008
.010
Strain
h/t = 4
fully mortared
uncapped
30.4 in.
area = 119.1 in.^{2}

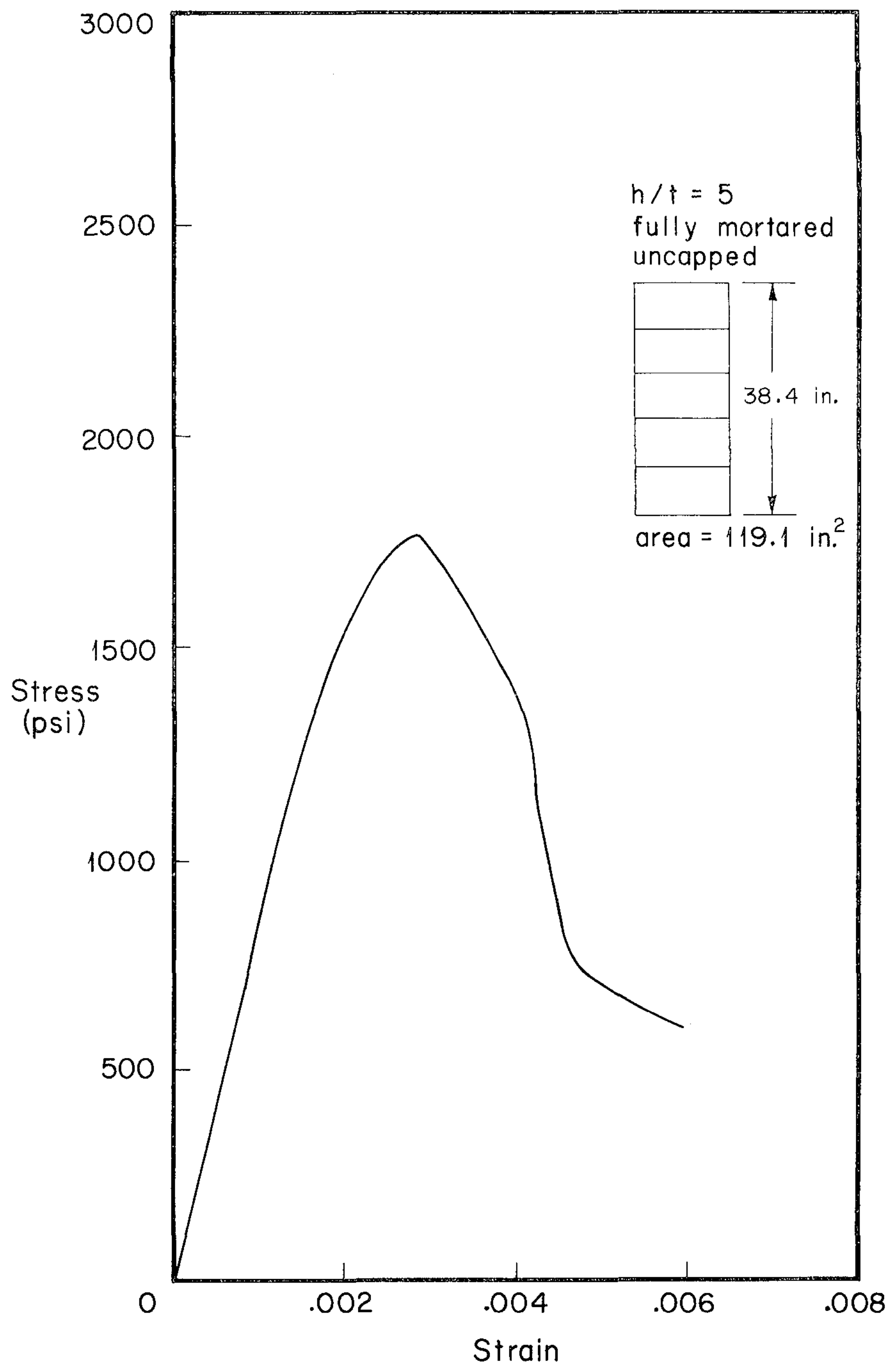

3000
2500
2000
1500
1000
500
0
Stress
(psi)
.002
.004
.006
.008
.010
Strain
h/t = 4
fully mortared
uncapped
30.4 in.
area = 119.1 in.^{2}


h/t = 4
fully mortared
uncapped
30.4 in.
area = 119.1 in.^{2}
Stress (psi)
3000
2500
2000
1500
1000
500
0
.002
.004
.006
.008
.010
Strain

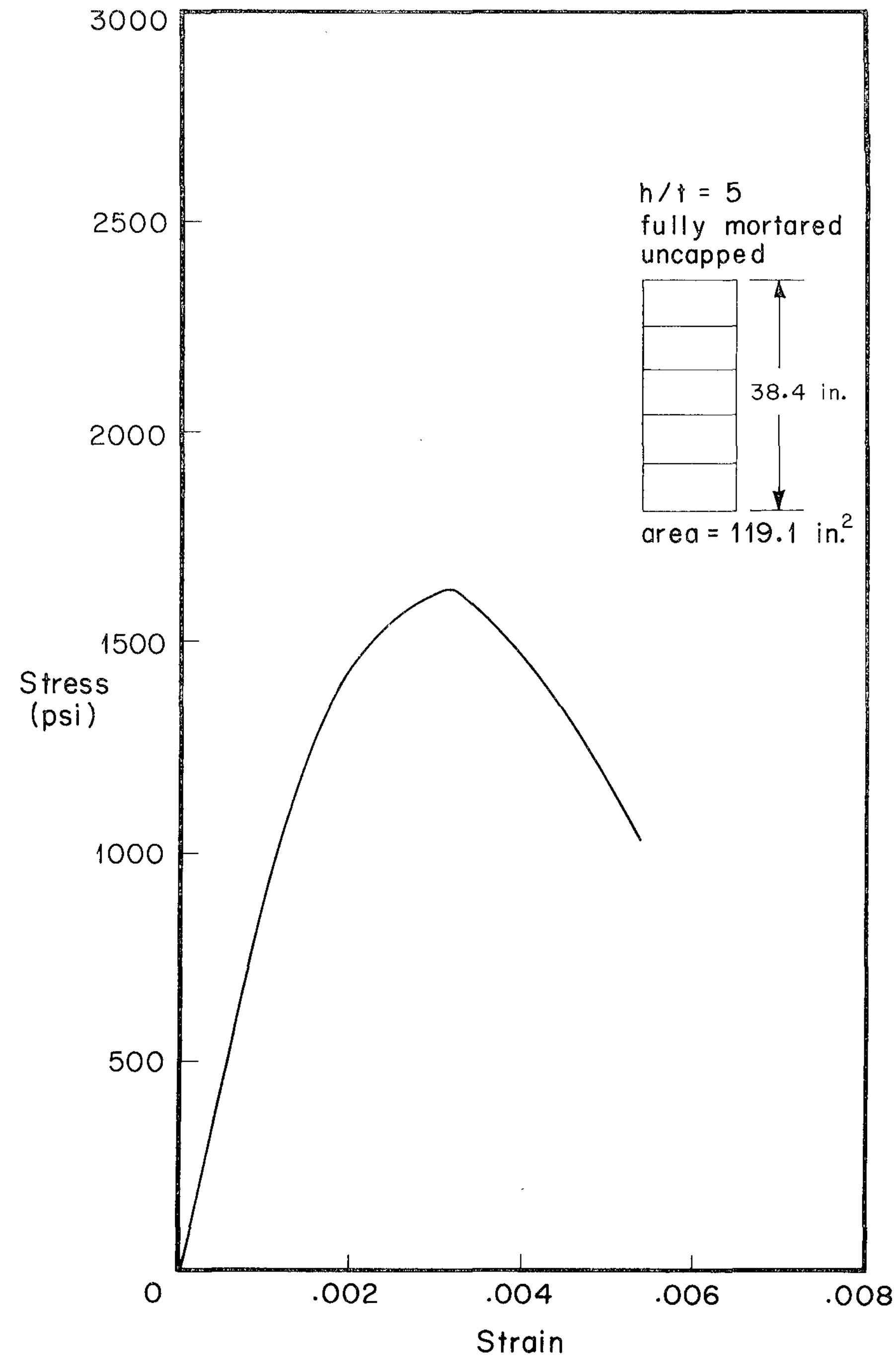

h/t = 4
fully mortared
uncapped
30.4 in.
area = 119.1 in.^{2}
Stress (psi)
3000
2500
2000
1500
1000
500
0
.002
.004
.006
.008
.010
Strain


h/t = 4
fully mortared
uncapped
30.4 in.
area = 119.1 in.²
Stress (psi)
Strain
3000
2500
2000
1500
1000
500
0
.002
.004
.006
.008
.010

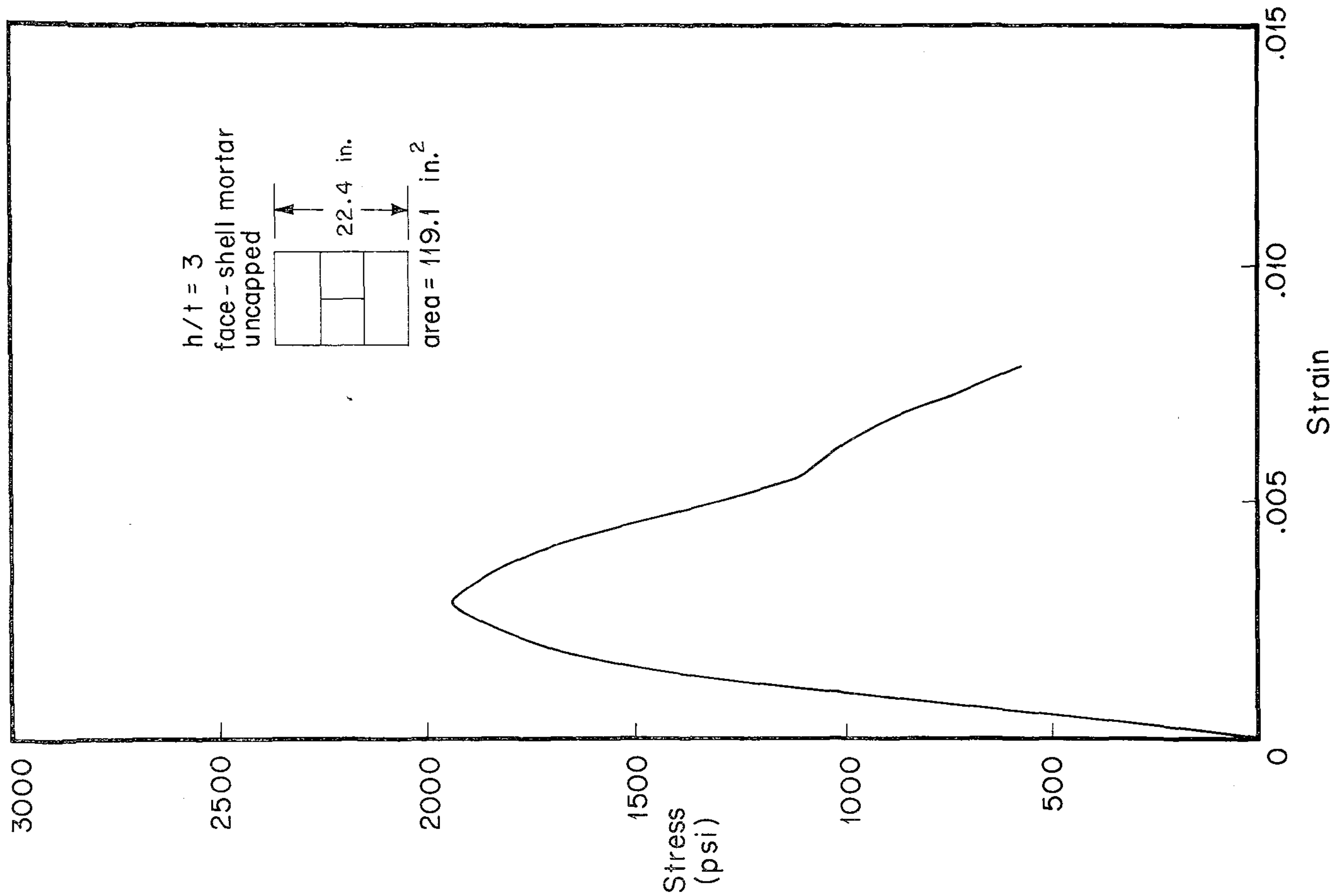

h/t = 5
fully mortared
uncapped
38.4 in.
area = 119.1 in.²
Stress (psi)
Strain
3000
2500
2000
1500
1000
500
0
.002
.004
.006
.008

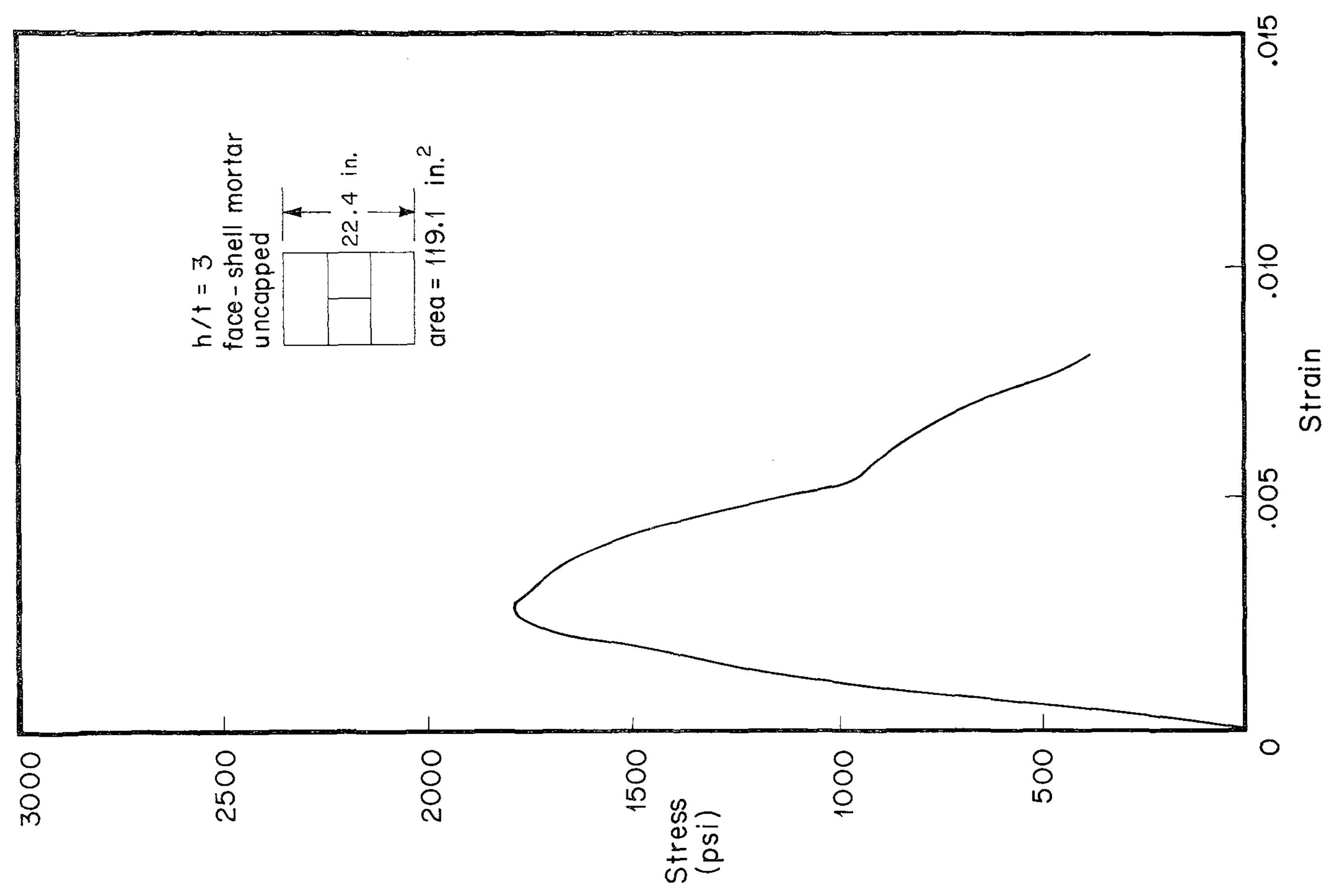


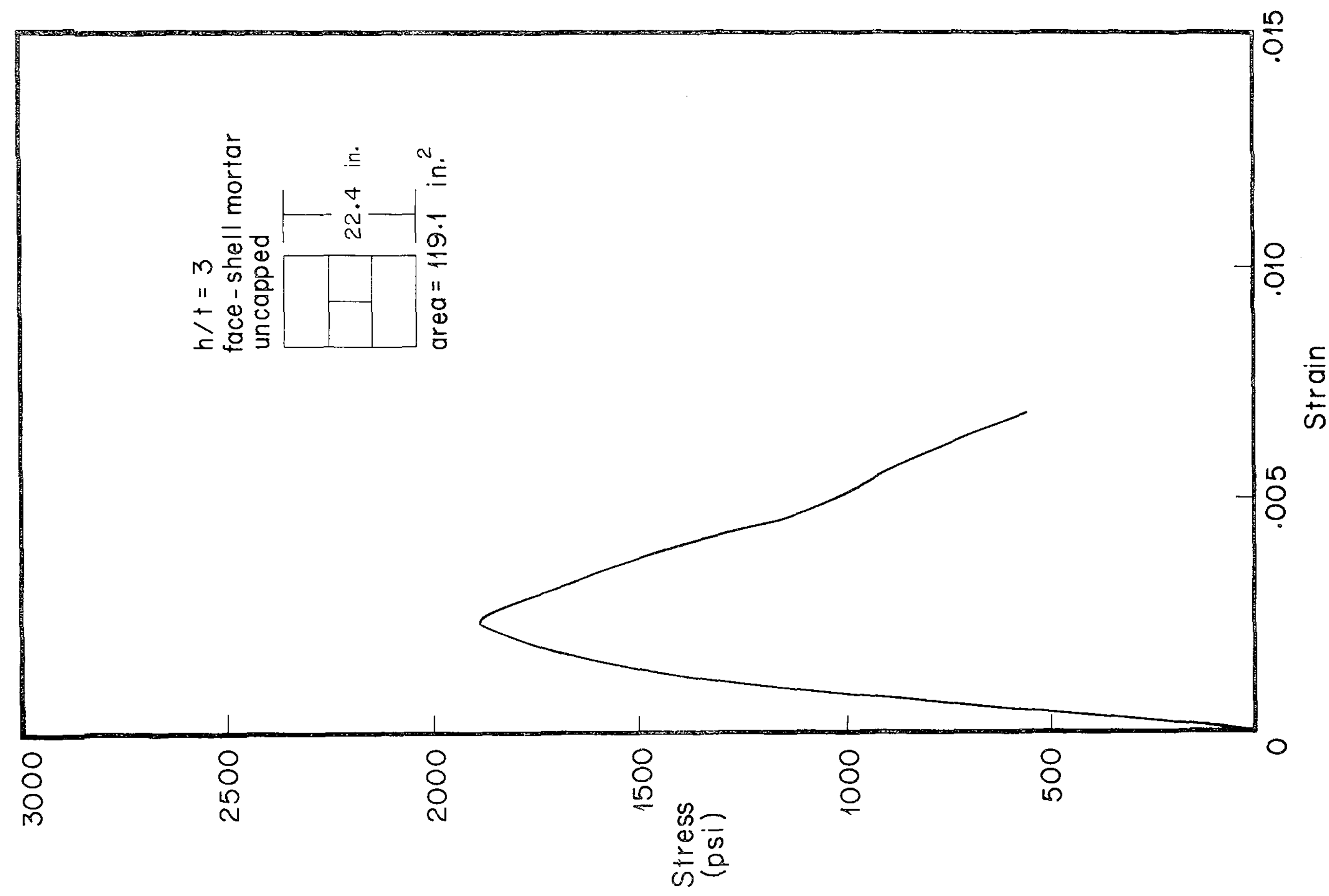
h/t = 5
fully mortared
uncapped
38.4 in.
area = 119.1 in.²
Stress (psi)
Strain
3000
2500
2000
1500
1000
500
0
.002
.004
.006
.008

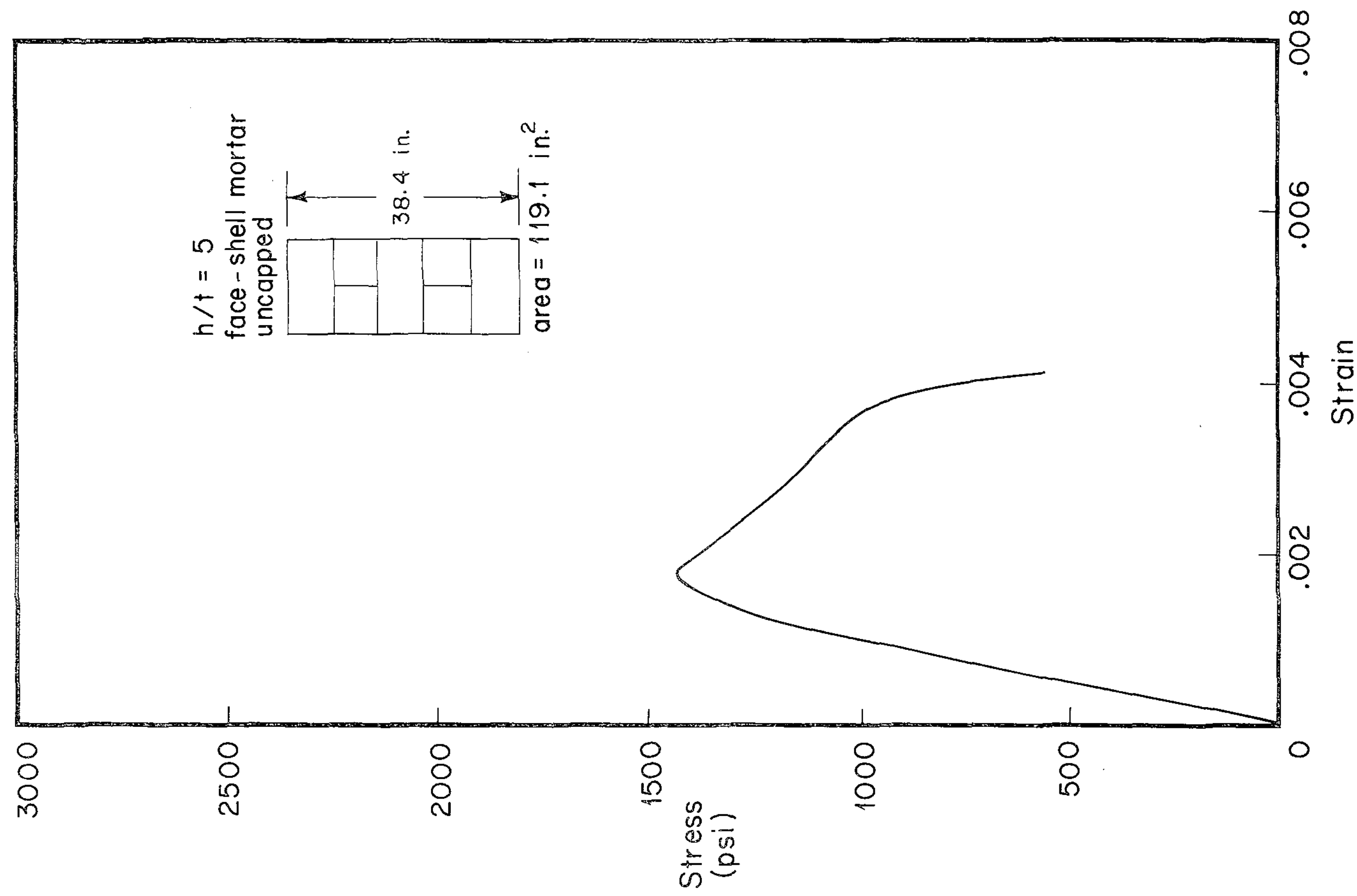


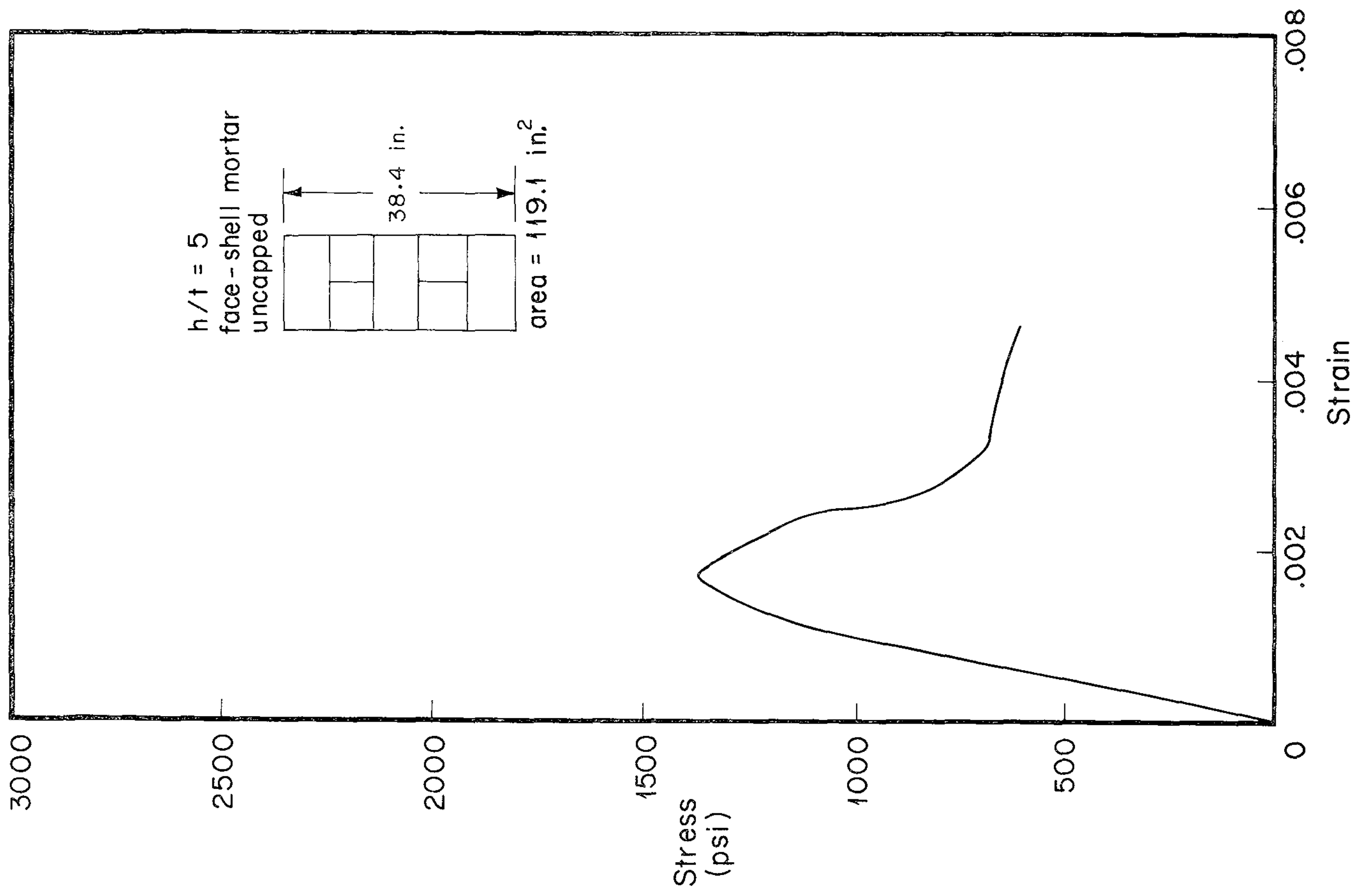
h/t = 5
fully mortared
uncapped
38.4 in.
area = 119.1 in.²
Stress (psi)
Strain
3000
2500
2000
1500
1000
500
0
.002
.004
.006
.008

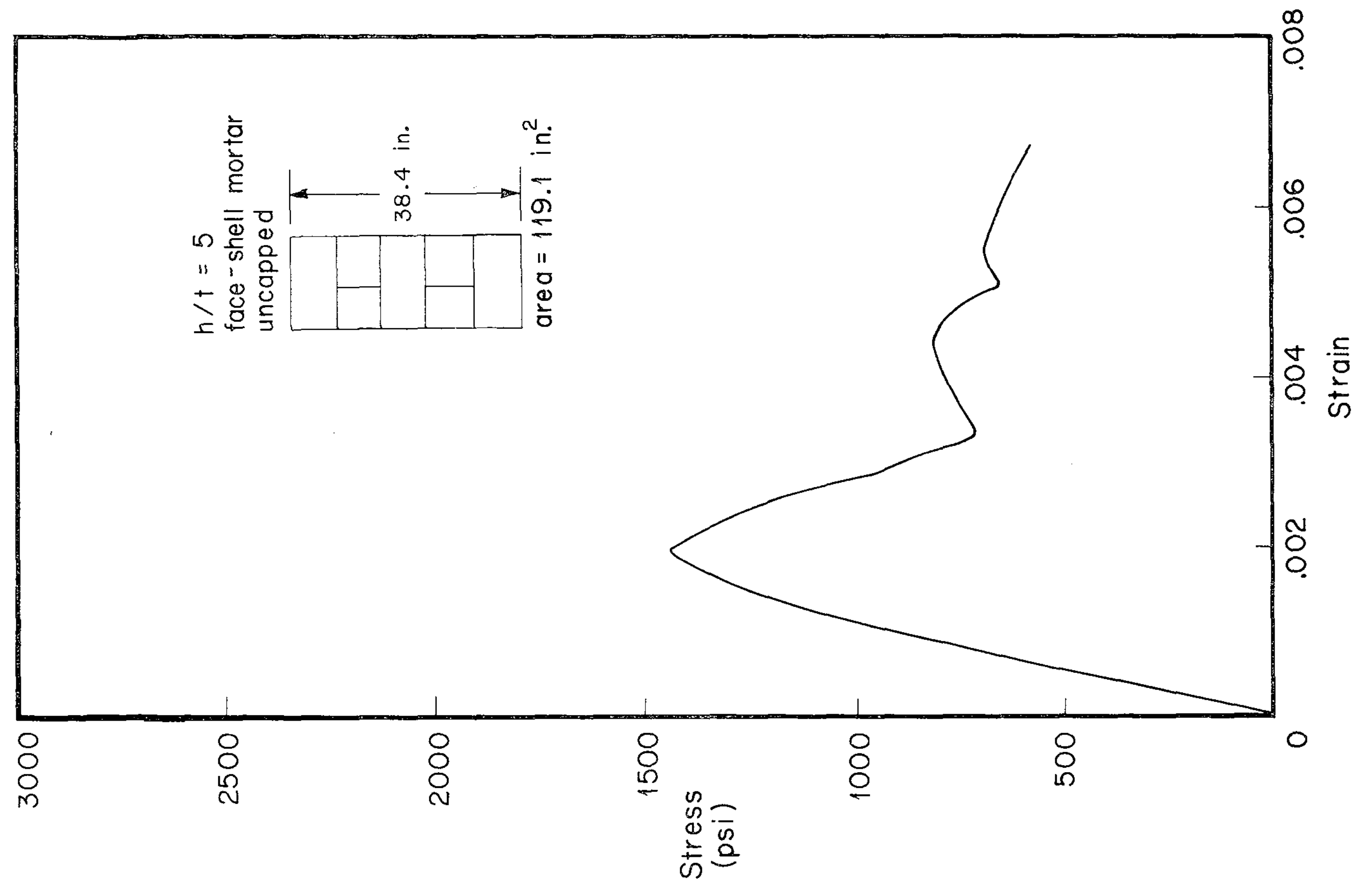

h/t = 3
face - shell mortar
uncapped
22.4 in.
area = 119.1 in.^{2}
Stress
(psi)
3000
2500
2000
1500
1000
500
0
.005
.010
.015
Strain

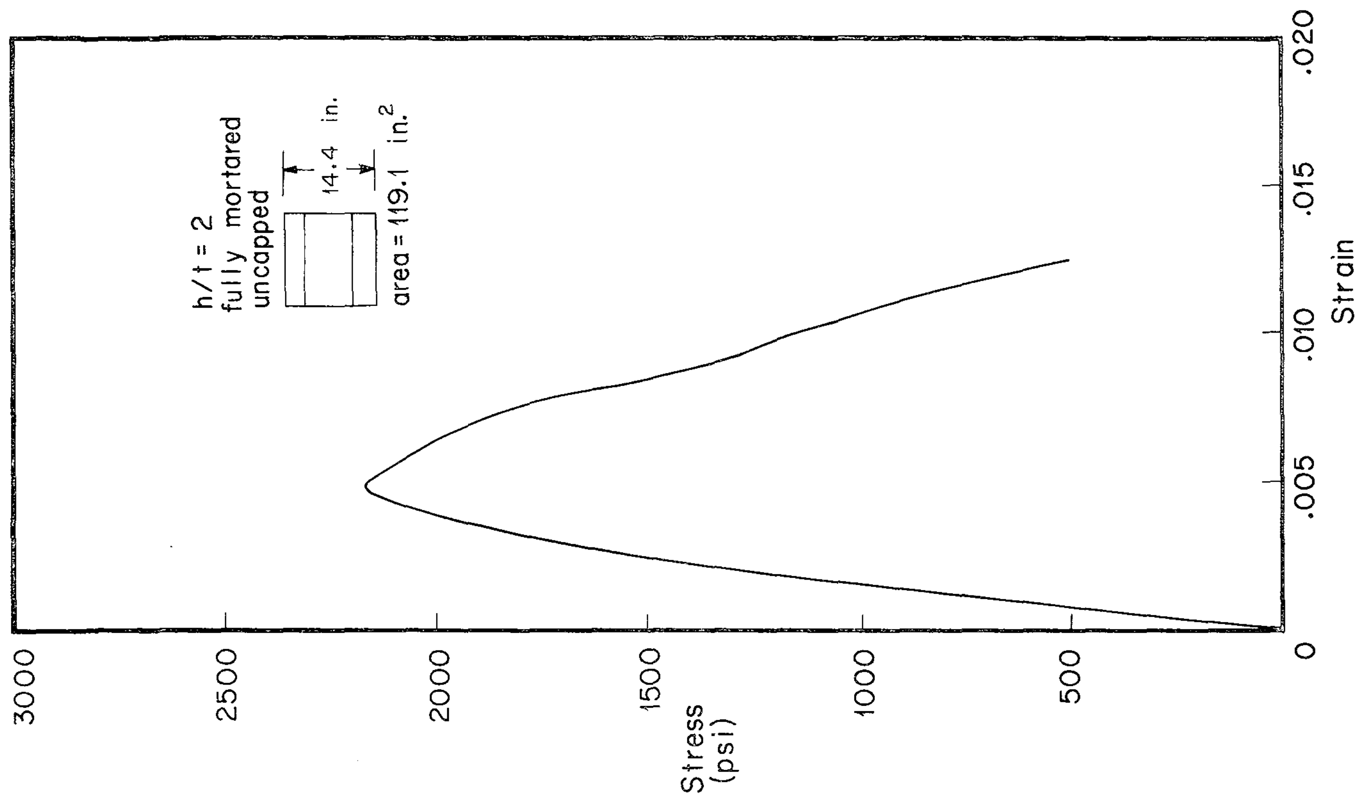

h/t = 3
face - shell mortar
uncapped
22.4 in.
area = 119.1 in.^{2}
Stress
(psi)
3000
2500
2000
1500
1000
500
0
.005
.010
.015
Strain


h/t = 3
face-shell mortar
uncapped
22.4 in.
area = 119.1 in.^{2}
Stress (psi)
3000
2500
2000
1500
1000
500
0
.005
.010
.015
Strain

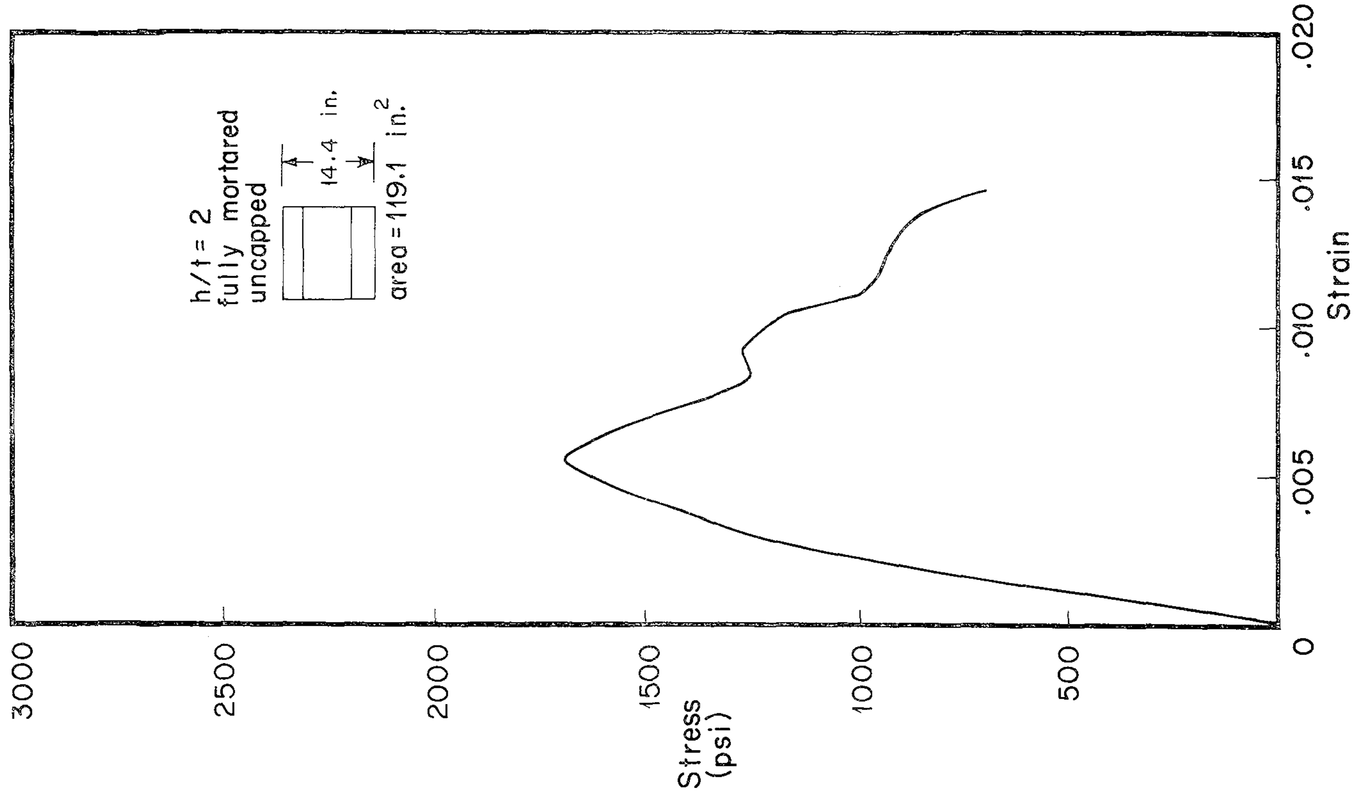

h/t = 5
face-shell mortar
uncapped
38.4 in.
area = 119.1 in.^{2}
Stress (psi)
3000
2500
2000
1500
1000
500
0
.002
.004
.006
.008
Strain


h/t = 5
face-shell mortar
uncapped
38.4 in.
area = 119.1 in.²
Stress (psi)
Strain
0
500
1000
1500
2000
2500
3000
.002
.004
.006
.008

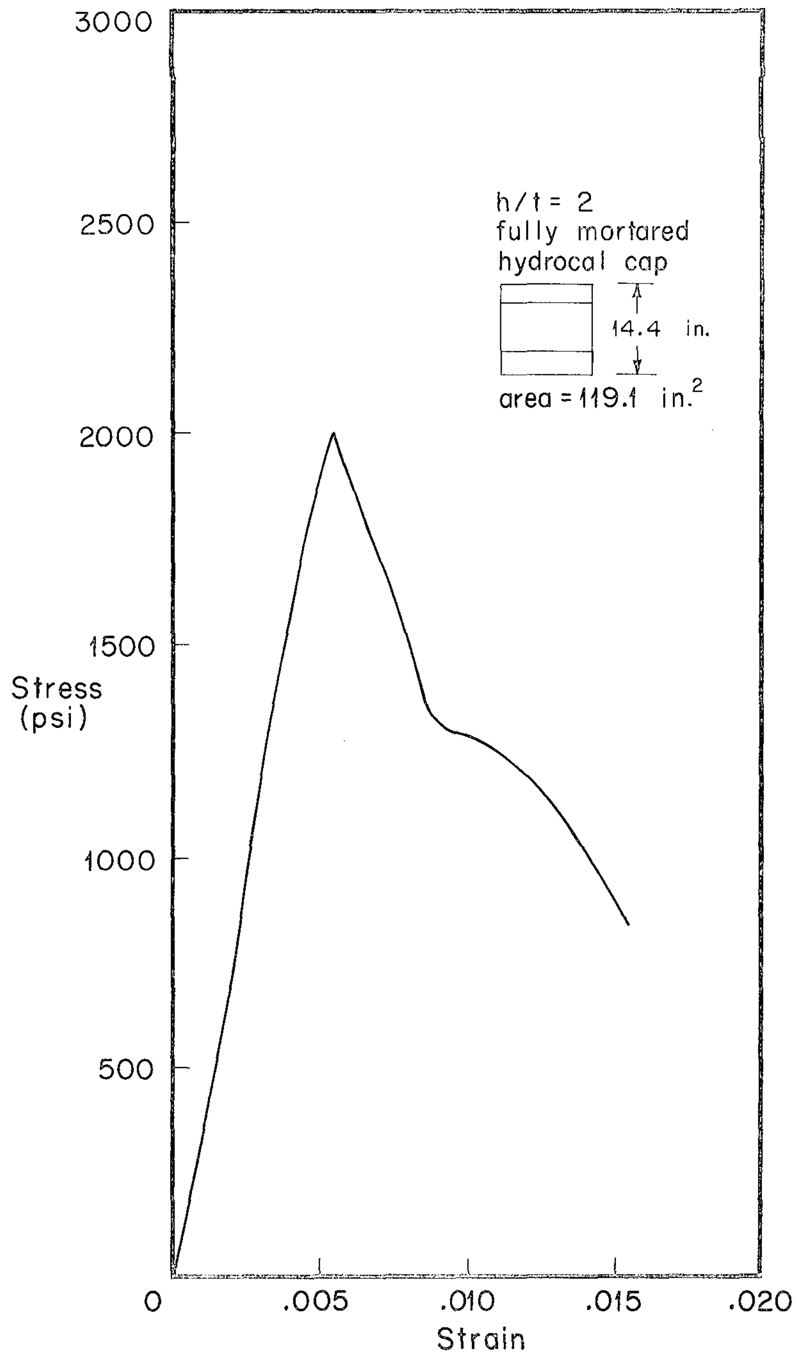

h/t = 5
face-shell mortar
uncapped
38.4 in.
area = 119.1 in.²
Stress (psi)
Strain
0
500
1000
1500
2000
2500
3000
.002
.004
.006
.008

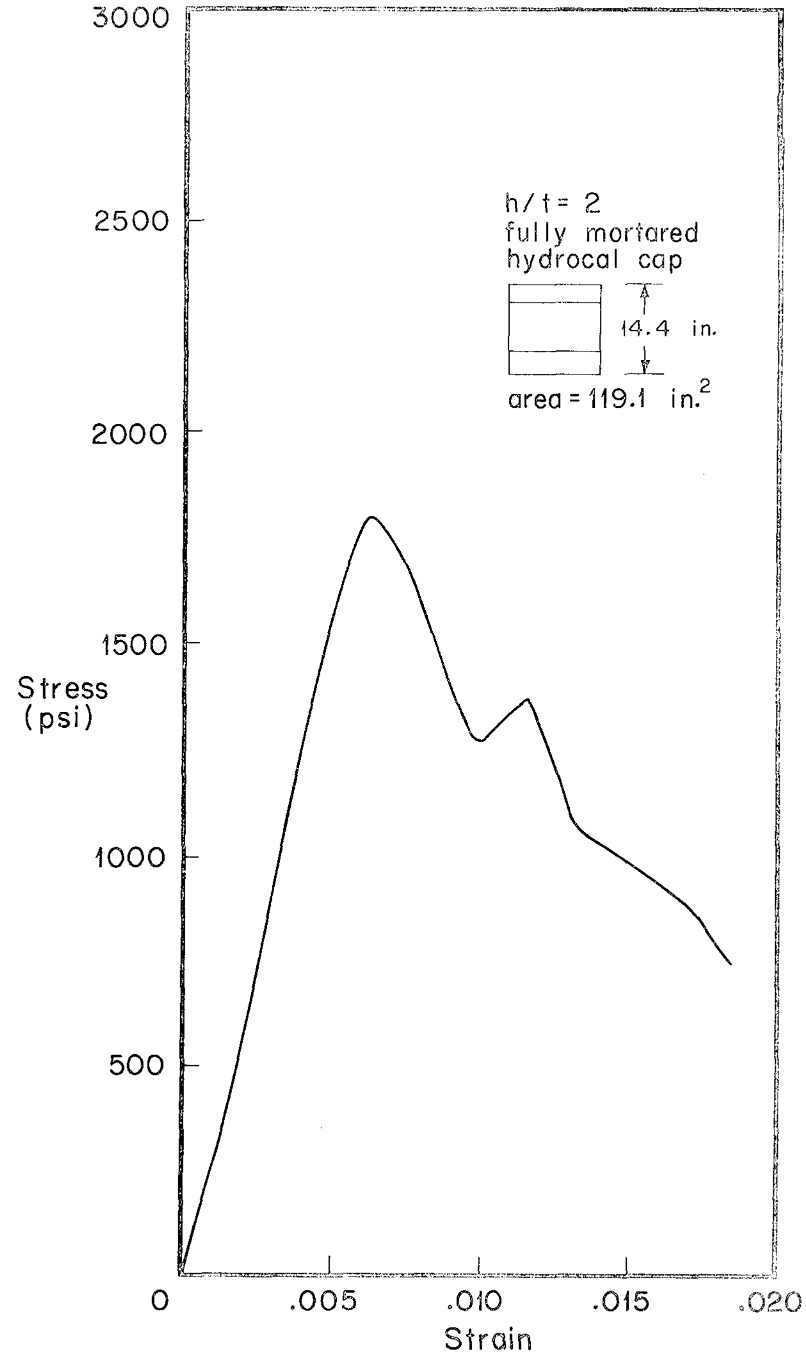


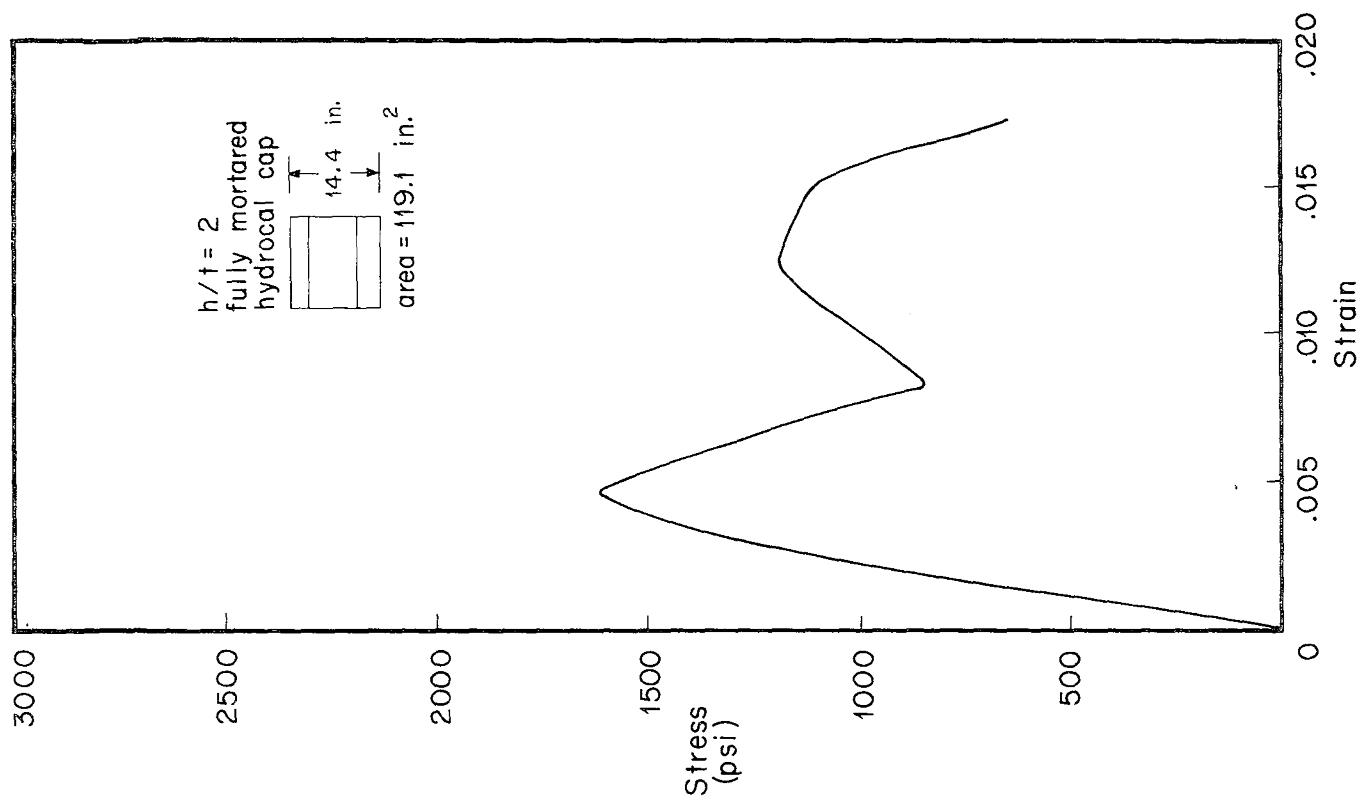
h/t = 2
fully mortared
uncapped
14.4 in.
area = 119.1 in.^{2}
Stress (psi)
0
500
1000
1500
2000
2500
3000
Strain
.005
.010
.015
.020



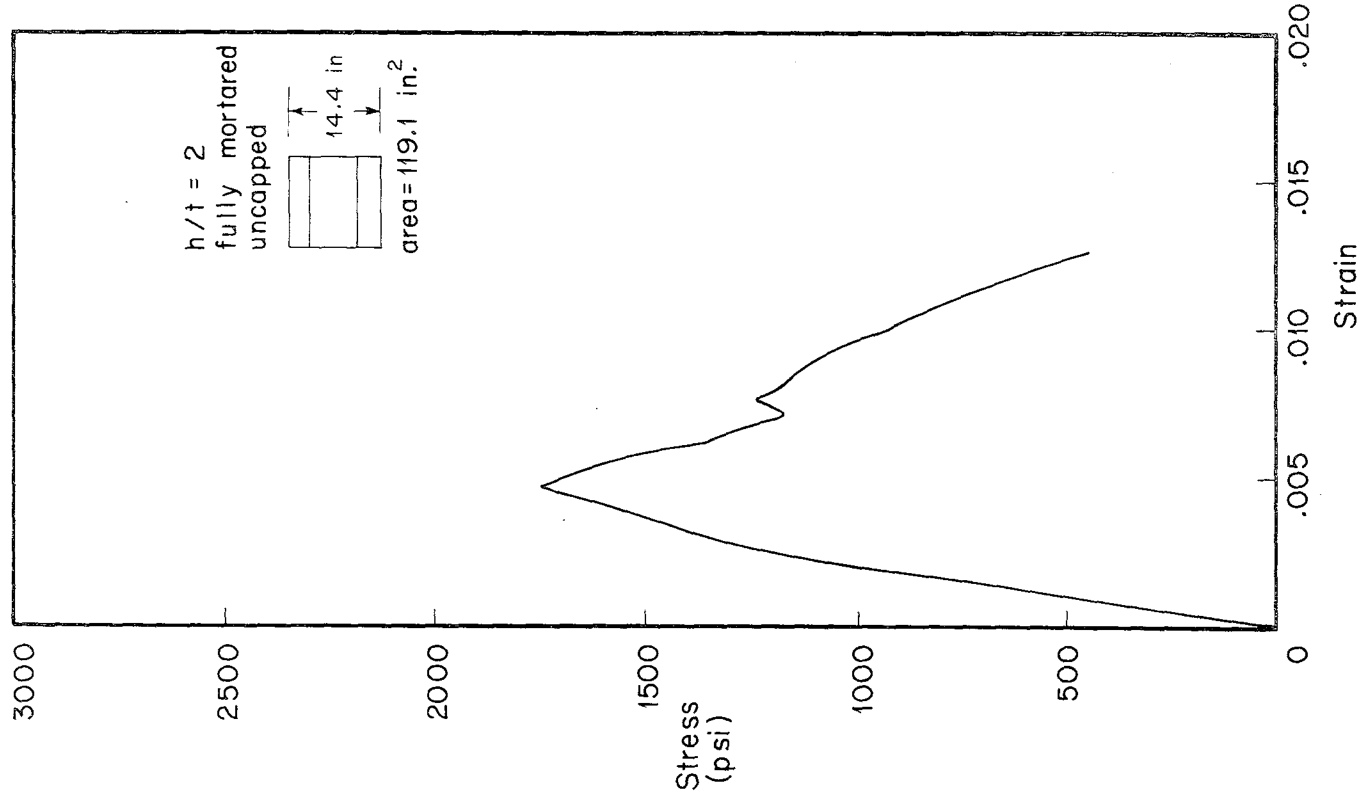
h/t = 2
fully mortared
uncapped
14.4 in.
area = 119.1 in.^{2}
Stress (psi)
0
500
1000
1500
2000
2500
3000
Strain
.005
.010
.015
.020


3000
2500
2000
1500
1000
500
0
Stress
(psi)
.005
.010
.015
.020
Strain
h/t = 2
fully mortared
hydrocal cap
14.4 in.
area = 119.1 in.2


3000
2500
2000
1500
1000
500
0
Stress
(psi)
.005
.010
.015
.020
Strain
h/t = 2
fully mortared
hydrocal cap
14.4 in.
area = 119.1 in.2



h/t = 2
fully mortared
hydrocal cap
14.4 in.
area = 119.1 in.^{2}
Stress (psi)
Strain
3000
2500
2000
1500
1000
500
0
.005
.010
.015
.020



h/t = 2
fully mortared
uncapped
14.4 in
area = 119.1 in.^{2}
Stress (psi)
Strain
3000
2500
2000
1500
1000
500
0
.005
.010
.015
.020