TEST REPORT

RESULTS of VARIATION of "b" OR EFFECTIVE WIDTH IN FLEXURE IN CONCRETE BLOCKS PANELS

CONTENTS

I. SYNOPSIS

During discussions of the wall width that can be assumed to work effectively for flexural computations in concrete block masonry, it was recognized that there was need for authentic test data to substantiate a spacing to thickness ratio assumption.

Twenty feet (20') high panels were constructed of conventional eight-inch (8") and six-inch (6") block and tested with uniform air pressure. All were similar with only the spacing of the vertical steel varying.

It was noted that eight-foot (8') steel spacing in panels in running bond was as effective as two-foot (2') spacing, that masonry deflects more without damage, and has greater earthquake damping characteristics than normally given credit.

The testing was done by Smith-Emery Test Laboratory under the direction of Mr. A. Mackintosh, Consulting Engineer, representing Concrete Masonry Association, and Mr. Walter L. Dickey, Structural Engineer, representing Masonry Research.

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by

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II. OBJECT

To determine the effect of the spacing of vertical reinforcing on the flexural resistance of reinforced concrete masonry walls and to establish the proper, effective width available for design calculations.

III. DISCUSSION

A. Need for Test Program

There was discussion regarding the Effective "b" that can be used for flexural computations of walls. There was little applicable data available in concrete design other than "rule of thumb" to dictate such widths and even less in masonry. However, in prestressed concrete publications indications were given that the effectiveness of reinforcing is related to the span, the flange thickness and the width, or spacing. Precise calculations and field verification of prestress deflection confirmed the validity of such interrelation.*

Preliminary calculations indicated that steel in walls of maximum h/t would be effective much farther apart than the present assumption of 4 x t and considerably beyond four feet (4') for eight-inch (8") block. In view of this, it was obvious that discussions as to proper placement of bars were based on "feel" rather than any factual data.

The Concrete Masonry Association of California and Masonry Research jointly arranged for a test program to be performed by Smith-Emery Test Laboratory under the direction of Mr. Albyn Mackintosh, Consulting Engineer for Concrete Masonry Association, and Mr. Walter L. Dickey, Consulting Structural Engineer for Masonry Research.

B. Theory

Investigation was made early for prestressed concrete elements because, in prestressed concrete, one must know not only the conservative answer but rather the correct answer accurately as deflections are a serious consideration in prestressed concrete design. If one over corrects rather than uses the right answer in deflection, there will be serious bowing and difficulty. The curves and data extrapolated, as well as one might in rather naive manner, indicated that eight to ten-foot (8' to 10') spacing would be effective for twenty-foot (20') high panels. Since deflections are the final results of load, load distribution, stress distribution and all the other variables, careful measurements of the deflections were made.

Uniform load distribution was applied to eliminate introducing another variable i.e. conversion from equivalent point loading to the uniform loading assumed for design and, also, this is the loading most likely to occur against the wall.

The maximum of eight-foot (8') spacing was selected so this data might be pertinent to the spacings utilized in "Partially Reinforced Masonry."**

* American Concrete Institute Journal, March 1961
**Uniform Building Code, Sec. 2417 (k); "Building Code Requirements for Reinforced Masonry; ASA 41. 2, paragraph 1. 3. 13

C. Construction - The Test Frame

The test frame was built of wood and braced with steel cable. The reactions were carried to the wall panel at the top and the bottom by bolts to trussed beams behind the panels providing space for an air bag between the test frame and the wall panel. In order that the frame would impose no lateral load and still rotate freely as the wall panel curved and deflected, the vertical weight of the test frame was balanced on a rocker at the base.

A fork lift picked up the test frame, held it in position until anchorage was placed at the top and bottom and the bags positioned, and then backed away leaving panel and test frame free to move under the uniform load between the two end reactions; i. e., simulating pin-ended uniform load moment conditions. The concrete footing at the base merely provided stability to the system to prevent its falling during the operation.

The large top end deflections would impose some additional bending due to the tilt of the wall and the resulting eccentricity of the weight of the wall. However, such deflections would occur to an appreciable amount only for those deflections well beyond the working range of masonry.

D. Construction - Panels

The block panels were eight feet eight inches (8'8") long and twenty feet (20') high. Six were eight-inch(8") block in running bond; one was eight-inch(8") in stack bond with required joint anchorgae (Dur-O-Wal) and one was six-inch (6") block in running bond.

A continuous concrete pad was used for the foundation to simulate field conditions and to provide some base stability. The foundation was doweled to each panel identically with 5/8" dowels at the center and at each end cell, grouted two feet(2') high.

All panels had a bond beam with two #4's grouted at top and they all contained a bond beam seven feet two inches (7'2") above the foundation to simulate the most likely field conditions.

All head and bed joints were full for the thickness of the face shells.

All mortar joints were cut flush, not tooled.

All were built simultaneously to line to avoid any difference in workmanship and weather.

All mortar was 1: 1/2: 4-1/2 of materials in compliance with ASTM and from the same piles.

All grout was 1: 3: 2 carefully mixed at the site from a constant stock pile.

All block, mortar, grout, steel, sand and gravel were checked for physical properties by the test laboratory.

All steel was taken from one batch to keep the "Es" the same.

All grouting of vertical steel was by the high lift grout procedure in six-foot (6') lifts with vibration at time of pouring.

All panels had bolts for brace at the door head bond beam and were braced each way at this point. In addition, all were tied at top to concrete fence anchors and braced with struts on the same side.

All panels contained the same amount of vertical steel but the spacings of bars within the total "b" of 104" varied as follows:

Panel		Spacing	Ratio of Spacing to Thickness
#1	At center, i. e.	8' 8"	13
#2	At edges, i. e.	8' 0"	12
#3	At third points, i. e.	5' 4"	8
#4	At edge and 2' 0"	2' 0"	3
#5	At center & edge, i. e.	4' 0"	6
#6	At edges (Stack bond), i. e.	8' 0"	12
#7	At edges, i. e.	5' 4"	8
#8	At third points (6" Wall), i. e.	5' 4"	11

E. Test Results

The readings taken from the field tests were plotted as indicated in the Appendix. Ordinates used were "equivalent pounds per square foot of panel" versus "deflections of the mid-height of the panel."

Two additional lines were plotted which indicate the deflection under these conditions for:

1. The uncracked section.
2. The cracked section.

It can be seen that the initial deflection of the panels was as if they were uncracked sections and then, after cracking, that the slope exactly parallels the cracked section line.

The calculations were based on the total width functioning. However, when the deflections for the stack bond panel were plotted, it was obvious that this assumption for "b" was incorrect. The slope was as if 1/2 the "b" were functioning and hence this line was plotted accordingly.

Span deflections were measured at the mid-height of the vertical center line and at the two vertical edges. These were all quite close as noted by the shaded area between the curves on the charts.

Visual observation during failure showed that the mortar joints function somewhat as an accordion; that is, there was a zig-zag type of crack alternately on one side and then on the other side of the bed joints. These returned and closed as if they were well within the elastic range when loads were removed from the panels.

Paneldemolition indicated that the panels dissipated energy to a great degree. For example, it was impossible to push the panels with rhythmic force in tune with the natural frequency to build up the amplitude adequate to cause failure. The energy dissipated too rapidly in the multiplicity of joints and in inelastic deformations.

A motion picture of the testing was made. It is being checked for amplitude and frequency, and the vibration damping or decadence curves are being plotted for part of another program.

Upon breaking open the panels, it was seen how well the grout cores had been filled by the High Lift grouting method and that the foundation dowels functioned well in their grouted cores without being in the same core as the vertical reinforcement and its grout.

The horizontal wire for measurement at the mid-height confirmed that the full width was acting because of the straight line that was maintained across the middle "b."

F. Calculations

Calculations for strength and for deflection were corrected as necessary due to the field variations. For example, in one test the air bag pressure did not cover the entire wall area and the total load was consequently reduced by that ratio.

Each of the spans was also corrected to the actual measured span instead of to the initially assumed span.

Similarly, each of the distances to the center of steel was corrected to measured values.

The stresses in the steel, in the concrete block material and in the grout were based on the actual strengths that were determined by testing the materials during and prior to panel construction. The modulus of elasticity used was as measured in the test laboratory.

In order that small variations in deflection and capacity might be compared, the calculations were carried out more precisely than would be justified for design.

Instead of assuming a k and j of a rectangular section, the np, k, etc. for Tee sections (where appropriate) were used since the actual web thickness in some cases had the effect of making the section like a Tee shape rather than a rectangular section.

The deflection calculations were made for deflection at the center of span and also for the deflection of the top (assuming fixity at the base to check that the panel deflection was free).

IV. GENERAL TEST PROCEDURE

The general steps of the testing procedures were;first to calculate the anticipated range of deflections, plot curves, set up the lines for readings accordingly; then test as follows:

1. Remove strut braces and ties from panel.
2. Use a fork lift to hold the test frame in proper position (resting on the rocker about one inch (1") from the panel) with the air bag hanging in the space.
3. Provide connection bolts at top and bottom to the trussed reaction beams and tighten slightly against spacers.
4. Back fork lift away.
5. Attach three taut horizontal piano wires at top and bottom reaction points, and at mid-height.
6. Attach air pressure surce to bag inlet and manometer to outlet.
7. Take initial readings:
   a. At end and center of all wires.
   b. Distance from top of wall to a base line.
   c. Manometer readings.
8. Add air pressure to bag in increments of approximately 5 psi,repeating readings.
9. Note first cracking and continue past yielding, i. e., until additional pressure cannot be held and deflection continues at lower pressure.
10. Vibrate panels to check action, e.g., natural frequency, decadence, kinetic damping, etc. (Record on timed motion picture film frame sequence.)
11. Break panels down checking exact location of reinforcing steel.
12. Tabulate and plot Load versus Deflection,comparing with calculated values.

V. CONCLUSIONS

A. In eight-foot (8') wide, twenty-foot (20') concrete block walls, eight-inch (8") and six-inch (6") thick in running bond, the vertical reinforcing bars function with the total width for stress and deflection as effectively at eight-foot spacing (8') as it does at two-foot (2') spacing. This represents an effective "b" for a spacing to thickness ratio of 104/8 or 13.

B. Vertical reinforcing eight feet (8') o. c. in eight-inch (8") thick stack bond panels twenty feet (20') high with required horizontal reinforcement anchorage functions as if working with about half the effective width, i.e., at some fifty-two inches (52").

C. Assuming the "Effective Width for Flexural Computation" as 6 times the thickness of block in running bond is considerably on the conservative side for flexural effectiveness.

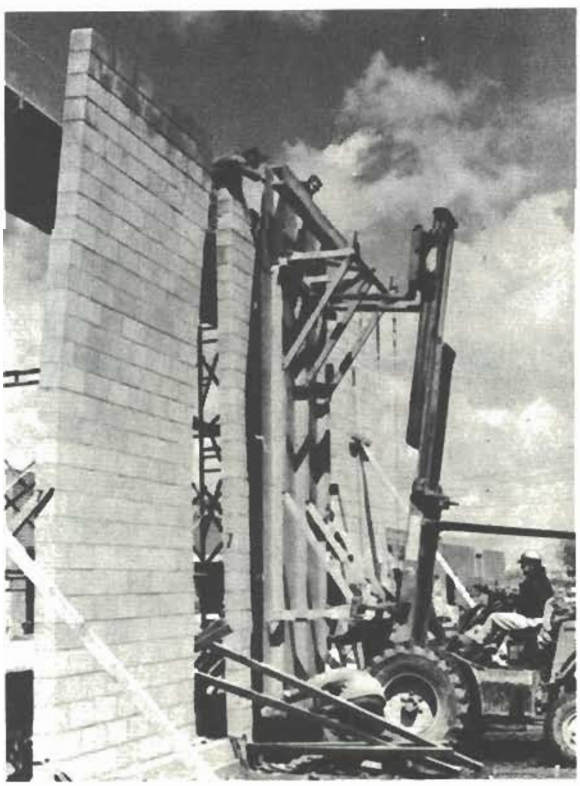
D. Reducing the "Effective Width for Flexural Computation" to 3 times the thickness of walls for block in stack bond would be indicated by the test results.

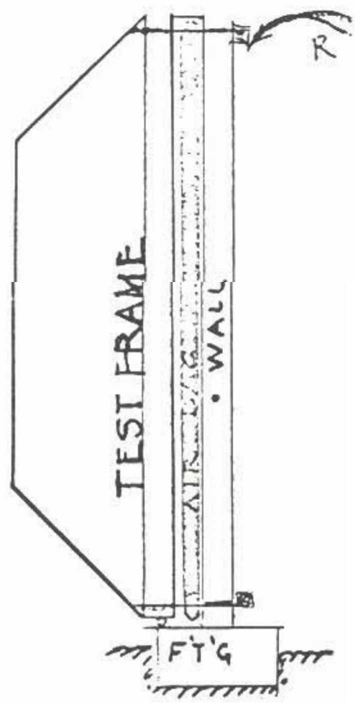
E. The high lift grout method in twenty-foot (20') high panels in eight-inch (8") and six-inch (6") thick block walls can be a satisfactory method of placing 1: 3: 2 grout mix. Grout in bond beams can flow laterally approximately eight feet (8') when poured and vibrated.

F. Reinforced block panels apparently dissipate earthquake energy much better than they are given credit for and this should be studied further.

G. Reinforced Concrete Block Masonry can withstand deflections without serious damage or reduction of effectiveness to such a degree that possible deflections should be checked for architectural considerations.

TEST EQUIPMENT





SCHEMATIC of TEST FRAME

VI. APPENDIX

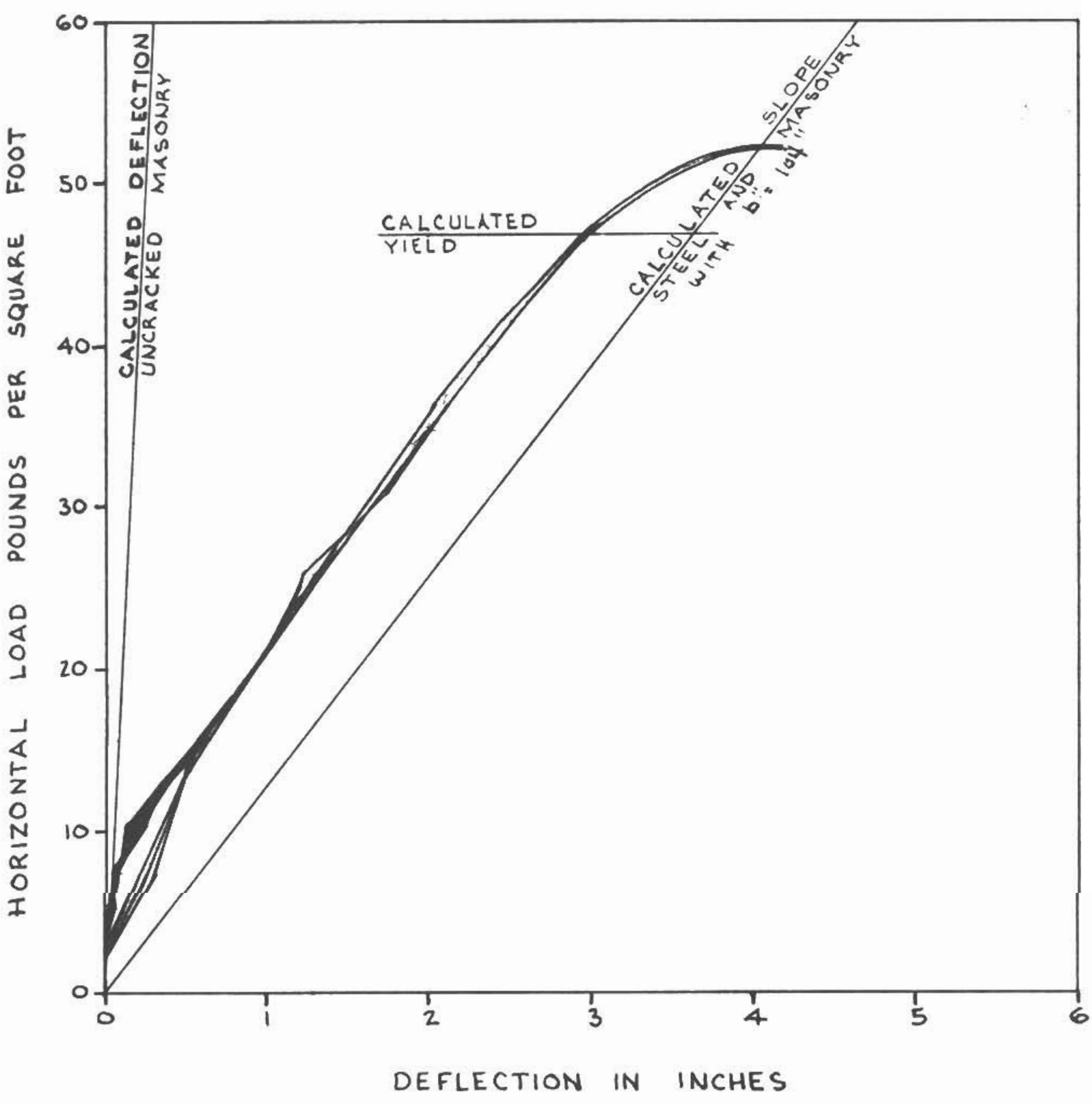
A. Load - Deflection Curves

B. Sample Calculations

C. Smith-Emery Reports

P-378889	-	Mortar
P-378890	-	Mortar
P-378891	-	Grout
P-378959	-	Mortar
P-378960	-	Mortar
P-378961	-	Mortar
P-378981	-	Mortar
P-379035	-	Grout
P-379057	-	Block, Sand, Gravel, Steel
P-379776	-	Load Tests

TEST WALL # 1



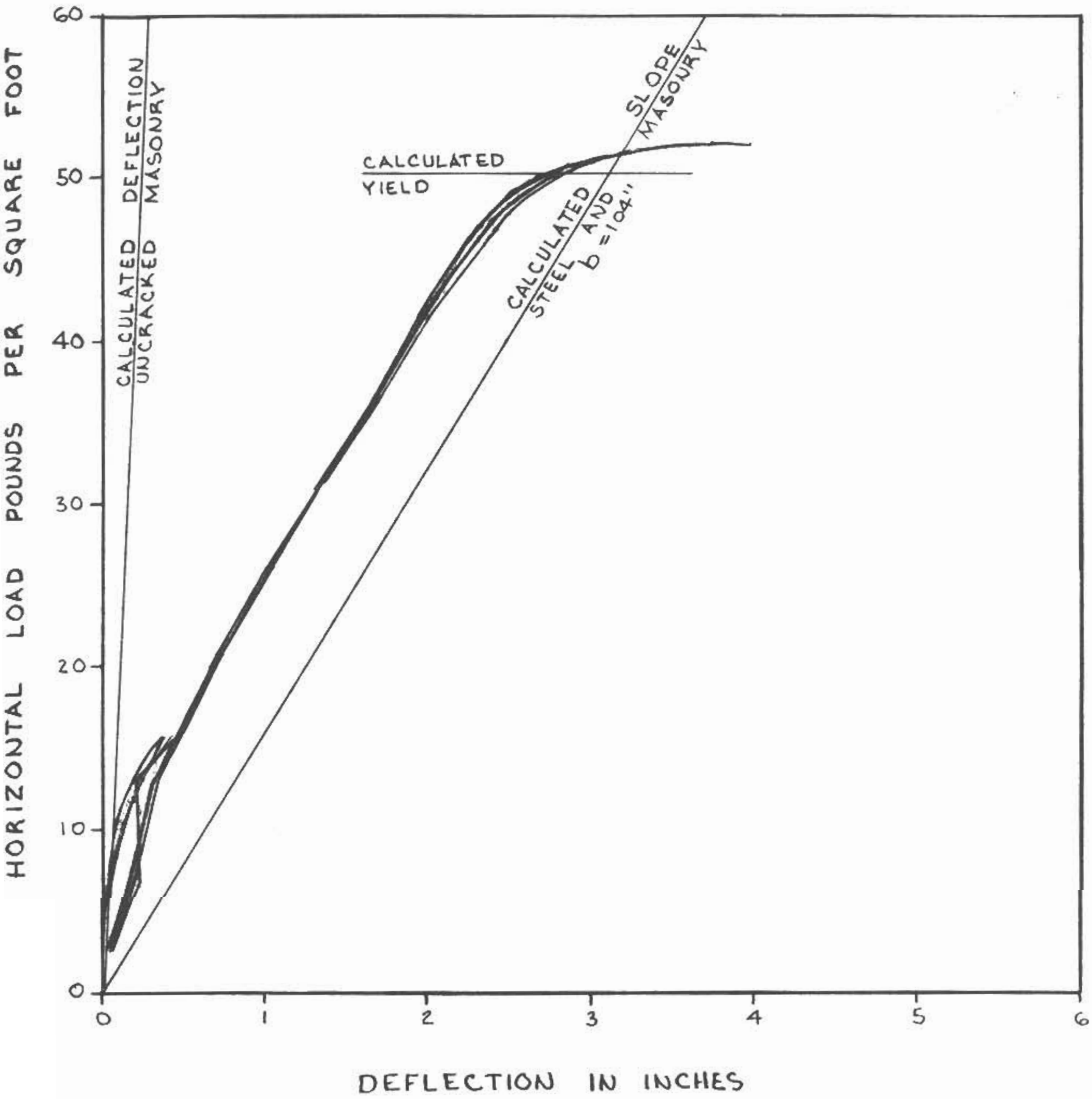
NOTES:

2 - #7 BARS @ ℄ OF WALL - EQUIVALENT 8'-0" O.C.

EFFECTIVE DEPTH = 3.6" (MEASURED)

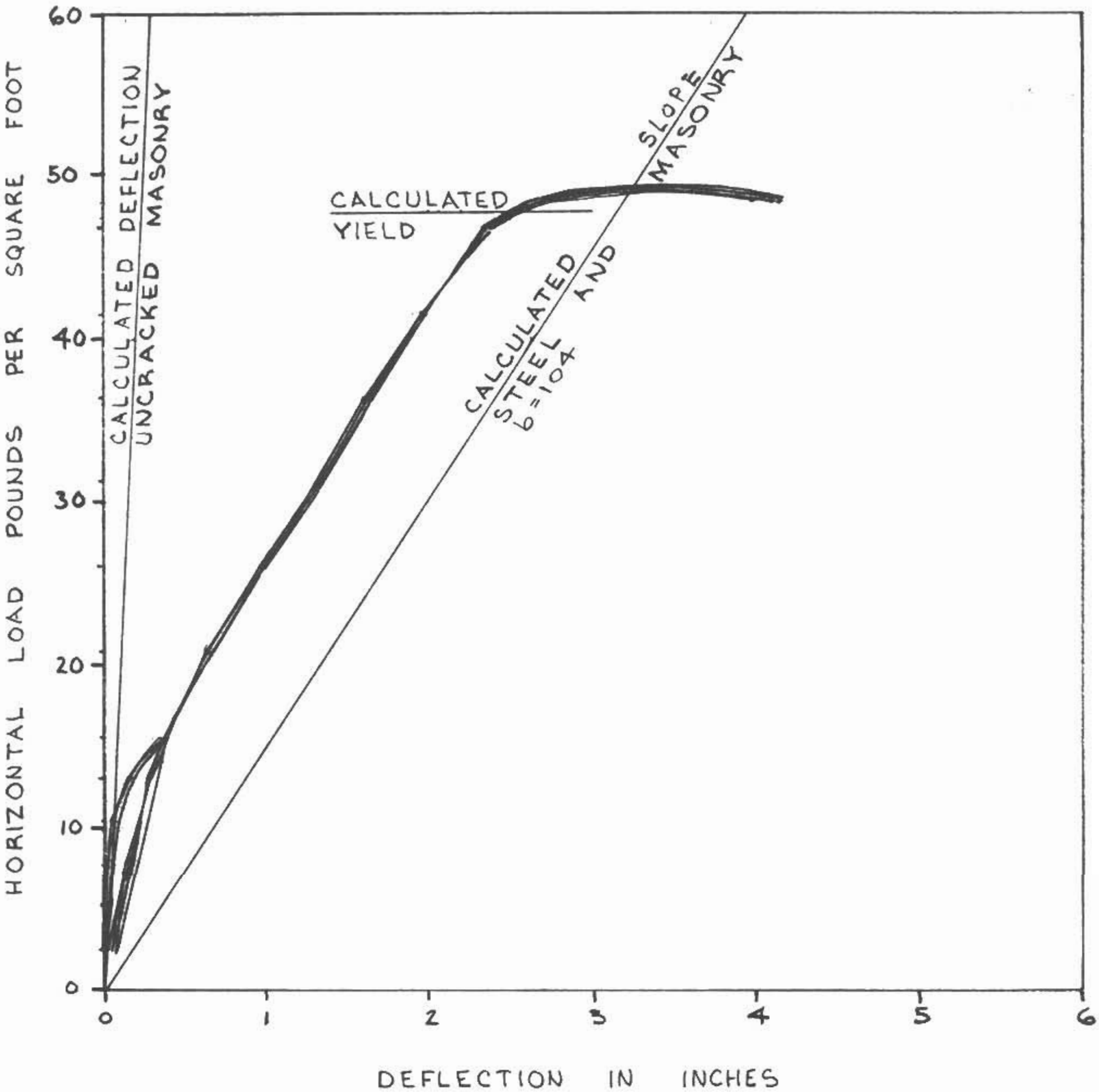
TEST REBOUNDED @ 15.6 #/□' TO 2.6 #/□'

TEST WALL #2



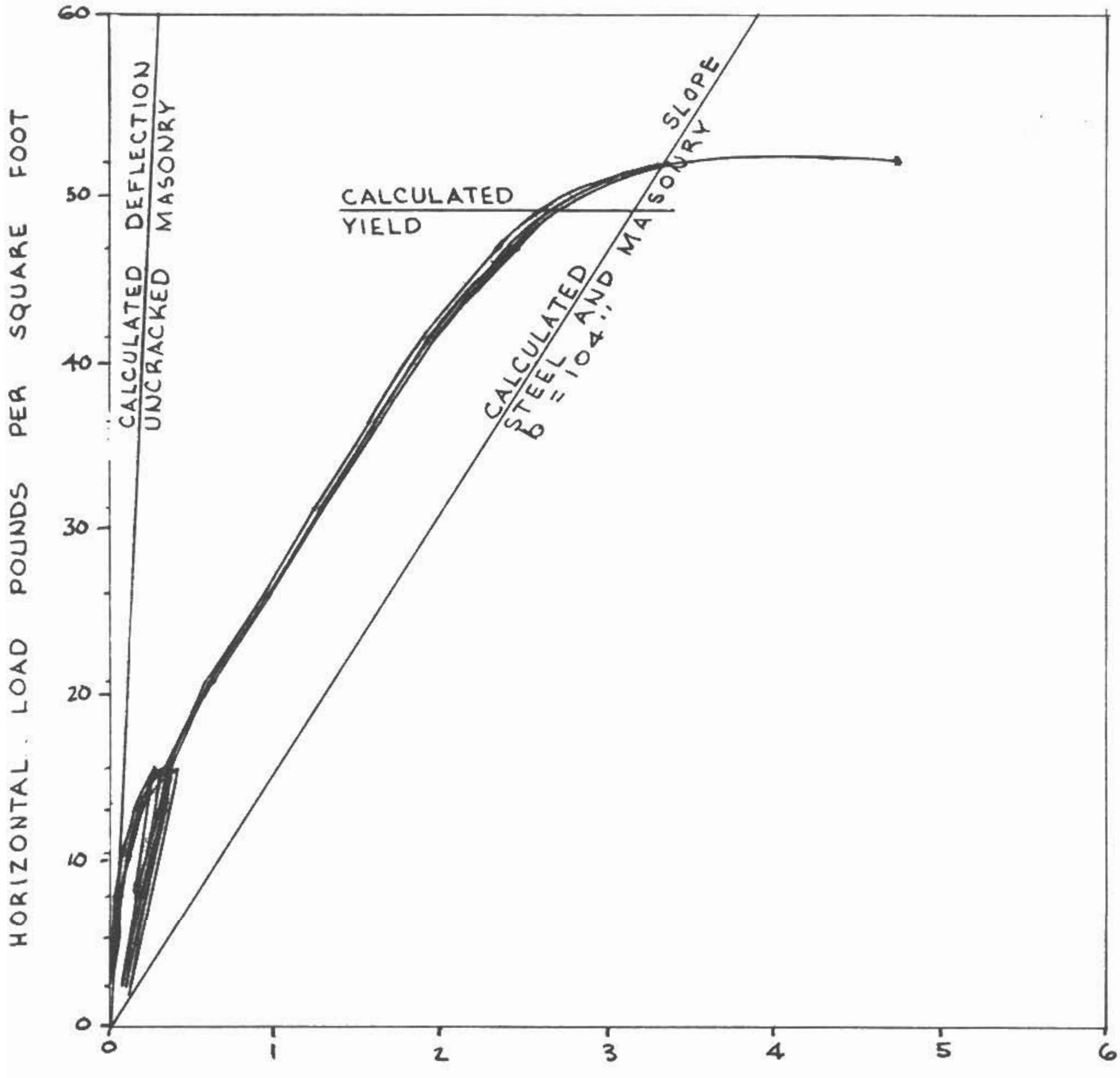
NOTES:
1 – #7 BAR EACH END OF WALL – BARS 8'-0" O.C.
EFFECTIVE DEPTH = 4" (MEASURED)
TEST REBOUNDED @ 15.6 #/□' TO 2.6#/□'

TEST WALL # 3



NOTES:
#7 VERTICAL BARS @ 5'-4" O.C.
EFFECTIVE DEPTH = 3.81" (MEASURED)
TEST REBOUNDED @ 15.6#/□' TO 2.6#/□'

TEST WALL #4



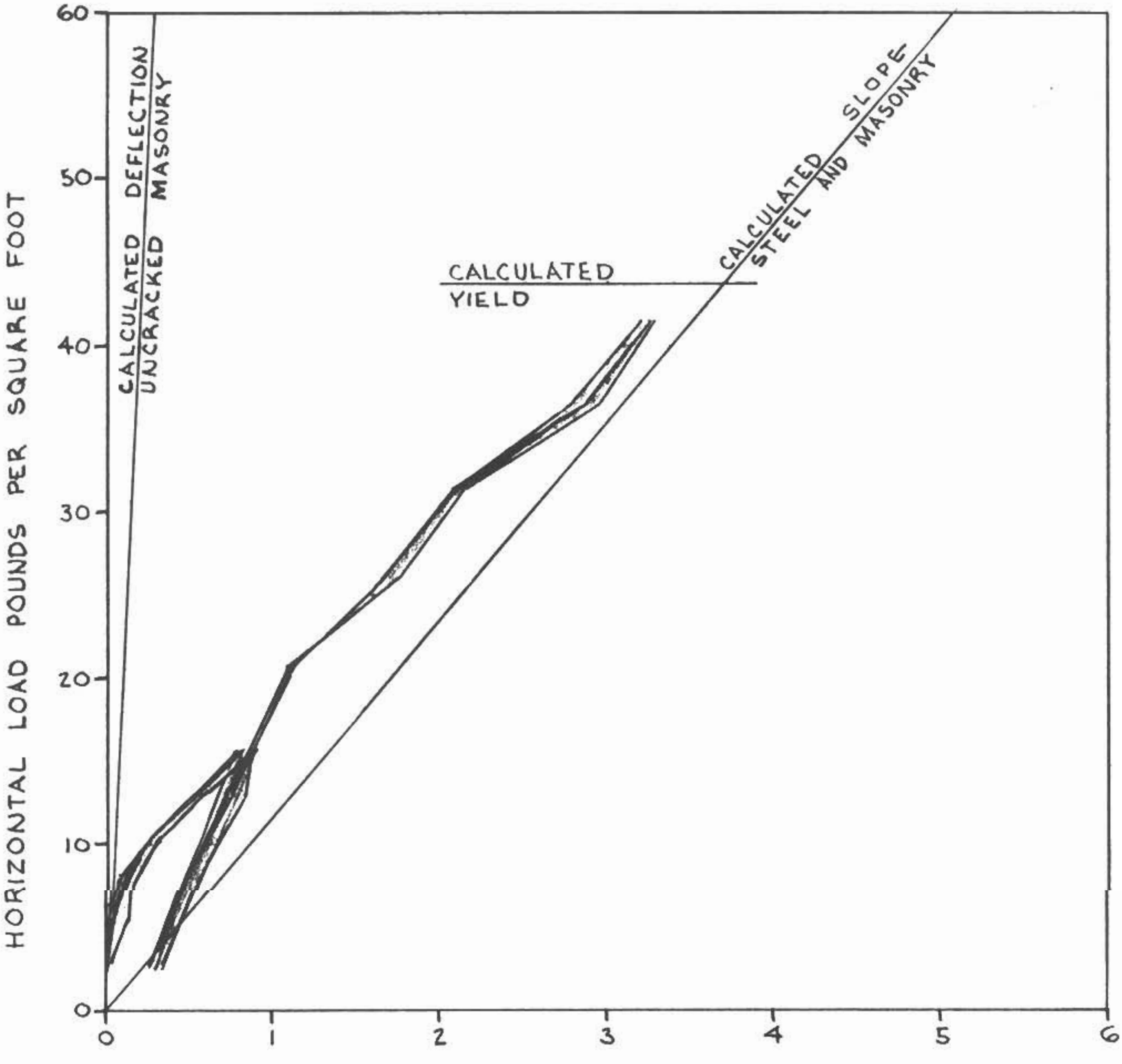
NOTES:

VERTICAL BARS @ 24" O.C.

EFFECTIVE DEPTH = 3.9" (MEASURED)

TEST REBOUNDED @ 15.6 #/□' TO 2.6 #/□'

TEST WALL # 5

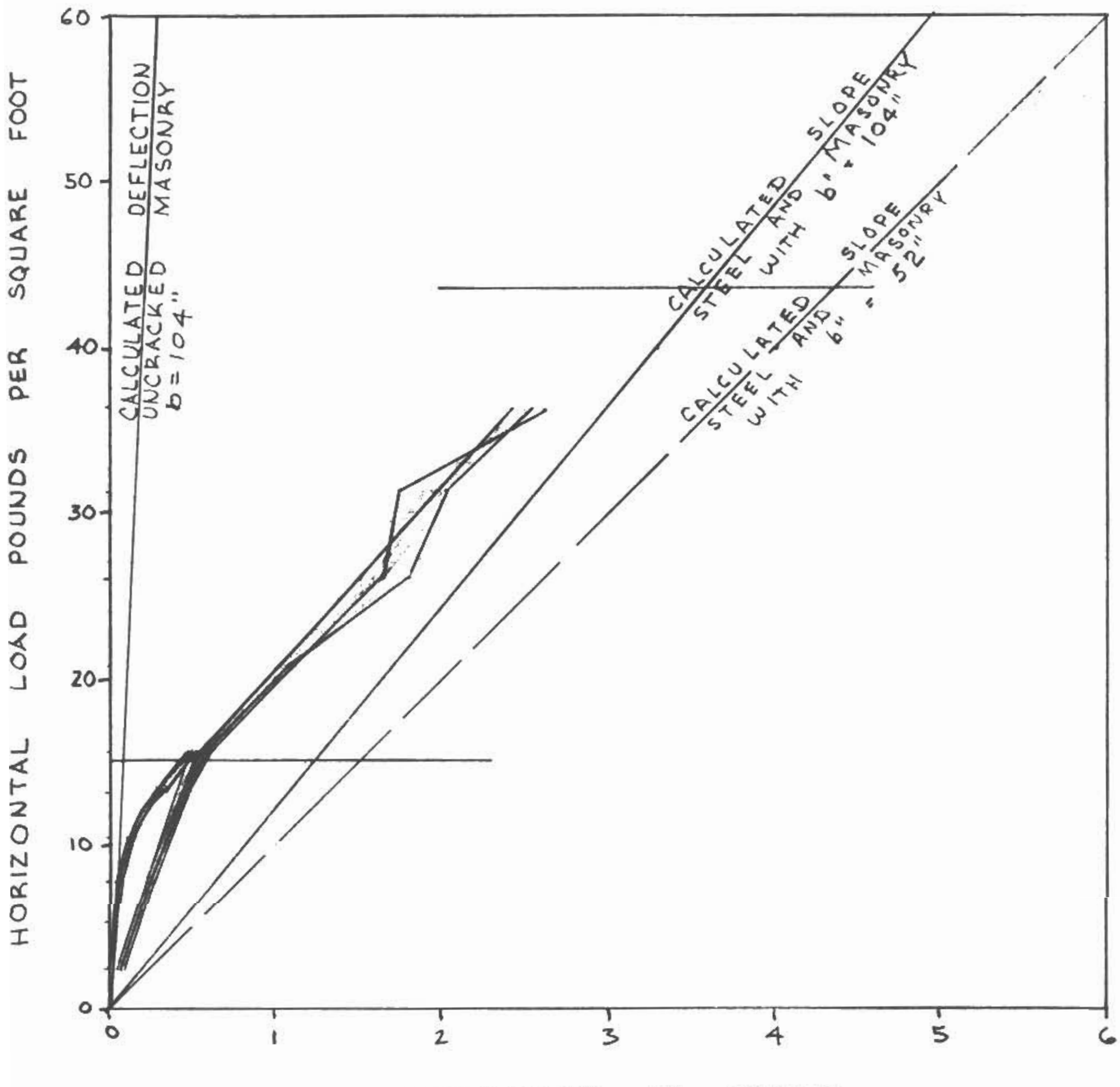


NOTES:

VERTICAL BARS @ 4'-0" O.C.

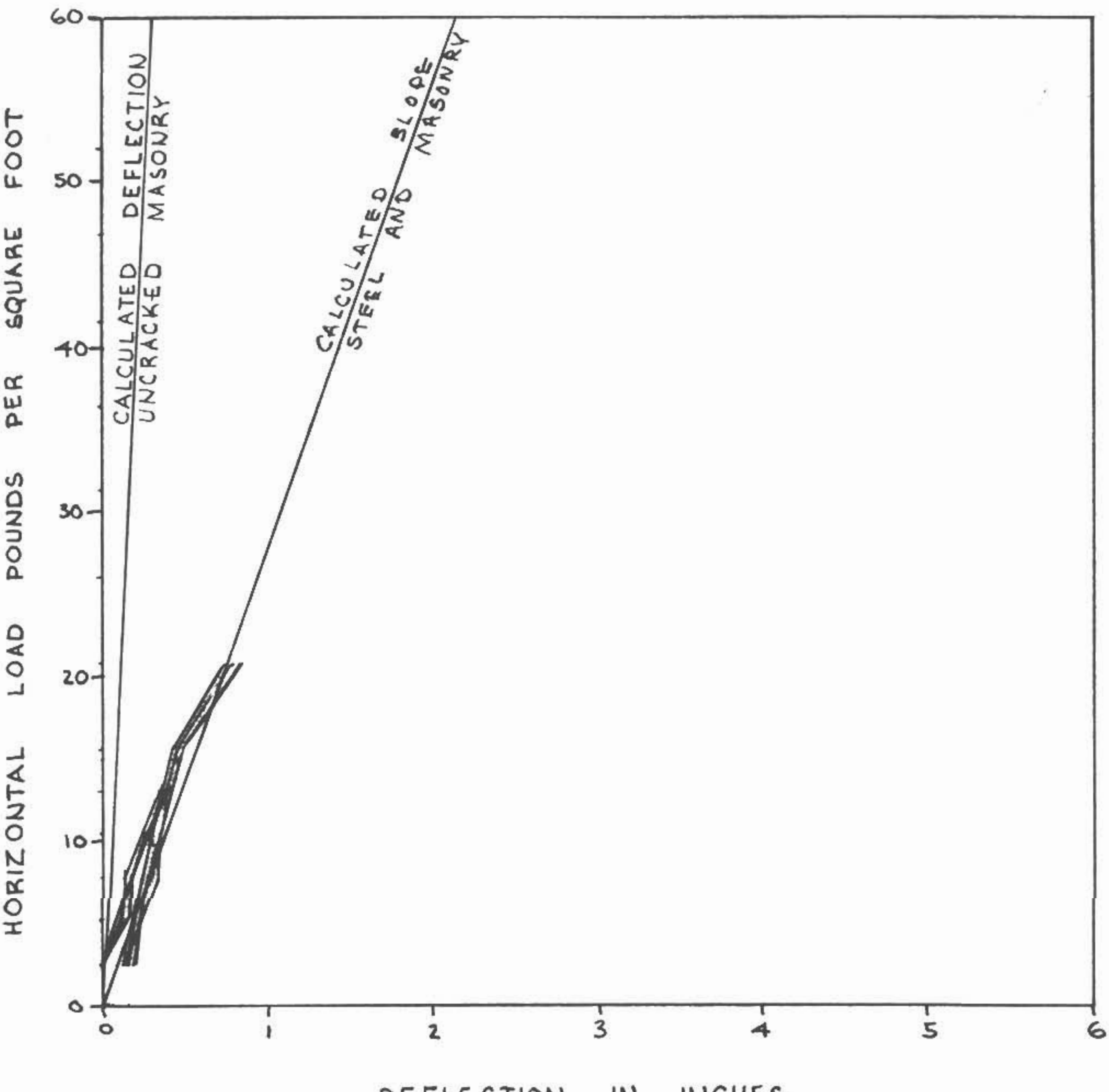
TEST STOPPED DUE TO DEFLECTION WIRES TOUCHING WALL
TEST ERRATIC DUE TO FORK LIFT TOUCHING WALL
TEST REBOUNDED @ 15.6 #/□' TO 2.6 #/□'

TEST WALL #6
(STACK BOND)



NOTES:
1 - #7 BAR EACH END OF WALL - 8'-0" O.C.
WALL LAID IN STACKED BOND USING JOINT REINFORCING
EFFECTIVE DEPTH = 3.5" (MEASURED)

TEST WALL # 7-A
(4'×10' BAGS - LEAKED)



NOTES:

SPECIAL GROUTING PROCEDURES

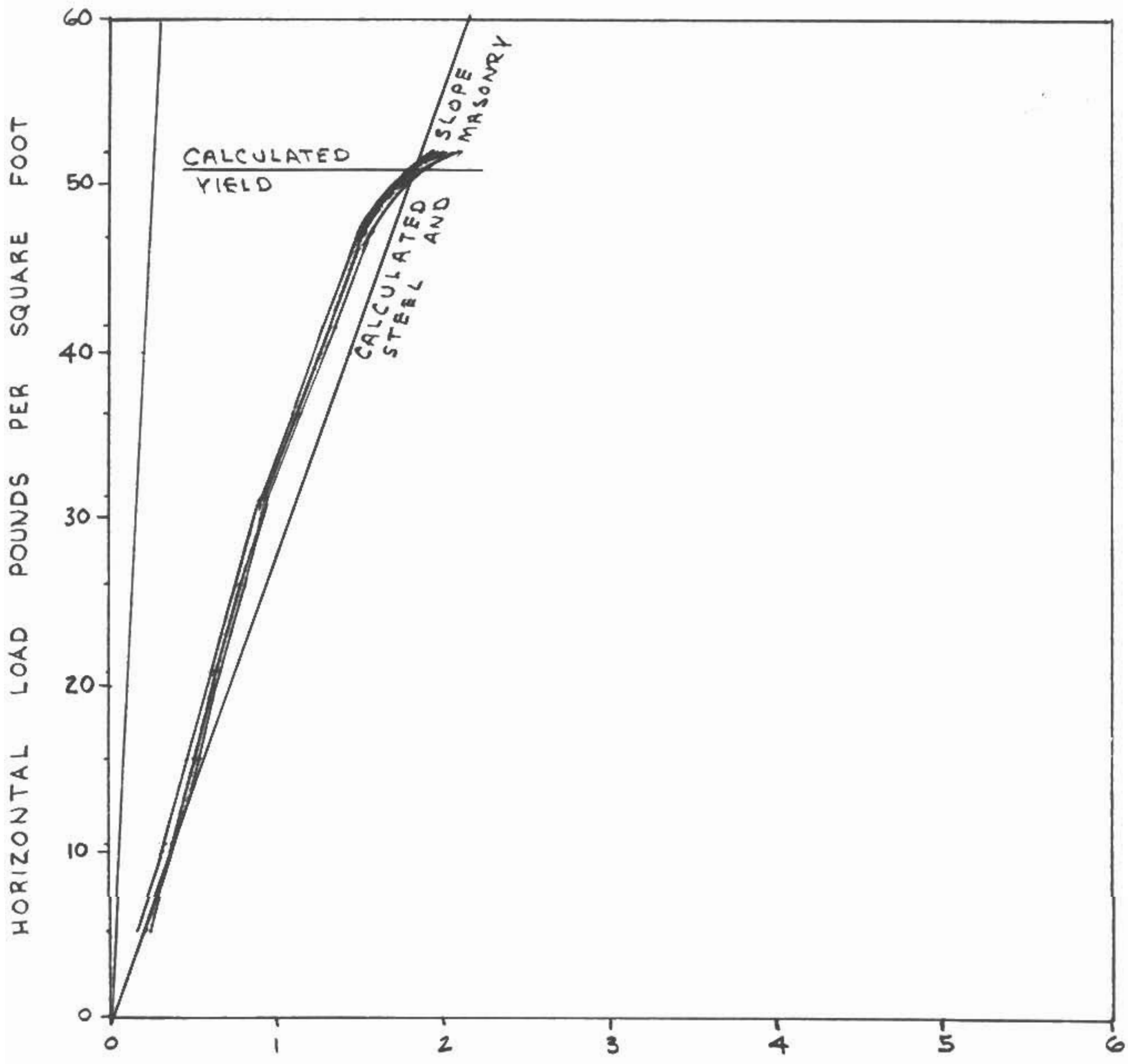
VERTICAL BARS # 7 @ 8'-0" O.C.

DEPTH = 5.1" & 3.7"

WALL TESTED TWICE WITH AIRBAGS FAILING

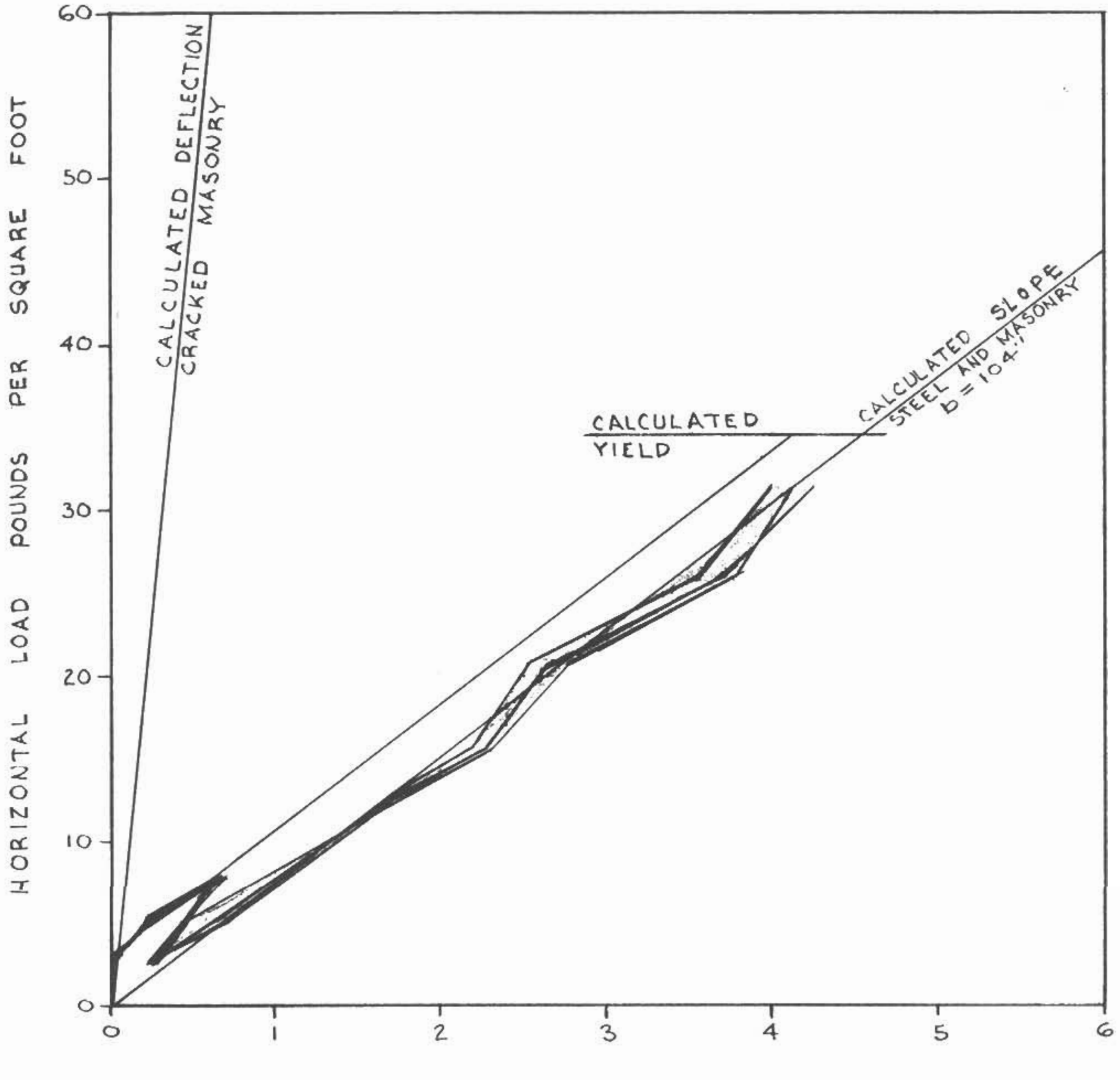
TEST REBOUNDED @ 13.0#/□' TO 2.6#/□'

TEST WALL #7-B (8'-4" x 20" BAG)



(SEE 7-A)

TEST WALL # 8
(6" BLOCK)



NOTES:
TEST REBOUNDED @ 7.8 #/□' TO 2.6 #/□'
DEFLECTION STOPPED TEST
EFFECTIVE DEPTH = 2.8" (MEASURED)

TEST CALCULATION

CONCRETE BLOCK

COMPRESSION – GROSS AREA 1789 PSI NET AREA 3769 PSI

TENSION – 185 PSI SHRINKAGE WET TO DRY .0422 %

MODULUS OF ELASTICITY – 1,734,000

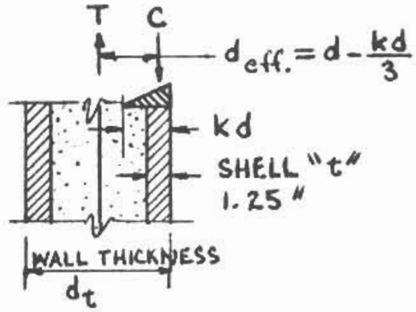
REINFORCING STEEL

MODULUS OF ELASTICITY – 28,500,000

YIELD POINT – 52,883 PSI

MORTAR – 2390 PSI

GROUT – 2354 PSI



$$n = \frac{28{,}500{,}000}{1{,}734{,}000} = 16.4$$

$$\Delta_s = \frac{5}{384}\ \frac{w l^4}{EI}$$

MOMENT OF INERTIA

UNCRACKED SECTION $= \frac{b(d_t)^3}{12}$ (APPROX.)

CRACKED SECTION MASONRY ONLY DEFLECTING $= b''t''(d''_{eff.})^2$

CRACKED SECTION STEEL ONLY DEFLECTING $= A_s''\,(d''_{eff})^2$

ASSUMING A CRACKED SECTION AND DEFLECTION IN STEEL ONLY

$$\Delta = \frac{5 \times 1728 \times 8.67\,(l')^4\,w}{384 \times 28{,}500{,}000 \times 1.2\,(d''_{eff.})^2} = .000{,}005{,}7\,\frac{(l')^4}{(d''_{eff.})^2}\,w$$

ASSUMING A CRACKED SECTION AND DEFLECTION IN MASONRY ONLY

$$\Delta = \frac{5 \times 1728 \times 8.67\,(l')^4 w}{384 \times 1{,}734{,}000 \times 104 \times 1.25 \times (d_{eff.})^2} = .000{,}000{,}87\,\frac{(l')^4}{(d'')^2}\,w$$

ASSUMING UNCRACKED SECTION

$$\Delta = \frac{5 \times 1728 \times 8.67\,(l')^4 w \times 12}{384 \times 1{,}734{,}000 \times 104\,(d_t)^3} = .000{,}012{,}9\,\frac{(l')^4}{(d_t)^3}\,w$$

8" WALL $\Delta = .000{,}000{,}03\,(l')^4 w$

6" WALL $\Delta = .000{,}000{,}07\,(l')^4 w$

TEST PANEL # 1 MEASURED $d = 3.6''$ $A_s = 1.2^{\square''}$ $\ell' = 18.46'$

USING $b = 104''$ $np = \frac{1.2 \times 16.4}{3.6 \times 104} = .0526$

THEN $k = .276$ $j = .908$ $2/_{kj} = 7.99$ $kd = 3.6 \times .276 = 0.99$

$3.6 - \left(\frac{0.99}{3}\right) = 3.27$ $\frac{(\ell')^4}{(d_{eff.})^2} = \frac{(18.46)^4}{(3.27)^2} = 10,900$

@ $w = 40$ PSF $\Delta_s = .000,005,7 \times 10,900 \times 40 = 2.48''$

$\Delta_m = .000,000,87 \times 10,900 \times 40 = \underline{0.38''}$

TOTAL $= 2.86''$

GRAPH CHECKS THIS LINE PARALLEL WITH MAIN SECTION OF CURVE. USING b" LESS THAN 104" WOULD CHANGE LINE SO THAT IT DID NOT PARALLEL TEST RESULTS.

UNCRACKED SECTION $\Delta_m = .000,000,03 \times (18.46)^4 40 = .15$

@ YIELD $f_s = 52,883$ PSI

$m = 1.2^{\square''} \times 52,883 \times .908 \times 3.6 = 207,400'$

$w = \frac{207,400 \times 8}{12 \times 8.67 \times (18.46)^2} = 46.8$ PSF

RE REBOUND

DEFLECTION @ 15.6 P.SF = .61" REBOUND TO 2.6 PSF $\Delta = .045$

$\frac{.61 - .045}{.61} = 92\,\%$

MASONRY STRESS @ STEEL YIELD

$f_m = \frac{207,400 \times 7.99}{104 \times (3.6)^2} = 1200$ PSI

NO COMPRESSION FAILURES NOTED IN THE TEST

REINFORCING STEEL AREA AND EFFECTIVE DEPTH GOVERNED DEFLECTION CURVE

TEST PANEL #6 MEASURED $d = 3.5$ $A_s = 1.2^{\square''}$ $\ell' = 18.85'$

USING $b = 104''$ $np = \dfrac{1.2 \times 16.4}{3.5 \times 104} = .054$

THEN $k = .279$ $j = .907$ $2/_{kj} = 7.92$ $kd = 3.5 \times .279 = 0.977$

$d_{eff.} = 3.5 - \left(\dfrac{.977}{3}\right) = 3.17''$ $\dfrac{(\ell')^4}{(d_{eff.})^2} = \dfrac{(18.85)^4}{(3.17)^2} = 12,560$

@ $w = 40$ PSF $\Delta_s = .000,005,7 \times 12,560 \times 40 = 2.86$

$\Delta_m = .000,000,87 \times 12,560 \times 40 = \underline{0.44}$

TOTAL $= 3.30$

GRAPH SHOWS THIS LINE IS NOT PARALLEL WITH MAIN SECTION OF CURVE.

USING $b = 52''$ $np = \dfrac{1.2 \times 16.4}{3.5 \times 52} = .108$

THEN $k = .369$ $j = .877$ $2/_{kj} = 6.18$ $kd = .369 \times 3.5 = 1.29$

$d_{eff.} = 3.5 - 1.29 = 3.07''$ $\dfrac{(\ell')^4}{(d_{eff.})^2} = \dfrac{(18.85)^4}{(3.07)^2} = 13,400$

@ $w = 40$ PSF $\Delta_s = .000,005,7 \times 13,400 \times 40 = 3.04$

$\Delta_m = .000,000,87 \times 13,400 \times 40 = \underline{0.93}$

TOTAL $= 3.97$

THIS LINE PARALLELS TEST RESULTS

UNCRACKED SECTION $\Delta_m = .000,000,03 \times (18.46)^4 40 = .15$

@ YIELD $f_s = 52,883$ PSI

$m = 1.2^{\square''} \times 52,883 \times .877 \times 3.5 = 195,000$

$w = \dfrac{195,000 \times 8}{12 \times 8.67 \times (18.85)^2} = 42.3$ PSF

RE REBOUND

DEFLECTION @ 15.6 PSF $= .61''$ REBOUND TO 2.6 PSF $\Delta = .045$

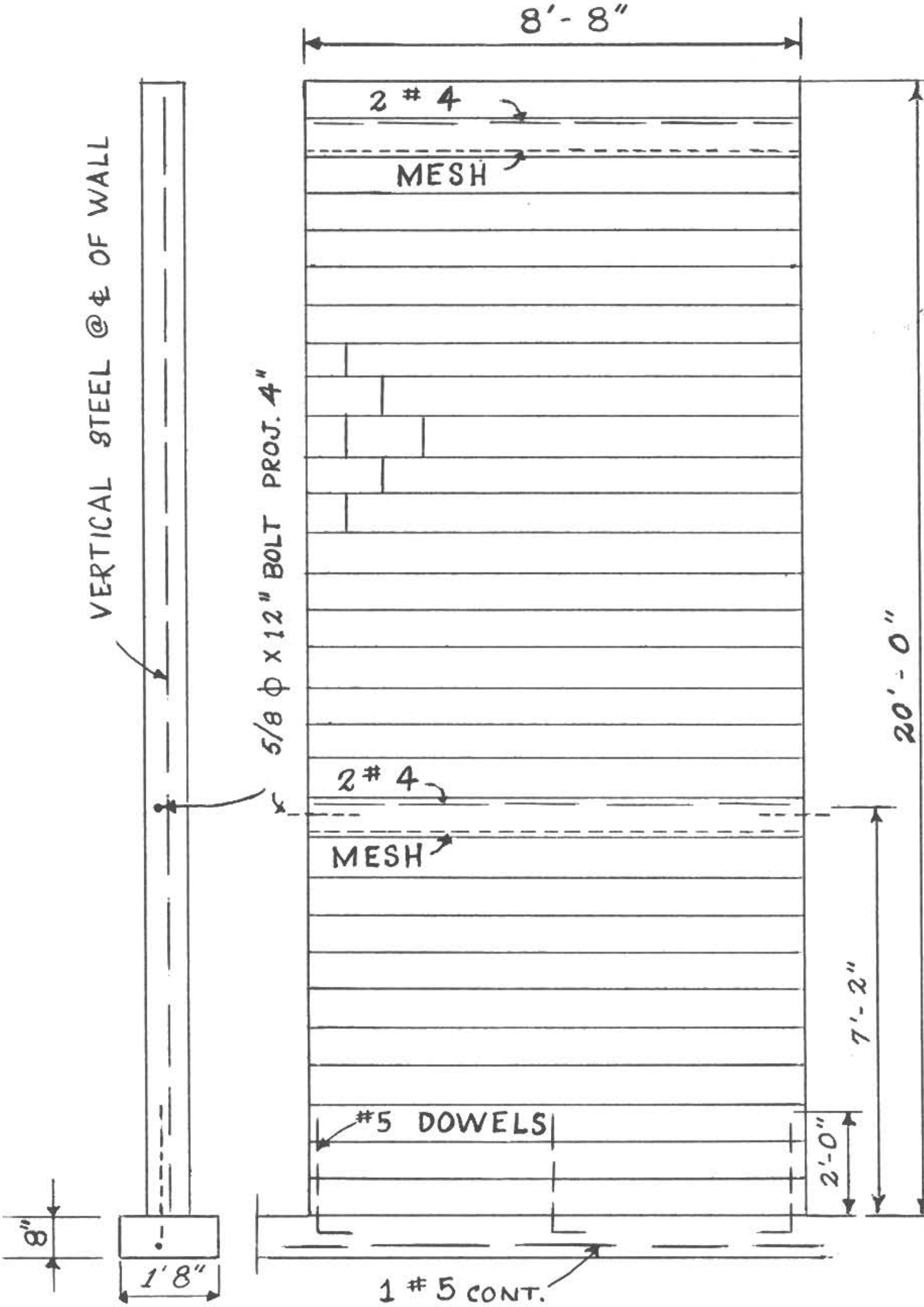
$\dfrac{.53 - .10}{.53} = 82\%$

MASONRY STRESS @ STEEL YIELD

$f_m = \dfrac{195,000 \times 7.92}{104 \times (3.5)^2} = 1210$ PSI

NO COMPRESSION FAILURES NOTED IN THE TEST

REINFORCING STEEL AREA AND EFFECTIVE DEPTH GOVERNED DEFLECTION CURVE



TYPICAL PANEL DETAILS