THE UNIVERSITY OF MANITOBA

SHEAR BEHAVIOUR OF MASONRY WALLS SUBDIVIDED BY FLOOR SLABS

BY

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TABLE OF CONTENTS

		Page
Title Page		i
Approval Sheet		ii
Abstract		iii
Acknowledgements		v
List of Tables		vi
List of Figures		viii
List of Photographs		x
CHAPTER I	INTRODUCTION	1
CHAPTER II	SHEAR STRENGTH OF WALLS	5
CHAPTER III	MATERIALS AND CONSTRUCTION DETAILS	10
CHAPTER IV	LOADING	17
CHAPTER V	TESTING APPARATUS AND PROCEDURE	23
CHAPTER VI	ANALYSIS BY FINITE ELEMENT METHOD	28
CHAPTER VII	THEORY AND RESULTS FROM STRAIN GAUGES	32
CHAPTER VIII	RESULTS OF TESTS	39
CHAPTER IX	DISCUSSION OF RESULTS	111
CHAPTER X	CONCLUSIONS	133
CHAPTER XI	RECOMMENDATIONS	135
LIST OF REFERENCES		137
APPENDIX A	Mortars and Concrete Data	140
APPENDIX B	Tensile Strength of Blocks	155
APPENDIX C	Rolling Resistance	160
APPENDIX D	Results from Strain Gauges	165
APPENDIX E	Deflection Data	173
APPENDIX F	Finite Element Data	199

Abstract

The use of concrete blocks as a building material has increased at a constant rate over the last ten years in almost every large city of Canada. The use of blocks is not a new idea - it is as old as the ancient megalithic structures of Greece, Egypt and Mesopotamia. The interest of the structural engineer in masonry construction is increasing because of the simplicity of design, the economy of construction and the pleasing appearance presented by masonry constructed structures.

Experimental work in this area has been carried out in many universities, and there is a large amount of information available to the structural engineer. However, one area which has not been investigated thoroughly is the shear capacity of the joint between the wall and the slab.

The present study looks into this area. The purpose of this work is to develop a rational procedure in calculating the shear capacity of the wall-slab joint and to find relationships between the type of construction (plain or reinforced masonry wall) and the type of mortar used.

The work is carried out experimentaly. Walls 5' x 8' made of 16" x 8" x 8" blocks and with an 8" slab at the mid point were made with different types of mortars. They were tested in direct shear with vertical load representing the live

and dead loads of an assumed 12-storey building. From the investigation, it was found that the shear capacity of the above mentioned connection depends on:

a) type of mortar as it relates to the shear capacity of the joint;

b) the vertical load on the connection as it relates to frictional resistance.

To increase the shear capacity of the wall-slab joint in masonry construction, it is recommended that high shear strength mortar be used with grooves cut off the block for better binding when small vertical loads exist, and in high seismic zones reinforcement should be present at the upper and lower part of the connection, with emphasis at the ends of the section for tension cracks which initiate at these points. After a compression stress of about 170 psi, the contribution of the mortar shear strength is small and it is shown in this work that the strength of the block is more important since the state of stress is such as to induce a failure other than shear failure. The existing code requirements are also discussed and a new allowable shear is recommended in this study.

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LIST OF TABLES

		Page
3-1	Walls Tested	15
4-1	Load Calculations of Wall	19
4-2	Reduction Factors	21
6-1	Stresses at Joint as Found Using Finite Element Analysis Method	31
8-1	Predicted Shear Capacity	41
8-2	Loading of Group A and Summary of Loads at Failure	42
8-3	Loading for Finite Element Analysis - Wall A_1	46
8-4	Deflections from Finite Element Analysis - Wall A_1	46
8-5	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall A_1, Corrected for Rolling Resistance	47
8-6	Loading for Finite Element Analysis - Wall A_2	52
8-7	Deflections from Finite Element Analysis - Wall A_2	52
8-8	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall A_2, Corrected for Rolling Resistance	53
8-9	Loading for Finite Element Analysis - Wall A_3	58
8-10	Deflections from Finite Element Analysis - Wall A_3	58
8-11	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall A_3, Corrected for Rolling Resistance	59
8-12	Loading for Finite Element Analysis - Wall A_4	64
8-13	Deflections from Finite Element Analysis - Wall A_4	64
8-14	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall A_4, Corrected for Rolling Resistance	65

		Page
8-15	Loading for Finite Element Analysis - Wall A_5	72
8-16	Deflections from Finite Element Analysis - Wall A_5	72
8-17	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall A_5, Corrected for Rolling Resistance	73
8-18	Loading of Group B with Loads at Failure	77
8-19	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall B_1, Corrected for Rolling Resistance	79
8-20	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall B_2, Corrected for Rolling Resistance	82
8-21	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall B_3, Corrected for Rolling Resistance	86
8-22	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall B_4, Corrected for Rolling Resistance	91
8-23	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall B_5, Corrected for Rolling Resistance	96
8-24	Loading of Group C and Test Results	100
8-25	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall C_1, Corrected for Rolling Resistance	102
8-26	Horizontal Displacement in Inches, Corrected for Rotation and Horizontal Forces, Wall C_2, Corrected for Rolling Resistance	107
9-1	Stresses at Selected Nodes	121

LIST OF FIGURES

		Page
2-1	State of Stress	7
2-2	Mohr's Circle Representation of State of Stress	8
3-1	Test Specimen	10
3-2	Type of Block Used for Walls	11
3-3	Contact Area Between Blocks	16
4-1	Section of Twelve-Storey Apartment Building	17
4-2	Typical Floor Plan and Cross Section of Apartment Building Showing Wall Sections Tested	22
5-1	Loading Apparatus Illustrating the Type of Supports and the Load Applying Mechanisms	26
5-2	Location of Deflection Measuring Devices	27
6-1	Loading and Types of Supports for Finite Element Analysis	30
7-1	Position and Numbering of Strain Gauges	33
7-2	Principal Strains	35
7-3	Strains in X-Direction, Wall A_1	38
8-1	Deflections Due to Horizontal Forces - Wall A_1	48
8-2	Load-Deflection Curve for a Point 5" Above the Slab on the Free of Stresses Side. Wall A_1	49
8-3	Deflections Due to Horizontal Forces - Wall A_2	54
8-4	Load-Deflection Curve for a Point 5" Above the Slab on the Free of Stresses Side. Wall A_2	55
8-5	Deflections Due to Horizontal Forces - Wall A_3	60
8-6	Load-Deflection Curve for a Point 5" Above the Slab on the Free of Stresses Side. Wall A_3	61
8-7	Deflection Due to Horizontal Forces - Wall A_4	66
8-8	Load-Deflection Curve for a Point 5" Above the Slab on the Free of Stresses Side. Wall A_4	67

		Page
8-9	Deflection Due to Horizontal Forces - Wall A_5	74
8-10	Load-Deflection Curve for a Point 5" Above the Slab on the Free of Stresses Side. Wall A_5	75
8-11	Deflections Due to Horizontal Forces - Wall B_1	80
8-12	Deflections Due to Horizontal Forces - Wall B_2	83
8-13	Load-Strain Relations	87
8-14	Deflections Due to Horizontal Forces - Wall B_3	88
8-15	Load-Deflection Curve for a Point 5" Above the Slab on the Free of Stresses Side. Wall B_3	89
8-16	Deflections Due to Horizontal Forces - Wall B_4	92
8-17	Load-Deflection Curve for a Point 5" Above the Slab on the Free of Stresses Side. Wall B_4	93
8-18	Deflections Due to Horizontal Forces - Wall B_5	97
8-19	Load-Deflection Curve for a Point 5" Above the Slab on the Free of Stresses Side. Wall B_5	98
8-20	Deflections Due to Horizontal Forces - Wall C_1	103
8-21	Load-Deflection Curve for a Point 5" Above the Slab on the Free of Stresses Side. Wall C_1	104
8-22	Deflections Due to Horizontal Forces - Wall C_2	108
8-23	Load-Deflection Curve for a Point 5" Above the Slab on the Free of Stresses Side. Wall C_2	109
9-1	Shear Distribution on 3-Block Shear Strength Test.	114
9-2	Assumed Loading of End Section	118
9-3	Analyzed Block Portion	120
9-4	Element and Nodes Analyzed	121
9-5	Failure Stages of Loaded Blocks	122
9-6	Relation Between Vertical Stress and Shear Capacity	125

LIST OF PHOTOGRAPHS

		Page
7-1	Wall A_1 with Strain Gauges	37
7-2	Wall B_3 with Strain Gauges and Deflection Measuring Devices Ready to be Tested.	37
8-1	Wall A_1 after Failure in Shear	50
8-2	Wall A_2 Placed on Testing Apparatus	56
8-3	Shear Failure of Wall A_2	56
8-4	Vertical Crack on Wall A_4	68
8-5	Wall A_4 after Explosive Failure	68
8-6	View of Wall A_4 Showing Severe Cracking	69
8-7	Final Condition of Wall A_4 after Loaded to 52 KIPS Horizontally	69
8-8	Wall A_5 after Failure	76
8-9	Third Vertical Block from Left Corner after Failure	76
8-10	Failure of Wall B_4	94
8-11	View of Failure - Wall B_4	94
8-12	Wall B_5 after Failure	99
8-13	Clear Shear Failure View of Wall C_1 at the Joint	105
8-14	Failure of Wall C_2	110
8-15	Close-up of Failure of Wall C_2	110
9-1	Failure Mechanisms of Loaded Block	127
9-2	Damage to Light Building in San Fernando Earthquake	128
9-3	View of Earthquake Damage	128
9-4	The damage in this picture is similar to the failure of Wall A_4	129

To my parents, especially my mother,

who always thought I would amount to nothing.

CHAPTER I

Introduction

Lateral loads in the design of tall buildings require special consideration by the designer. These loads are induced on the structure primarily by earthquakes and wind. Blast loads and loads due to atomic explosions are not commonly considered due to the difficulty in predicting the probability of occurence and the economics of such a design.

The most common method of designing for earthquakes for relatively small, uncomplicated structures is the use of a certain portion of the total load above the point of consideration as the force induced by the earthquake. Many empirical formulae are available for calculating the force on the structure due to earthquakes at a given level. The most common one is

$$V = KCW$$

(Canadian Structural Design Manual)[1]

where W is the total weight of the structure, C is a coefficient given by the relation

$$C = \frac{.05}{\sqrt[3]{T}}$$

where T equals the period of vibration of the building given by

$$T = \frac{.05H}{\sqrt{D}}$$

where H is the height of the building and D is the dimension

parallel to the direction of motion.

K is a coefficient depending on the ability of the structure to withstand movement as a unit.

For buildings with a complete moment-resisting space frame the recommended value of K is .67. For buildings with a box system the value of K is 1.33. (Canadian Structural Design Manual.) [1]

In the case of wind load the approach is similar. The lateral force is given by some empirical relations, the most common of which is

$$V = q\ Ce\ Cg\ Cp$$

(1970. N.B.C. 4.16.1. (1)) [2]

where V is the design external pressure in the direction normal to the surface as pressure or as suction

q is a velocity factor

Ce is an exposure factor

Cg is the gust effect

Cp is external pressure coefficient.

The values of these coefficients can be found in any design manual which provides for wind loading. [1]

Once the external loads due to these effects are estimated, then the structure is checked to see if it can withstand these forces.

For buildings designed with columns, beams and slab systems, the check is easily done especially if a computer is used. Recently the use of shear wall structures has produced a situation in which, if further development is to be made with safety, a greater knowledge of their structural behavious is necessary. Recently a great effort has been directed towards this area, but only for shear walls, and their interactions with slabs and service-cores which have been constructed cast in place. The increasing use of concrete blocks to construct shear walls requires research in this area.

The damage that occurred in the San Fernando earthquake at the Juvenile Facility, whose buildings were braced by concrete masonry shear walls, showed that although the allowable shear stresses were 33 psi (2.31 K/cm^2), (A.C.I. Journal, May, 1972),[3] the average shear stresses were 87 psi. This is not the ultimate, since the walls did not fail in shear. The walls showed little effect from seismic shear of at least 87 psi. It appears that the maximum allowable shear stresses for reinforced concrete block walls could be considerably increased and still have an adequate factor of safety.

The above mentioned earthquake had an estimated acceleration of 0.83g, a force not normally provided for in the design. (A.C.I. Journal, May, 1972).[3]

It is the purpose of this study to look into the shear capacity of masonry constructed walls subdivided by slabs and to give an explanation of the behaviour of masonry constructed buildings in the San Fernando Earthquake. The effects of using different mortars and types of construction are investigated in the study.

CHAPTER II

Shear Strength of Walls.

Masonry walls subjected to combined compression and shear exhibit two distinct types of failure:

1) <u>shear failure at the block/mortar interface</u>. The shear strength in this case consists of the initial bond shear and the resistance due to the normal stress. This resistance is attributed to the friction between block and mortar. The maximum stress then at failure would be

$$V = Vo + \mu\sigma y$$

where

V = maximum shear stress

Vo = shear strength of mortar in pure shear

μ = friction coefficient

σy = normal stress

From this relation for a constant μ and σy the ultimate shear strength will depend only on the shear capacity of the mortar.

2) <u>diagonal tensile cracking through bricks and mortar, governed by the maximum tensile stress</u>. The available information for this tensile strength, obtained experimentally, indicates that

$$f_t = k\sqrt{f_c'} \qquad (2)$$

where

f_t - tensile strength

k = splitting ratio $\geqslant 2$.

f_c' = compressive strength of block.

The splitting ratio is found by the author to be 2 in accordance with the A.C.I. Building Code which indicates that for $f_c' = 2500$, $k = 2$ (Table 3, 6.2.3.)[1] Also see Appendix B for experimental data.

The compressive strength of the blocks was found experimentally to be 2480 psi (see Table A-8 in the appendix). Then

$$f_t = 2\sqrt{2480} = 2 \times 49.8 = 99.6 \approx 100 \text{ psi}$$

If we assume that the maximum tensile stress governs failure then we can use the relation

$$f_t = \sqrt{\frac{\sigma y^2}{4} + \tau^2} - \frac{\sigma y}{2} = \text{constant} \qquad (1)$$

This relation is obtained assuming that any element on the structure is at a state of stress as shown in Figure 2-1. The figure shows an element in the interior of the wall. The vertical shear stress shown on the figure is a result of the vertical load.

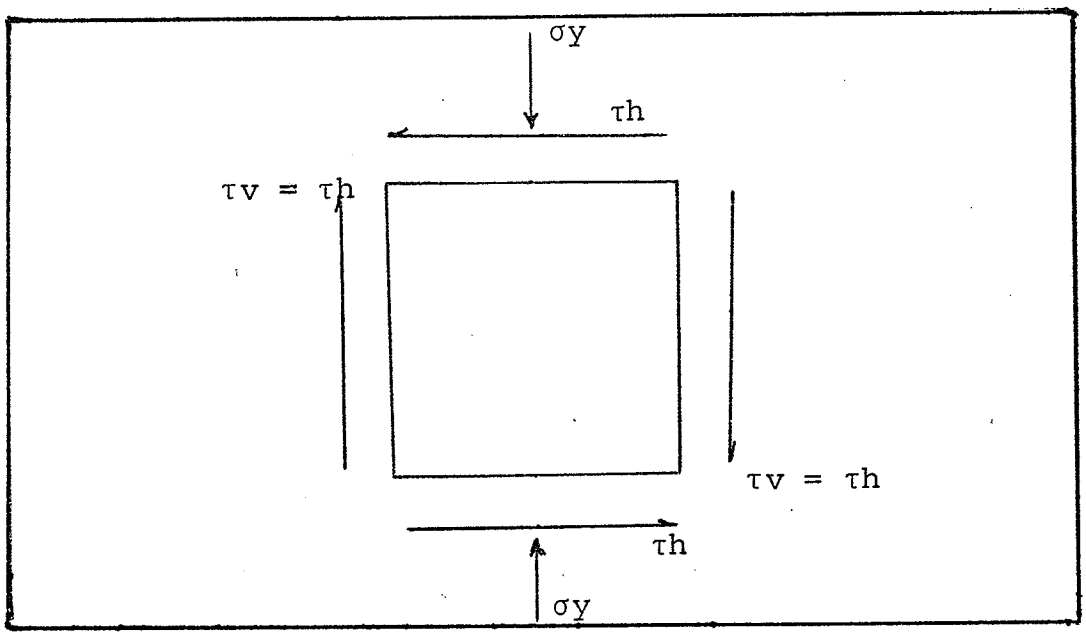


FIGURE 2-1. STATE OF STRESS

This is a reasonable approximation because the horizontal force is resisted by the shear stress developed, and the vertical edges of the wall are free of stress. (See Figure 3-1.)

With this assumed stress distribution, Equation (1) can be derived from equilibrium.

The stresses from the element, plotted on a Mohr's circle, give the diagram shown in Figure 2-2. The sign convention is compressive stress positive and counter clockwise shear positive.

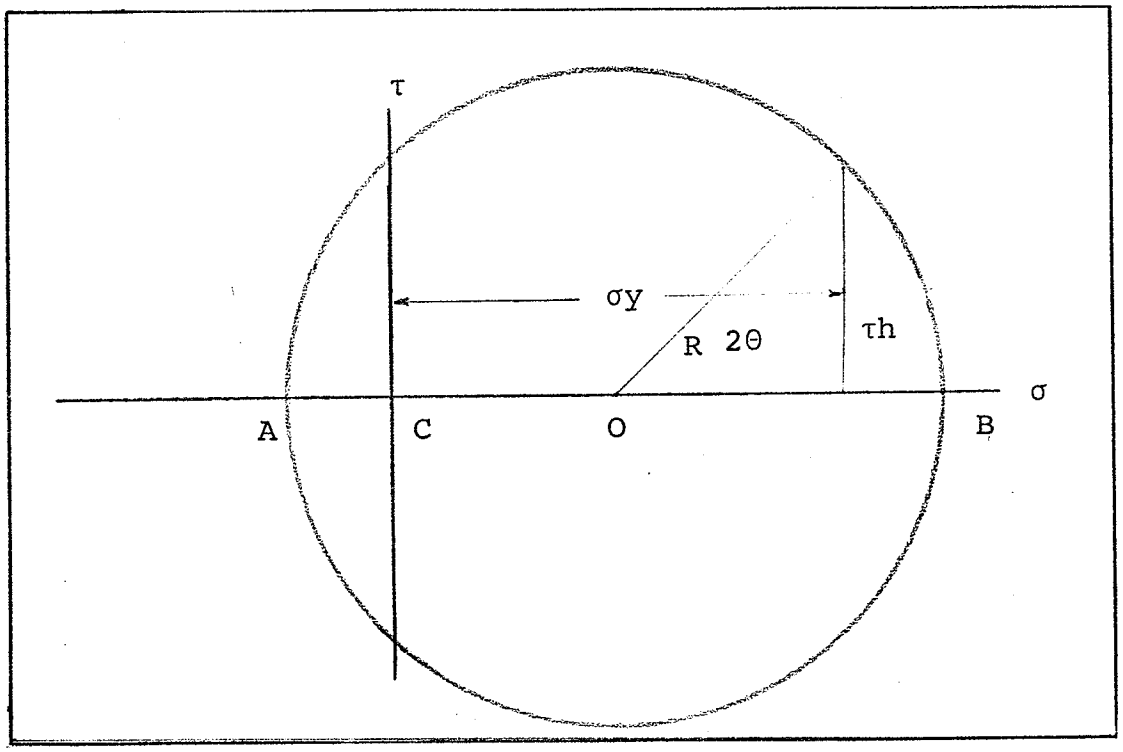


FIGURE 2-2. MOHR'S CIRCLE REPRESENTATION OF STATE OF STRESS

From this diagram it is easily observed that the maximum tensile stress would be

$$AC = R - OC = \sqrt{\frac{\sigma y^2}{4} + \tau_h^2} - \frac{\sigma y}{2} = f_t \qquad (2)$$

R being the radius of the circle. The angle of the plane where this stress occurs is given by

$$\theta = \frac{1}{2} \tan^{-1} \left(\frac{\tau}{\sigma y}\right) \quad \text{(relative to vertical)} \qquad (3)$$

The value of the horizontal shear strength has two components:

Vo and $\mu\sigma y$

where Vo = bond shear strength

$\mu\sigma y$ = frictional resistance

μ = coefficient of friction.

It is worth noting here that failure of the wall in compression is not considered because the vertical stresses are assumed to be less than the compressive strength of the concrete blocks.

From the above relations, the shear capacity of the connection can be calculated, if f_t, Vo, σy and μ are known. The equation $f_t = \sqrt{\frac{\sigma y^2}{4} + V^2} - \frac{\sigma y}{2}$ (4)

can be solved for V and from the relation

$$V = Vo + \mu\sigma y \qquad (5)$$

the contribution of the mortar can be calculated. (Value of Vo in (5) is the mortar contribution.)

If f_t is not to exceed 100 psi, then the expression for maximum tensile stress (4) becomes

$$100 = f_t = \sqrt{\frac{\sigma y^2}{2} + V^2} - \frac{\sigma Y}{2} \qquad (6)$$

In summary, then, it is proposed that the strength of the shear wall be estimated by simple maximum stress theory, but with V the maximum shear stress at failure as a function of the vertical stress σy. It is further assumed that the average shear stress concept can be used for the wall. Since the material is brittle, this assumption is justified. In this study, μ is taken as0.75 which is an average of values found in leterature.

CHAPTER III

Materials and Construction Details

Test Specimens

The tested specimens consist of wall sections made by using concrete blocks. The dimensions and the layout are shown in Figure 3-1.

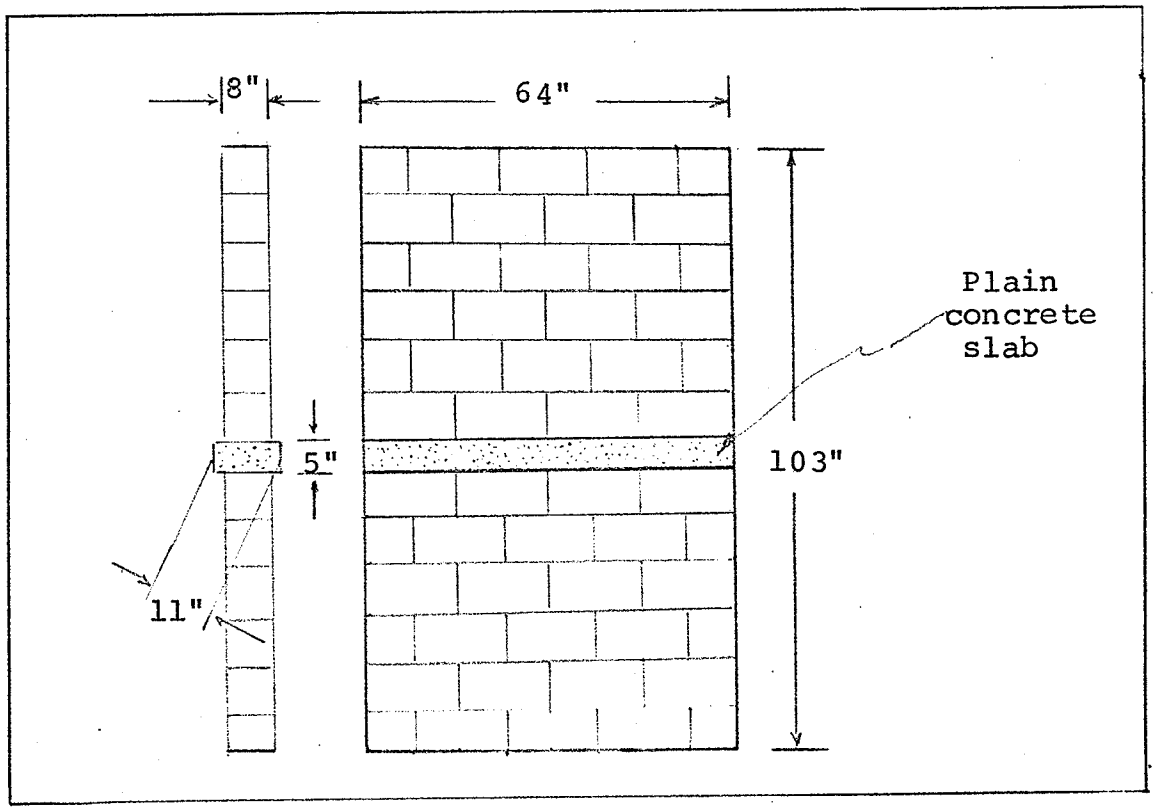


FIGURE 3-1. TEST SPECIMEN

The figure illustrates the number of blocks and the overall dimensions of the walls.

Blocks

For the construction of the walls, hollow concrete blocks 8" x 8" x 16" were used, of the form shown in Figure 3-2, and with a crushing strength of 2480 psi. See Appendix A, Table A-9 for compressive strength.

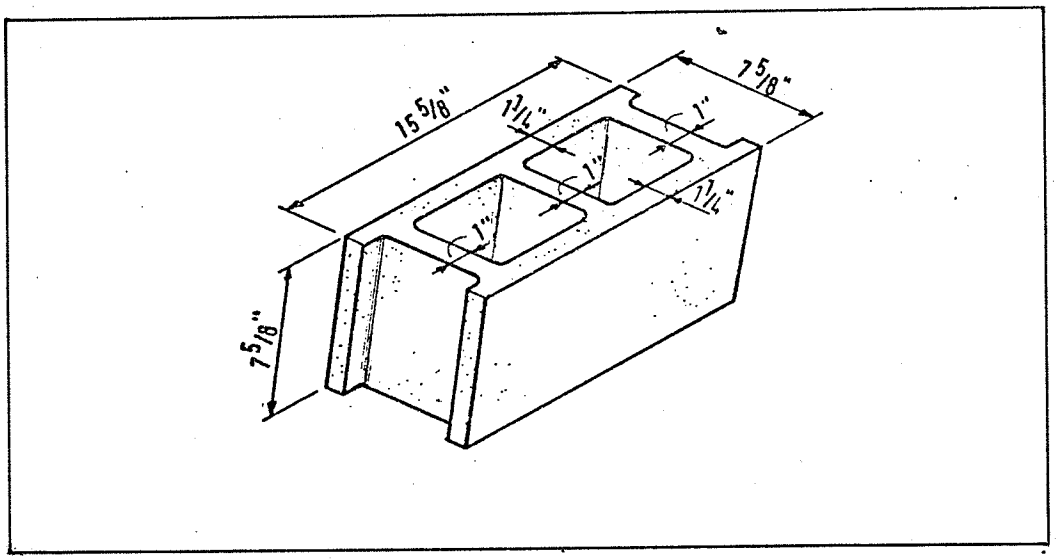


FIGURE 3-2. TYPE OF BLOCK USED FOR THE WALLS

Equivalent Thickness of Blocks

The equivalent solid thickness of the blocks used in the investigation may be calculated using the relations

$$\text{Eq.th.} = \frac{TPs}{100}$$ where T = actual width of unit

Ps = percent of solid volume

gross volume = 7 5/8 x 7 5/8 x 15 5/8

= 910 in^3

Solid volume = gross volume - void volume

$$= 910 - ((5\ 3/4 \times 4\ 3/4 \times 7\ 5/8)\ 2 + 2\ (3/4 \times 3\ 5/8 \times 7\ 5/8))$$

$$= 910 - (416 + 41.5) = 452.5\ in^3$$

$$Ps = \frac{452.5}{910} \times 100 = 49.75\%$$

$$Eq.th. = \frac{7.625 \times 49.75}{100} = 3.8"$$

Another method for calculating the equivalent thickness is the method of moment of inertia. The moment of inertia of the area of the block about the center is

$$I = \frac{7.625 \times 15.625^3}{12} - 2\ (\frac{4.625}{12} \times 5.625^3 + 4.625 \times 5.625 \times 2.812^2 + 4.625 \times \frac{0.75^3}{12} + 4.625 \times .75 \times 6.425^2)$$

$$= 2420 - 2(68.6 + 207 + .12 + 143.5)$$

$$= 2420 - 2(420) = 2420 - 840 = 1580\ in^4$$

Using the same length h = 15.625" the equivalent thickness - b - can be found using the relation

$$I = \frac{bh^3}{12} = 1580$$ substituting for h and solving for b

we obtain

$$b = \frac{1580 \times 12}{15.625^3} = 4.95"$$

i.e. equivalent thickness using this method is 4.95 inches.

From the two methods for calculating equivalent thickness the one that uses the volumes is believed to give an equivalent thickness which is better for our study because the walls are subjected to lateral loading in which contact area is more important. If bending was the governing factor, then the inertia approach would be more appropriate.

Cement and Sand

The sand used for the mortar is the masonry sand with properties given in more detail on page142 of Appendix A. The sand's fineness modulus was 2.84.

Two types of cement were used; normal Portland cement Type 1 and masonry cement.

Mortar: A ratio of 1:4 cement and sand by weight was used in preparing the mortar.

As an additive, a special blend based on polyvinyl acetate was used. The effects of adding this material are discussed in Appendix A.

The shear strengths of the mortars were:

a) for mortar produced by using 1:4 masonry-cement to sand - 48.5 psi (experimental average).

b) for mortar with the same masonry-cement to sand ratio and 10% additive with respect to the weight of cement - 54.5 psi. (experimental average.)

c) Mortar produced by using normal Portland cement and sand with ratio 1;4 and 10% by cement weight additive - 158 psi (experimental average).

All shear strengths refer to a 28-day test. Further information on mortar can be found in Appendix A. The shear strength of the mortars was found using 3-block tests. The procedure for these tests is given in Appendix A.

The complete contact area across the wall at the joint was estimated as 1.5 x 2 x 63.5 = 190.5 in^2, the total length of the wall being 63.5 inches. In the calculation of shear, this contact area is used as the resisting surface, except for the filled walls in which the concrete fill takes part in resisting the shear. See Figure 3-3.

The fifteen walls were divided into three groups.

Group A

The mortar used was the previously mentioned, with shear strength of 48.5 psi. The blocks were not filled.

Group B

This group had the same materials as Group A with the use of mortar with 54.5 psi shear strength.

Group C

This group used mortar with shear strength of 158 psi. Four walls were filled with 2500 psi concrete. One was left hollow.

A summary table of the test specimens indicating their properties is Table 3-1. The vertical stresses in this table are obtained from information given in Chapter IV.

TABLE 3-1. WALLS TESTED

Wall	Mortar Strength psi	Compressive Strength of Mortar psi	Compressive Strength of Blocks	Tensile Strength of Blocks psi	Vertical Stress σ_y psi
A_1	48.5	2150	2480 psi	100	12.3
A_2	48.5	2150	2480 psi	100	82.5
A_3	48.5	2150	2480 psi	100	149
A_4	48.5	2150	2480 psi	100	210
A_5	48.5	2150	2480 psi	100	270
B_1	54.5	3320	2480 psi	100	12.3
B_2	54.5	3320	2480 psi	100	12.3
B_3	54.5	3320	2480 psi	100	149
B_4	54.5	3320	2480 psi	100	210
B_5	54.5	3320	2480 psi	100	270
C_1	158	3340	2480 psi	100	0
C_2	158	3340	2480 psi	100	12.3

Assembly

The wall panels were built vertically on a base so they could be lifted and placed under the testing apparatus. Figure 3-1 shows a wall with the slab section at the middle. The slab had a 5-inch thickness and the concrete used for the slab sections had an average compressive strength of 2500 psi.

Fifteen wall sections were built, all of them using the same blocks, the same slab thicknesses and the same overall dimensions, except for mortar variations. No reinforcement was provided in any of the walls, in the vertical or horizontal directions. Mortar was placed on the outer part of the blocks as shown in Figure 3-3.

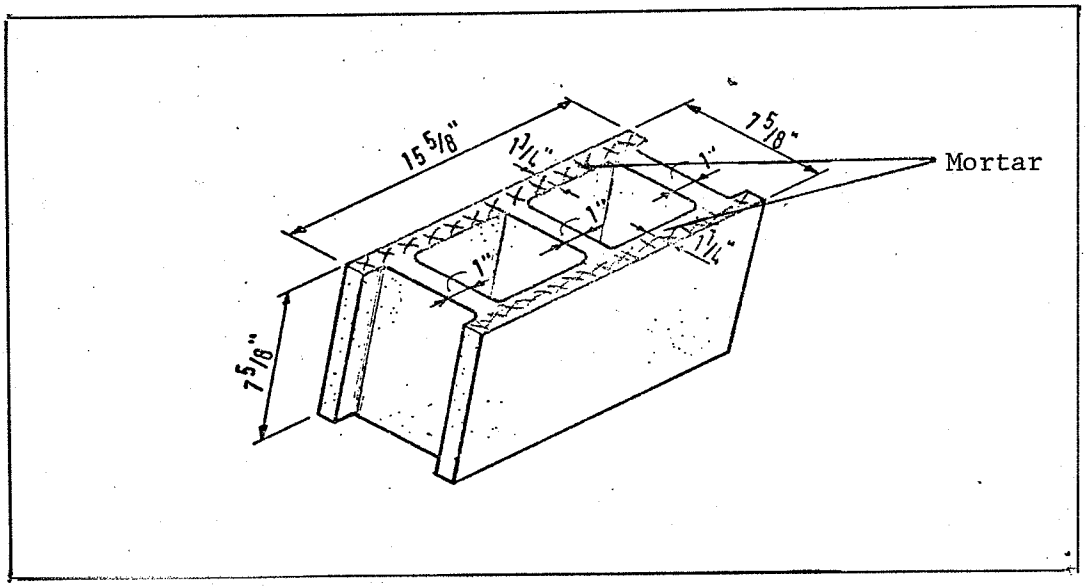


FIGURE 3-3. CONTACT AREA BETWEEN BLOCKS

CHAPTER IV

Loading

Vertical Loads

The vertical loads applied on the walls were chosen to represent loads on the floor level at various storey-heights of a 12-storey apartment building, as shown in Figure 4-1.

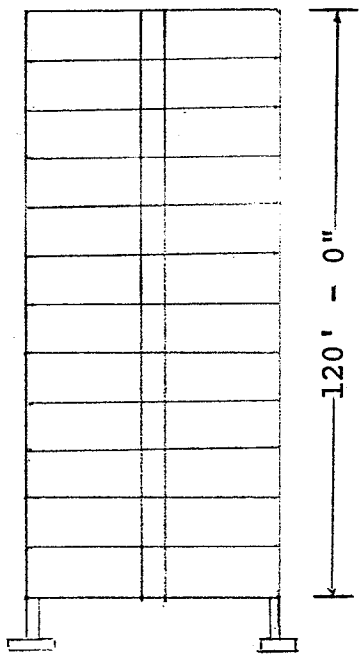


FIGURE 4-1. SECTION OF 12-STOREY APARTMENT BUILDING

The plan of a typical floor in such a building is shown in Figure 4-2. The wall investigated in this study is the wall of line 3, which is shown on the plan diagram. The dimensions are shown on the plan with the assumption that the loading is:

Roof	Live Load		50 lb/ft^2
	Dead Load	5 lb/ft^2	
	5" slab	65 lb/ft^2	(Table 4.1.4.A. 1970. National Building Code)
	ΣDL	70 lb/ft^2	
Floor Level			
	Live Load		40 lb/ft^2
	5" slab	65	
	ΣDL	65 lb/ft^2	

The loads per linear foot are calculated and they are given in Table 4-1 for every floor level.

TABLE 4-1. LOAD CALCULATIONS OF WALL

Tributary Area 21 x 21 = 420 Ft^2

Storey Height 10'

A.S.	L.L.R.F.	L.L.	R.L.L.	D.L.	Des.L.	Comments	Level
420	1.00	1.05	1.05	1.47 .44 1.91	2.96klf	LL=4.50x21= 1.05 klf DL=70x21= 1.47 klf weight of block.44klf	Roof 12
420	0.80	0.80	.64 1.69	1.74 3.68	5.34	DL=65x20+ wall=1.74 LL=40x20= .80	11
420 840	0.65	0.80 1.60	1.04 2.09	1.74 5.45	7.57		10
420 1260	0.60	0.80 2.40	1.44 2.49	1.74 7.19	9.68	Beyond cap- acity of hollow blocks. Go to filled wall.8x8x16	9
420 1680	0.55	0.80 3.20	1.76 2.81	2.21 9.40	12.21	DL= 1.47 Blocks .74 2.21	8
420 2100	0.55	0.80 4.00	2.20 3.25	2.21 11.61	14.86		7
420 2520	0.50	0.80 4.80	2.40 3.45	2.21 13.82	17.27		6
420 2940	0.50	0.80 5.60	2.80 3.85	2.21 16.03	19.88	Beyond cap- acity.Go to 10"x10"x16". DL=1.47+93= 2.40	5
420 3360	0.50	0.80 6.40	3.20 4.25	2.40 18.40	22.60		4

Continued

TABLE 4-1. (Continued)

A.S.	L.L.R.F.	L.L.	R.L.L.	D.L.	Des.L.	Comments	Level
420 / 3780	0.50	0.80 / 7.20	3.60 / 4.65	2.40 / 20.80	25.45	Beyond capacity of 10" Go to 12" block. DL=1.10+1.47 = 2.57	3
420 / 4200	0.50	0.80 / 8.00	4.00 / 5.05	2.57 / 23.37	28.42		2
420 / 4670	0.45	0.80 / 8.8	4.40 / 5.45	2.57 / 25.94	31.49		1
420 / 5040	0.45	0.80 / 9.60	4.80 / 5.85	2.57 / 28.51	34.36		Ground

A.S. = Area Supported

L.L.R.F. = Live Load Reduction Factor

L.L. = Live Load

R.L.L. = Reduced Live Load

D.L. = Dead Load

Des.L. = Design Load

NOTE: The reduction factors used in this table are given in Table 4-2.

The dead weight of the blocks includes 5 lb/ft^2 for finishing both sides. The reduction for live load is calculated using the formula

$$R.F. = .3 + \frac{10}{\sqrt{A}}$$

(Section 4.1.4.3.6. 1970. National Building Code)[2]

where A = tributary area.

Also, the assumption is made that the safe load capacity of hollow blocks is 1000 psi and of filled blocks, 1800 psi.

TABLE 4-2. REDUCTION FACTORS[2]

Factor	Tributary Area (Sq.Ft.)	Factor	Tributary Area (Sq.Ft.)
1.00	0 - 236	0.60	1120 - 1600
0.95	236 - 278	0.55	1600 - 2500
0.90	278 - 330	0.50	2500 - 4450
0.85	330 - 400	0.45	4450 - 10000
0.80	400 - 493	0.40	10000 - 40000
0.75	493 - 625	0.35	40000 - 100000
0.70	625 - 825	0.30	100000 - ∞
0.65	825 - 1120		

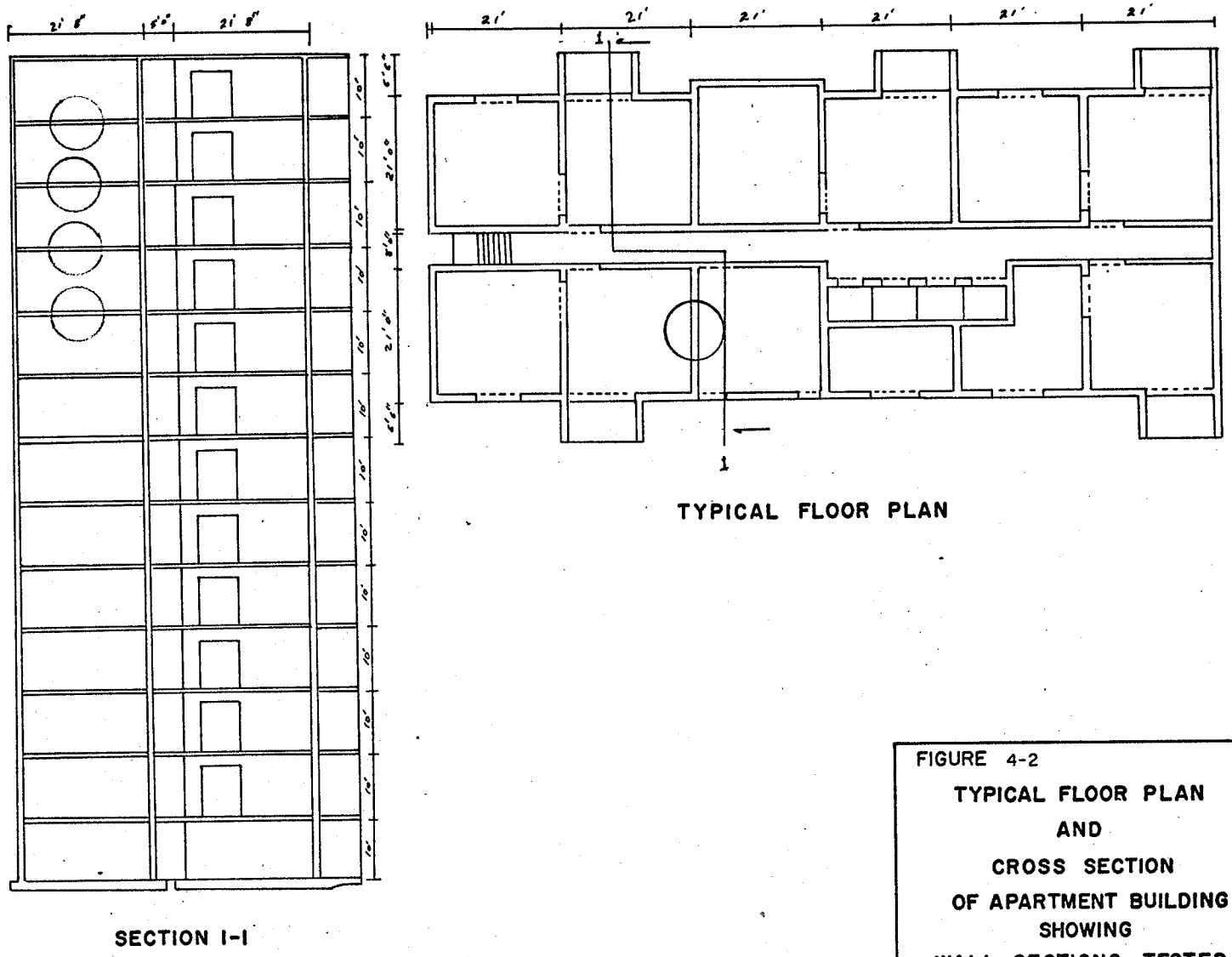


FIGURE 4-2

TYPICAL FLOOR PLAN
AND
CROSS SECTION
OF APARTMENT BUILDING
SHOWING
WALL SECTIONS TESTED

CHAPTER V

Testing Apparatus and Procedure

The wall sections were lifted and placed under the loading apparatus, pre-compressed by means of two vertical rods on each side to avoid damage in transporting them. The walls on the loading frame were supported by two 12"-deep I'beams, 83" long, bolted at the ends and supported at the center by a 12"-deep I-beam. The two I-beams were connected at the bottom, the ends and at the center by 1/2" steel plates, and the top was covered by a 1/4" thick plate, 6 inches wide, 83" long, and which was welded onto the two beams. See Figure 5-1. Rollers 1 1/2" in diameter were placed on top of the 1/4" plate at 6" center to center. On top of the rollers was placed a 1/2" x 6" x 83" steel plate upon which the wall was set.

Horizontal support was provided by a heavy steel section and 3/4" diameter rollers provided freedom of movement in the vertical direction. This heavy steel section was supported at the mid point by means of a pivot to provide rotational freedom.

The vertical load was provided by a 200,000 lb. capacity loading jack and the load was distributed uniformly by a stiffened I-column section resting on 1" diameter rollers. The rollers were resting on a 1/4" x 6" x 64" plate at 6" center to center. (See Figure 5-1.)

The horizontal load was provided by four 10-ton capacity jacks spaced 12" apart. The load was transmitted to the wall by a steel plate 1/2" x 6" x 50" thick. Provision for freedom of movement was made by means of rollers arranged similarly to those previously mentioned. The vertical load was applied first and the horizontal load was applied in increments of 2000 lb - the first horizontal load being 4000 lb. At every increase of horizontal load, the vertical load was checked and was maintained the same throughout the test.

To measure deflections and to correct these measurements for rotation, dial gauges were attached onto a frame which was welded at the lower part on to the I-beam supporting the wall. The position of the dial gauges is illustrated in Figure 5-2. This provision was made to avoid measuring relative movements instead of actual deflections of the wall. At this point, provision was made to measure the rolling resistance of the rollers, once the vertical load was applied, in order to adjust the horizontal applied force. The correction for rolling resistance applies for vertical loads higher than 20^k in which the smaller vertical loads were applied using a smaller jack and loading cell with a steel ball transfering the load. Although the rolling resistance in this case was very small, corrections were made. See Appendix C, page 160.

Dials 1 and 2 measured vertical displacements and dials 3, 4, 5 and 6 measured horizontal displacement. The readings of dials 1 and 2 were used to correct the readings of the other four

dials for rotation.

On two of the walls (Wall A_1 and B_3), strain gauges were used to investigate the stresses at these points. The position of these gauges is shown in Figure 7-1.

Due to limitations of cost, no strain gauges were used on the other walls.

The walls tested and their properties are given on Table 4-1.

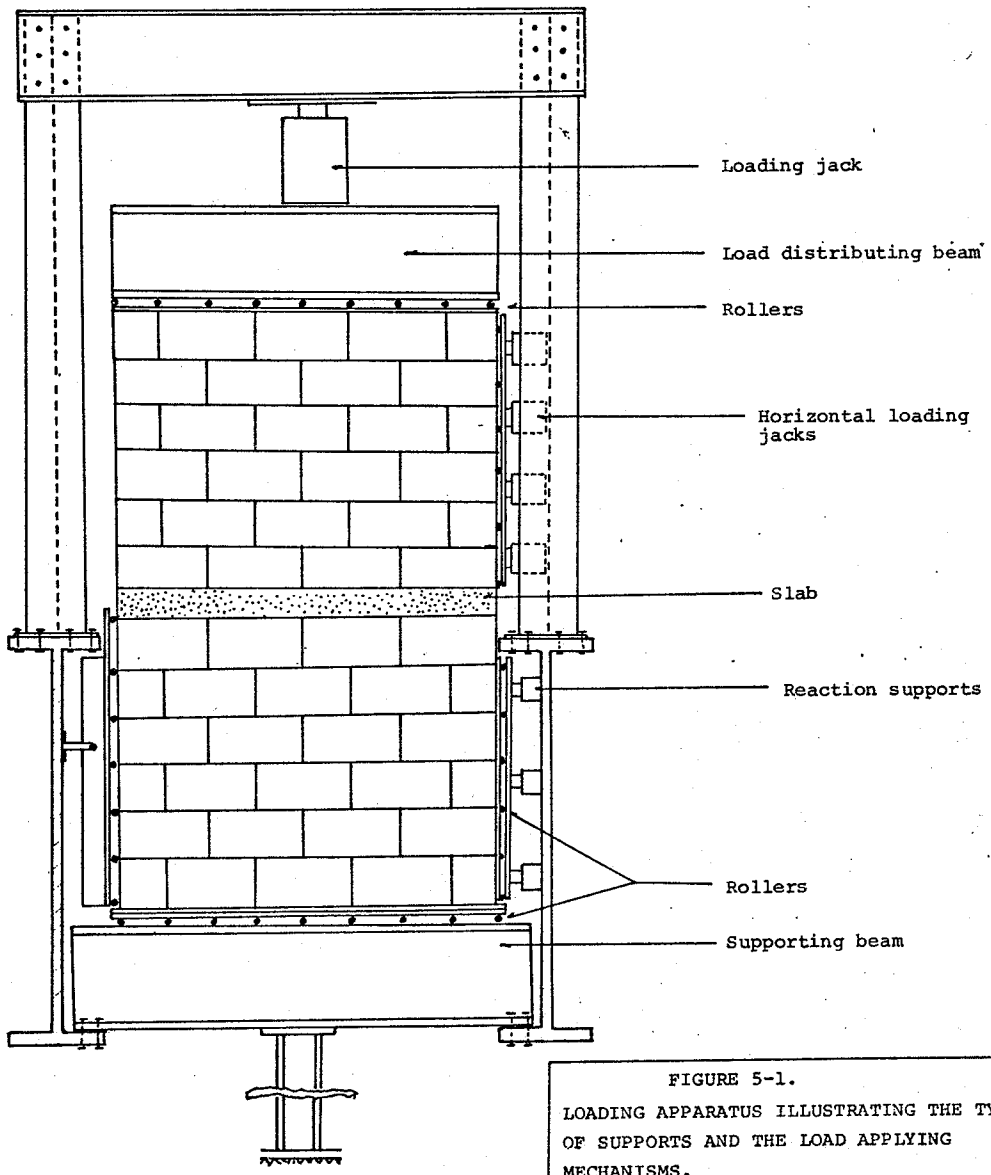


FIGURE 5-1.
LOADING APPARATUS ILLUSTRATING THE TYPE OF SUPPORTS AND THE LOAD APPLYING MECHANISMS.

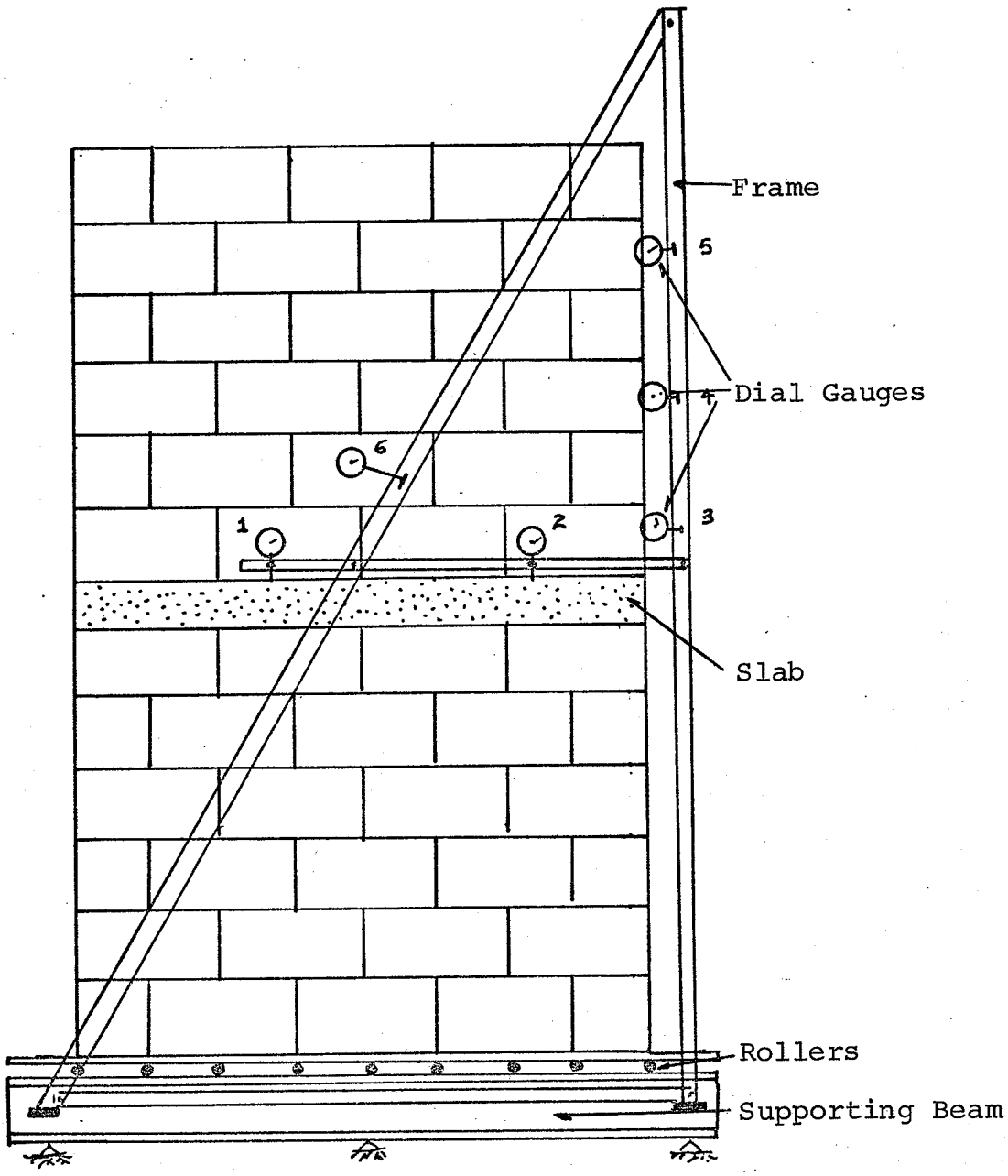


FIGURE 5-2. LOCATION OF DEFLECTION MEASURING DEVICES

CHAPTER VI

Analysis by Finite Element Method

Along with the experimental investigation, application of the finite element analysis was made and stresses and deflections were compared with those found using analytical and experimental means for Walls A_1 to A_5. The analytical results were obtained using an existing computer program, the results of which are given in Appendix VI. The loads used in the computer analysis were the experimentally found failure loads.

If the theory discussed in Chapter III is reasonable and applicable, then the stresses at any point on the element, and especially in the connection zone, should be reasonably close to those which can be found using the finite element analysis computer program from the computer library of the University of Manitoba. See Appendix F. In this analysis, the following assumptions are made:

a) The modulus of elasticity of mortar is the same as that of the concrete blocks and equal to 3000 ksi;

b) The equivalent thickness of the block used for the hollow walls is 3.8 inches, as calculated on page 12;

c) Poisson's ratio of mortar is the same as the ratio of concrete and equal to 0.20;

d) Support conditions are as shown in Figure 6-1;

e) The loads used in the analysis are:

i) vertical - the calculated loads (dead and live load) of the twelve-storey apartment building (See Table 4-1). These are the same vertical loads used in the experimental work;

ii) horizontal - experimentally found shear capacity of the walls.

For the analysis, the wall was divided into 104 rectangular elements, 126 nodes as shown in Figure 6-1. The loads applied are also shown in Figure 6-1.

A complete statement of the results of this analysis is given in Appendix F. The stresses at the joint between slab and wall are given in Table 6-1. The displacements of characteristic points are plotted with the corresponding values obtained from the experimental work in Chapter VIII.

This analysis uses an elastic solution on the ultimate strength of the structure. However, this can be justified if we state that what we are interested in obtaining is the point of maximum stress, and if this is known, the place where failure or cracking is expected to initiate can be predicted.

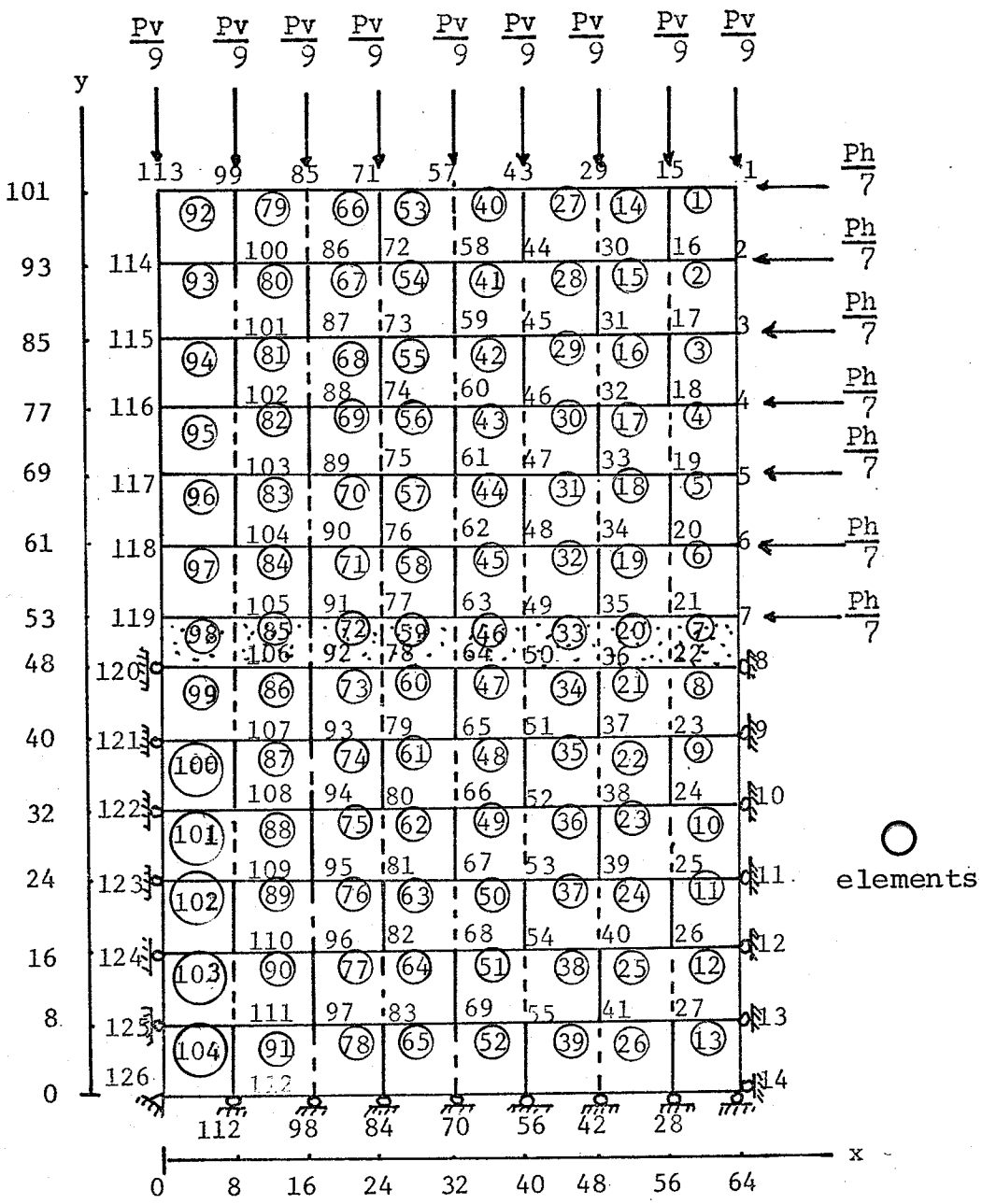
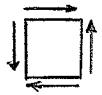


FIGURE 6-1. LOADING AND TYPE OF SUPPORTS FOR FINITE ELEMENT ANALYSIS

TABLE 6-1. STRESSES AT JOINT AS FOUND USING FINITE ELEMENT ANALYSIS METHOD (IN KSI)

Wall	Direction	Node									Average Shearing Stress
		7	21	35	49	63	77	91	105	119	
A_1	x	.255	0.760	.450	.349	.221	.115	.046	.012	.003	.224 ksi
	y	4.92	5.38	2.77	2.05	1.61	1.24	.876	.48	.274	
	xy	3.26	1.88	.982	.329	.170	.650	1.170	1.74	2.041	
A_2	x	.195	.329	.431	.335	.212	.110	.044	.012	.003	.182 ksi
	y	4.89	4.04	2.73	2.02	1.58	1.216	.854	.468	.268	
	xy	3.29	2.34	1.04	.364	.132	.651	1.129	1.699	2.00	
A_3	x	.163	.301	.411	.321	.202	.103	.041	.011	.002	.173 ksi
	y	4.88	4.02	2.71	1.99	1.19	1.55	.835	.457	.262	
	xy	3.32	2.37	1.04	.402	.573	.093	1.087	1.65	1.957	
A_4	x	.115	.260	.381	.299	.186	.094	.035	.008	.002	.155 ksi
	y	4.89	4.02	2.69	1.973	1.52	1.16	.813	.445	.256	
	xy	3.38	2.43	1.105	.459	.034	.513	1.028	1.598	1.89	
A_5	x	.128	.270	.387	.301	.191	.097	.038	.010	.002	.158 ksi
	y	4.810	3.966	2.104	1.942	1.500	1.166	.801	.439	.253	
	xy	3.330	2.397	1.211	.441	.048	.555	1.041	1.600	1.90	

Direction of Stress 

CHAPTER VII

Theory and Results from Strain Gauges

The strains on a surface are conveniently measured by means of electric-resistant strain gauges. The most simple strain gauge is a short length of wire insulated from, and glued to the surface. When stretching occurs the resistance of the wire is increased, and the strain can thus be measured electrically. The use of strain gauges is simple when the principal directions are known. The gauges are placed along each principal direction and measure principal strains εx and εy. The principal stresses σx and σy then may be calculated from Hooke's law. An assumption is here made that no stresses act normal to the surface where the gauges were glued.

i.e. $\sigma z = 0$. Then (4)

$$(1 - \nu^2)\ \sigma x = E(\varepsilon x + \nu \varepsilon y),$$

$$(1 - \nu^2)\ \sigma y = E(\varepsilon y + \nu \varepsilon x)$$

If the principal stress directions are not known, the measurements must be made in three directions at the point of consideration. Such arrays of strain gauges are called "Strain Rosettes", and measure strain in three or more directions.[4] The "crowfoot" rosette is indicated in Figure 7-1.

Strain gauges were attached on Walls A_1 and B_3. The walls are shown on Photos 7-1 and 7-2. Figure 7-1 shows the direction and location of the strain gauges.

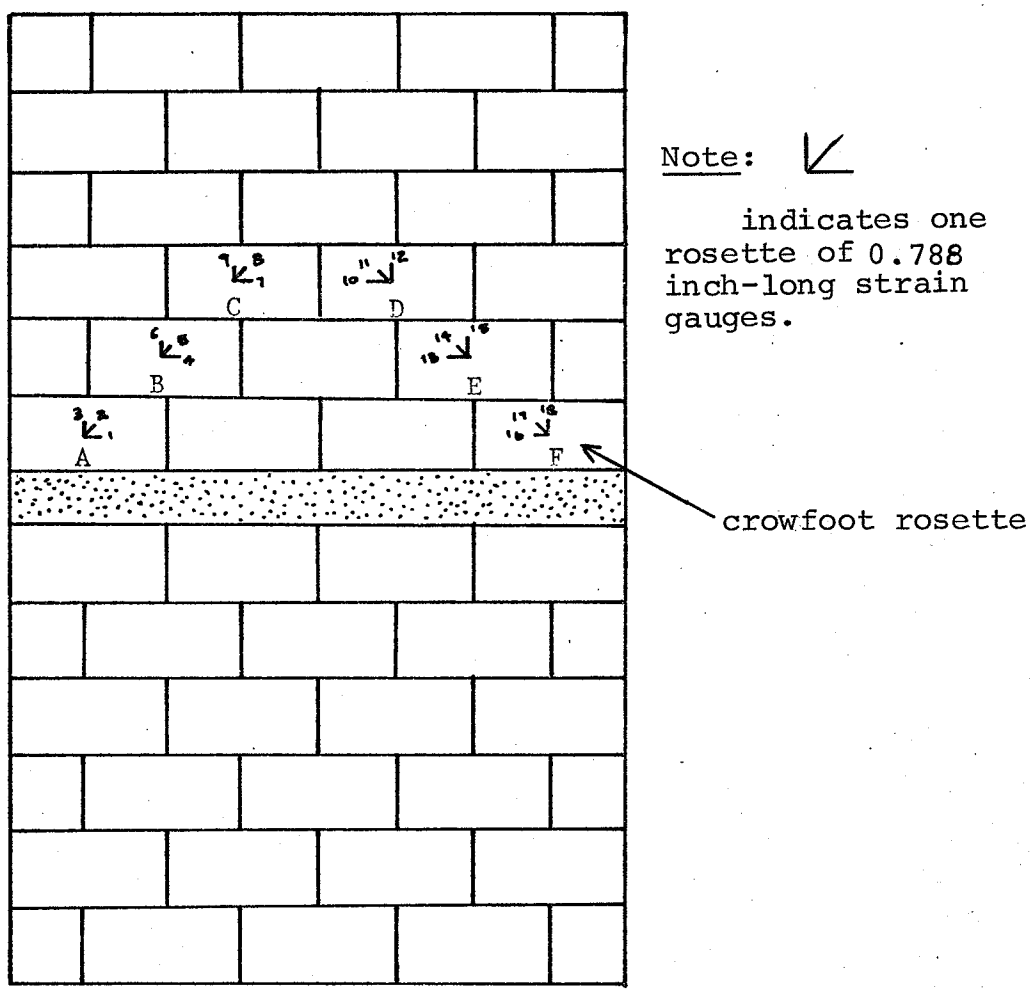


FIGURE 7-1. POSITION AND NUMBERING OF STRAIN GAUGES.

(Wall A_1 and B_3 only)

After the observation of the strains in the directions of the gauge was made, the following procedure was used to find the principal stresses.[4] A temporary horizontal ε-axis was drawn horizontally from any origin o', Figure 7-2, and the three measured strains $\varepsilon(\phi)$, $\varepsilon(\alpha + \phi)$, $\varepsilon(\alpha + \beta + \phi)$ laid off along it. Verticals were drawn through these points. Selecting any point D on the vertical through $\varepsilon(\alpha + \phi)$, lines DA, DC were drawn at angles α and β to the vertical at D as shown, to meet the other two verticals at A and C. The circle drawn through D, A, and C was the required circle. Its center F was determined by the intersection of the perpendicular bisectors of CD, DA. The points representing the three gauge directions were A, B, and C. The angle AFB, being twice the angle ADB at the circumference, was 2α, and BFC was 2β. Thus, A, B, C were at the required angular intervals round the circle, and had the required abscissas. The $\varepsilon\rho$ axis could now be drawn as OF, and the distances from O to the intersections with the circle gave ε_1, ε_2. The angle 2ϕ was the angle of FA below this axis.

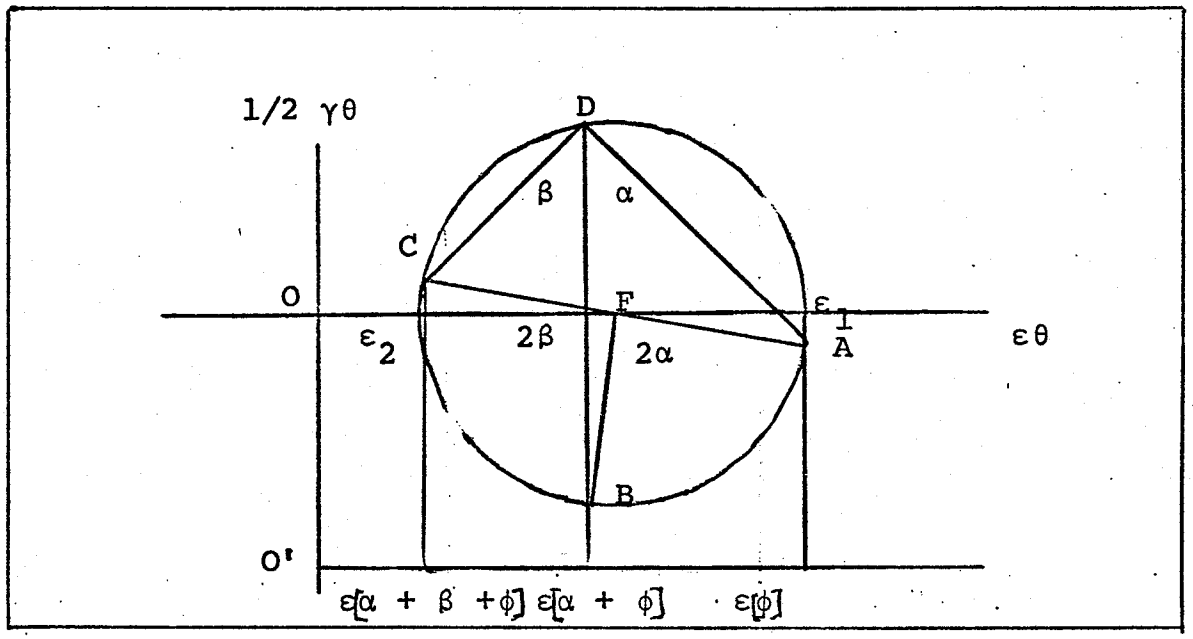


FIGURE 7-2. PRINCIPAL STRAINS.

For an active strain gauge with a second strain gauge for temperature compensation, the strains are found using the relation $\varepsilon = \frac{4}{k} \frac{\delta v}{2xV}$

where k = gauge factor = 2.04

δv = change in voltage

V = initial voltage, before loads are applied.

The results from the strain gauges converted to strains in inches per inch are tabulated in Appendix D.

An examination of these results indicated that the strain gauges were giving no systematic information about the strains in hollow blocks, for no particular orderly behaviour was observed in the results. No prediction can be made about the deformations within the wall from these results. The author believes that the deformations take place

at the joints when relatively low compressive loads exist and that when high vertical loads are present, the blocks behave in a complex manner. To illustrate the point, Wall A_1 is examined. With vertical stress 12.3 psi and with horizontal load of 4000 lb., the strain in the x-direction was -.00412 in/in and with a horizontal load of 6000 lb., the strain was -.000014. (The negative sign indicated compressive strain.) The strains in the x-direction for Wall A_1 are plotted in Figure 7-3.

The same type of discrepancies occurred with Wall B_3. From this investigation, it is believed that strain gauges will give little or no information about the stresses on masonry hollow walls since the mortar seems to behave like a spring. However, this investigation should be carried further, and in greater detail, with more strain gauges attached to the blocks. The results are discussed in more detail in Chapter IX.

It is interesting to note that the results shown in Figure 7-3 are typical of a "stick-slip" situation, in which frictional resistance builds up, and then is momentarily released. It would appear that major slippages occurred at about 5 kips and about 14 kips. Such discontinuous action has been observed in other bond phenomena in concrete structures.[5]

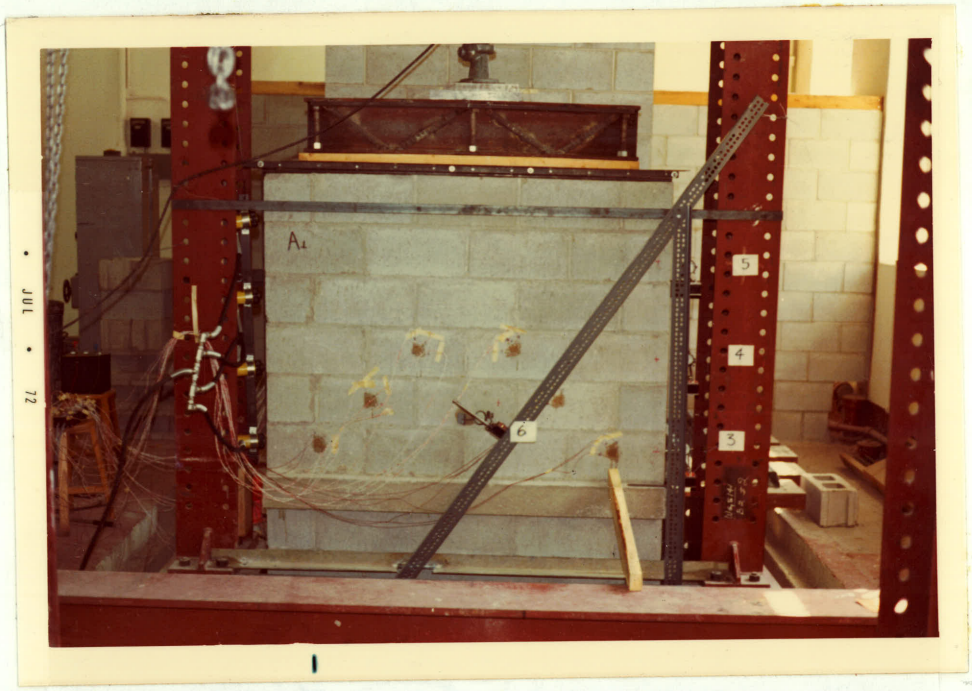


Photo 7-1. Wall A_1 with strain gauges.

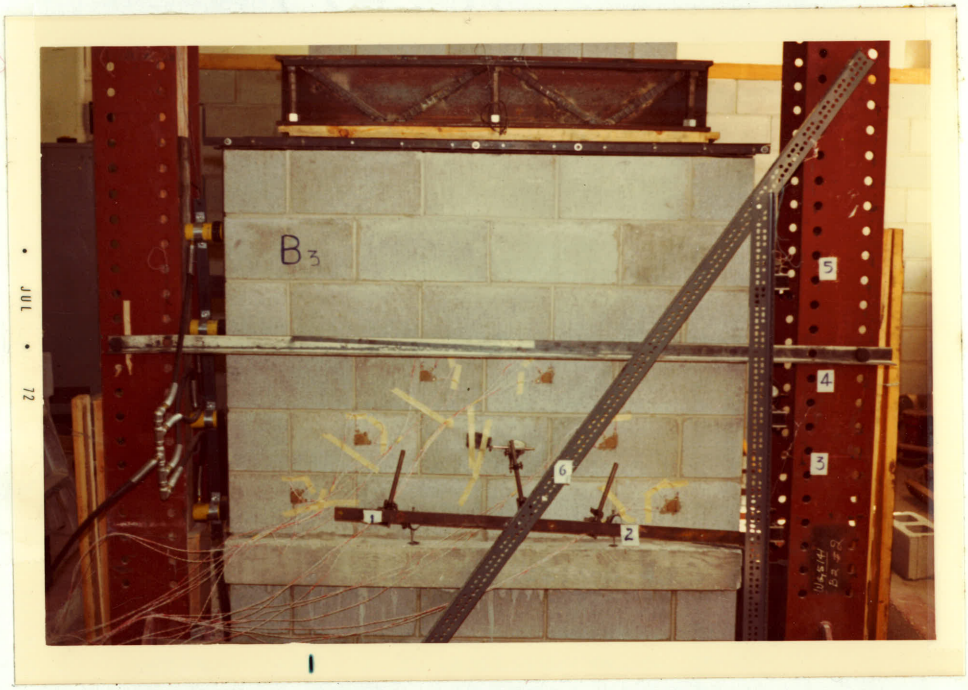


Photo 7-2. Wall B_3 with strain gauges and deflection measuring devices ready to be tested.

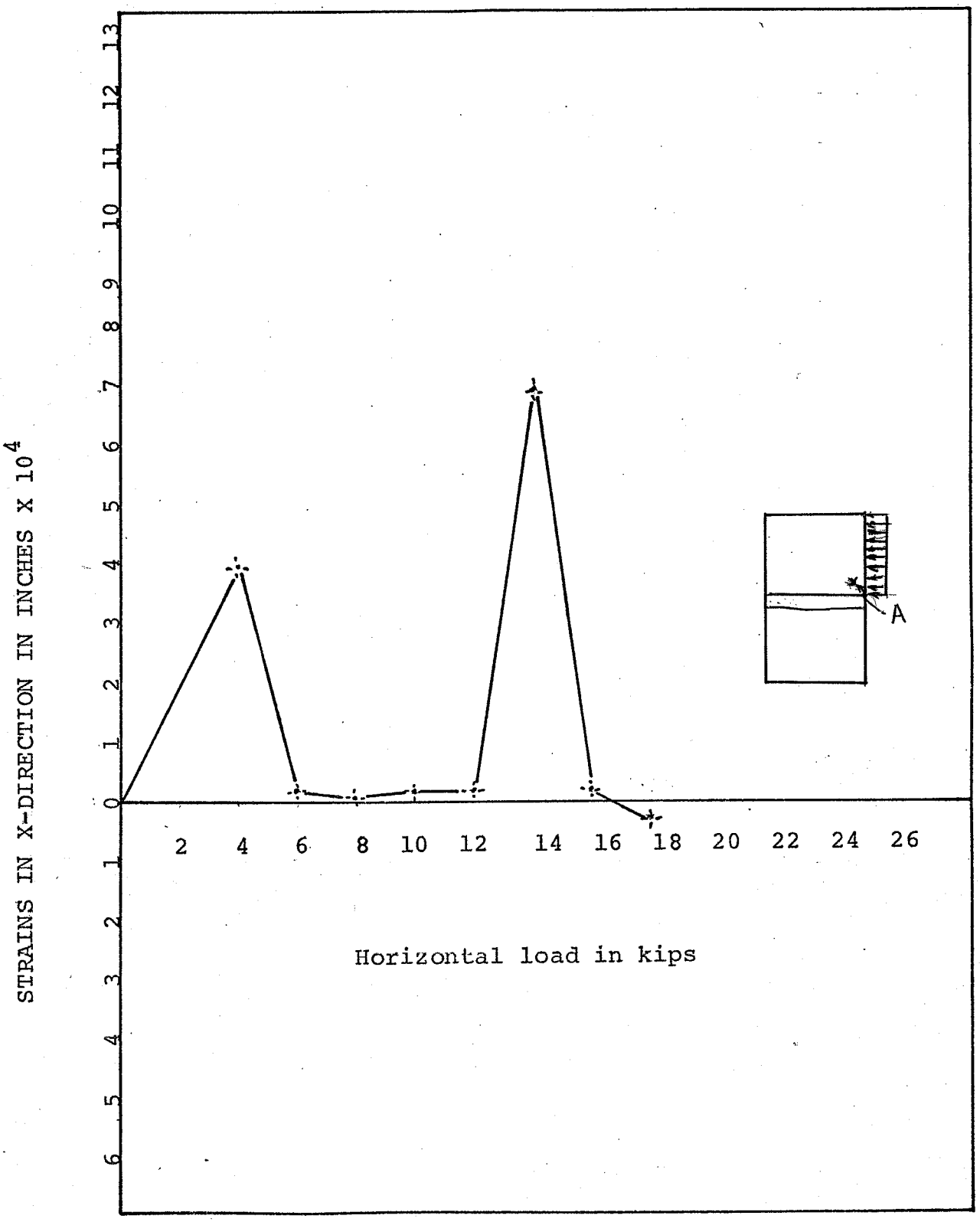


FIGURE 7-3. STRAINS IN THE X-DIRECTION, WALL A_1

CHAPTER VIII

Results of Tests

From tests carried out by the author the tensile strength of the blocks is found to be 102 psi. (Appendix B) If the wall is to fail because of diagonal tension the following stresses are to be expected on the slab-wall connection (Table 8-1). The values of σy in Table 8-1 are obtained from Table 4-1 to correspond to the stresses at various storey heights of an apartment building. The same vertical stresses were used in the experimental investigation. The values of V given in Table 8-1 are obtained using the relations

$$f_t = 100 = \sqrt{\frac{\sigma y^2}{4} + V^2} - \frac{\sigma y}{2} \quad (1) \qquad \text{and}$$

$V = Vo + \mu\sigma y$ (2) by substituting σy in (1) and solving for V. Then from (2), Vo is obtained.

This analysis shows that the shear strength of the mortar is important only when vertical loads are small. After to a vertical load of 149 psi or 5.34 k/ft the shear strength of the mortar is not utilized.

The shear stress to be resisted by the mortar for Walls A_5 and B_5 is shown on Table 8-1, (Vo) as -10.14 psi. This is a calculated value for the relation $V = Vo + \mu\sigma y$ with $\sigma y = 270$ and $\mu = 0.75$. The value of V is 192.35 obtained for the relation for maximum value (100 psi) tensile stress. The value of μ is taken as 0.75 in this work, this being the average of the values found in the literature. (0.72 + 00.85)

Deflection calculations in this study are not made, since the actual situation, because of coupling effect, will be completely different. The effect of the floor slab and the slab of the upper floor contribute nothing in resisting the lateral load in the experimental investigation as carried out in this study. However, in the actual situation, the slabs connect wall sections and the system will resist the lateral forces as a unit and it will also deflect as a unit.

TABLE 8-1. PREDICTED SHEAR CAPACITY

Wall	σy	Vo	$V=Vo+\mu\sigma y$	Predicted Type of Failure	Lateral Load at Failure
A_1	12.3	96.74	105.97	shear	11,000 lb
A_2	82.5	73.21	112.09	shear	21,100 lb
A_3	149	46.04	157.75	shear or diagonal tension	30,060 lb
A_4	210	18.56	176.06	diagonal tension	33,540 lb
A_5	270	-10.14	192.35	diagonal tension	36.643 lb
B_1	12.3	96.14	105.97	shear	10,350 lb
B_2	12.3	96.14	105.97	shear	10,350 lb
B_3	149	46.04	157.79	shear or tension	30,060 lb
B_4	210	18.5	176.06	diagonal tension	33,540 lb
B_5	270	-10.14	192.35	diagonal tension	36,643 lb
C_1	0	158	158	shear	30,050 lb
C_2	12.3	96	105.97	shear	51,000 lb

1. Test of Group A

Five walls were tested. The vertical loads are given in Table 8-2 which also gives the horizontal loads at failure. The mortar strength in each case was 48.5 psi.

TABLE 8-2. LOADING OF GROUP A AND SUMMARY OF LOADS AT FAILURE

Wall	Total Vertical Load in kips	Vertical Stress in psi	Total Horizontal Load in lbs. at failure	Shear Stress due to horizontal load in psi
A_1	.44 x 5.3 = 2.33	12.3	14,400	75.5 psi
A_2	2.9 x 5.3 = 15.7	82.5	23,400	123 psi
A_3	5.3 x 5.3 = 28.3	149	32,400	170 psi
A_4	7.5 x 5.3 = 39.95	210	39,600	207 psi
A_5	9.68 x 5.3 - 51.4	270	41,400	218 psi

The vertical loads represent:

Wall 1 - load due to weight of blocks only

Wall 2 - load (dead + live) at 12th floor

Wall 3 - load (dead + live) at 11th floor

Wall 4 - load (dead + live) at 10th floor

Wall 5 - load (dead + live) at 9th floor

On the following pages, the behaviour of each test specimen is analyzed. The test results are discussed in the following order:

a) stresses - strains

b) loads at failure

c) deflections

In every stage, a comparison with the results from the analytical and theoretical work is made.

Wall A_1

a) Stresses.

Wall A_1 failed in shear at the connection between slab and wall at a horizontal load of 14,400 lb. This load gives a shear stress at the joint such that

$$14{,}400 = 190.5\ V$$

$$V = \frac{14{,}400\text{lb}}{190.5\text{in}^2} = 75.5 \text{ psi}$$

The contribution of the mortar is then

$$Vo = V - \mu\sigma y = 75.5 - .75 \times 12.3 = 66.3 \text{ psi}$$

The mortar had an average shear strength of 48.5 ≈ 50 psi, which is much less than the stress at failure. The expected shearing stress at failure from the analytical investigation was 105.00 psi. The average shearing stress obtained from finite element analysis method is 224 psi. (See Table 6-1.)

b) Loads at Failure

Photo 8-1 shows the wall after failure in shear. The load at failure was 14,400 lb. which is 3,400 lb. more than the calculated 11,000 lb. in the analytical work, based on the theory of Chapter III.

c) Deflections

The deflections as measured and found using finite element analysis are plotted for selected loads in Figure 8-1. The deflections for the other loads are given in Table 8-5. The values of the deflections obtained using finite element

analysis are given in Table 8-4. On the same page, the loading of the nodes used in the analysis is given. The load-deflection curve for a point 5" above the slab in the free of stresses side is given in Figure 8-2.

TABLE 8-3. LOADING FOR FINITE ELEMENT ANALYSIS

WALL A_1

NODE	PX(KIPS)	PY(KIPS)
1	2.3900	0.2590
2	2.3900	0.0000
3	2.3900	0.0000
4	2.3900	0.0000
5	2.3900	0.0000
6	2.3900	0.0000
7	2.3900	0.0000
15	0.0000	0.2590
29	0.0000	0.2590
43	0.0000	0.2590
57	0.0000	0.2590
71	0.0000	0.2590
85	0.0000	0.2590
99	0.0000	0.2590
113	0.0000	0.2590

TABLE 8-4. DEFLECTIONS FROM FINITE ELEMENT ANALYSIS

WALL A_1

Nodal Displacements (in)

NODE	X	Y
1	0.0045	-0.0065
2	0.0041	-0.0064
3	0.0035	-0.0061
4	0.0029	-0.0054
5	0.0022	-0.0044
6	0.0018	-0.0027
7	0.0019	-0.0000

TABLE 8-5. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR ROTATION AND HORIZONTAL FORCES, CORRECTED FOR ROLLING RESISTANCE.

WALL A1

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6
3,600	0.0000	0.0000	0.0000	0.0000
5,400	0.0115	0.0040	0.0059	0.0072
7,800	0.0106	0.0101	0.0143	0.0146
9,000	0.0178	0.0173	0.0217	0.0200
10,800	0.0188	0.0218	0.0539	0.0382
12,600	0.0204	0.0327	0.0377	0.0330

NOTE: DEFLECTION IS TAKEN TO BE ZERO AT 3,600 LBS. HORIZONTAL LOAD

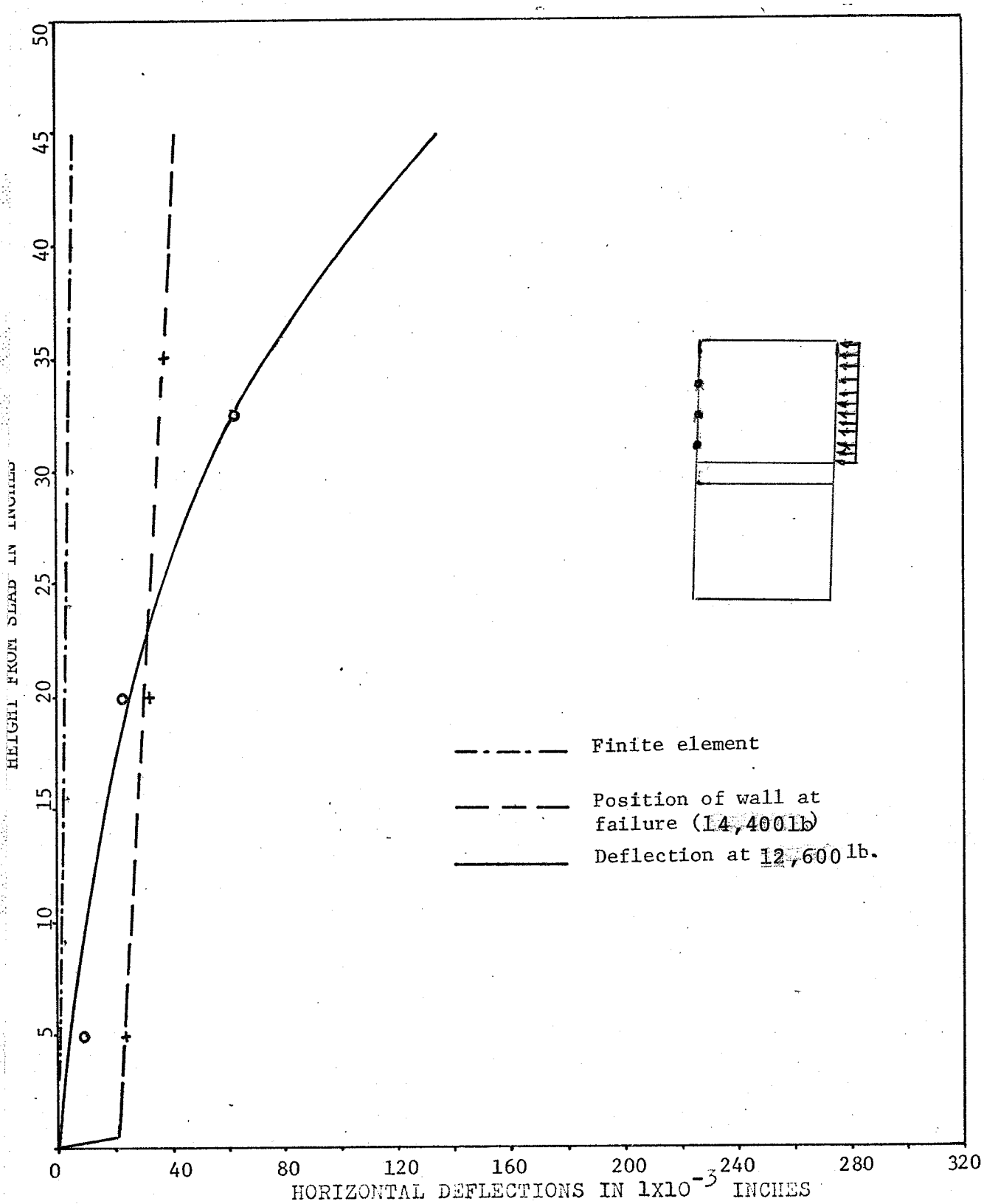


FIGURE 8-1. DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL A_1

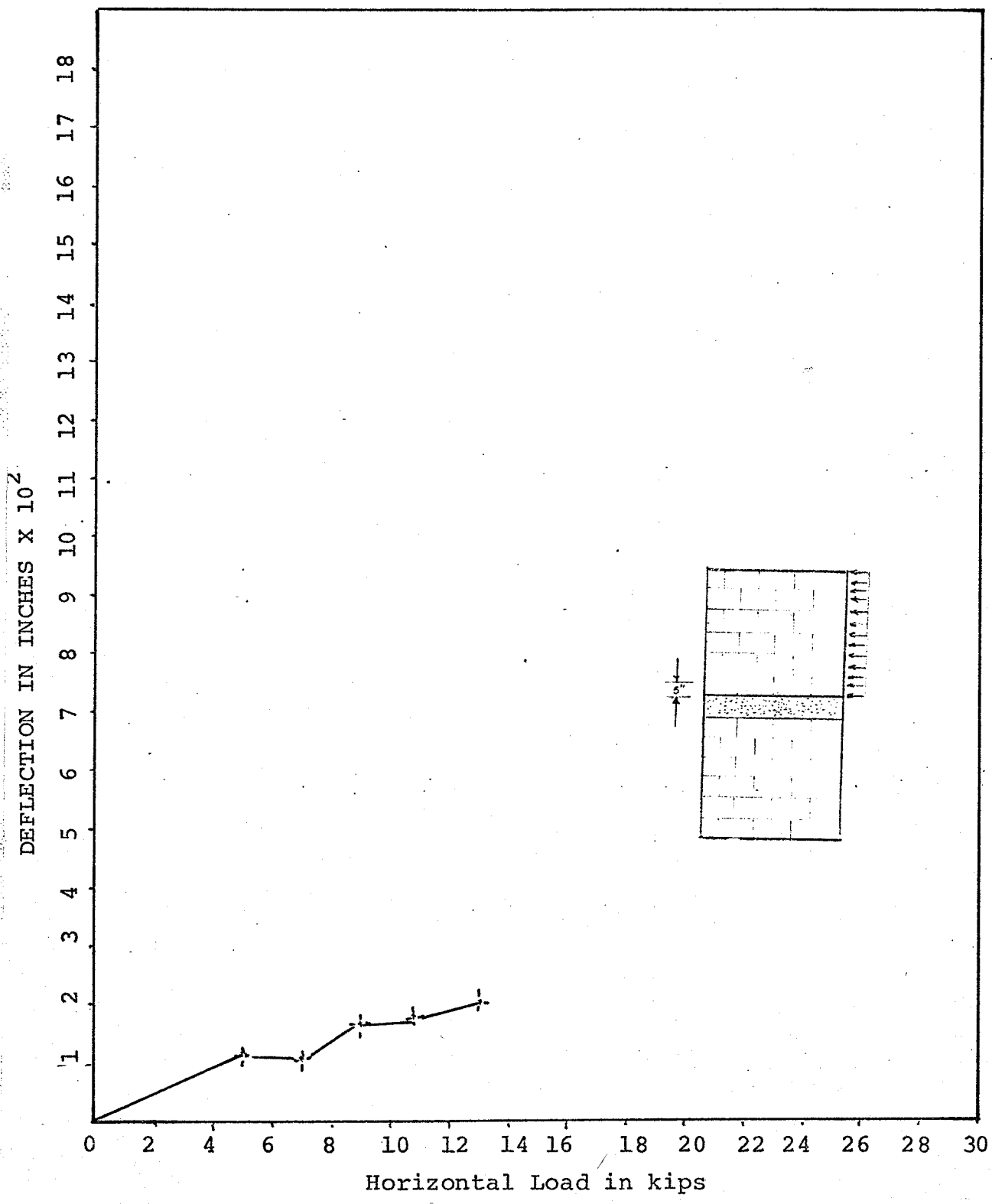


FIGURE 8-2. LOAD-DEFLECTION CURVE FOR A POINT 5" ABOVE THE SLAB ON THE FREE OF STRESSES SIDE. WALL A_1

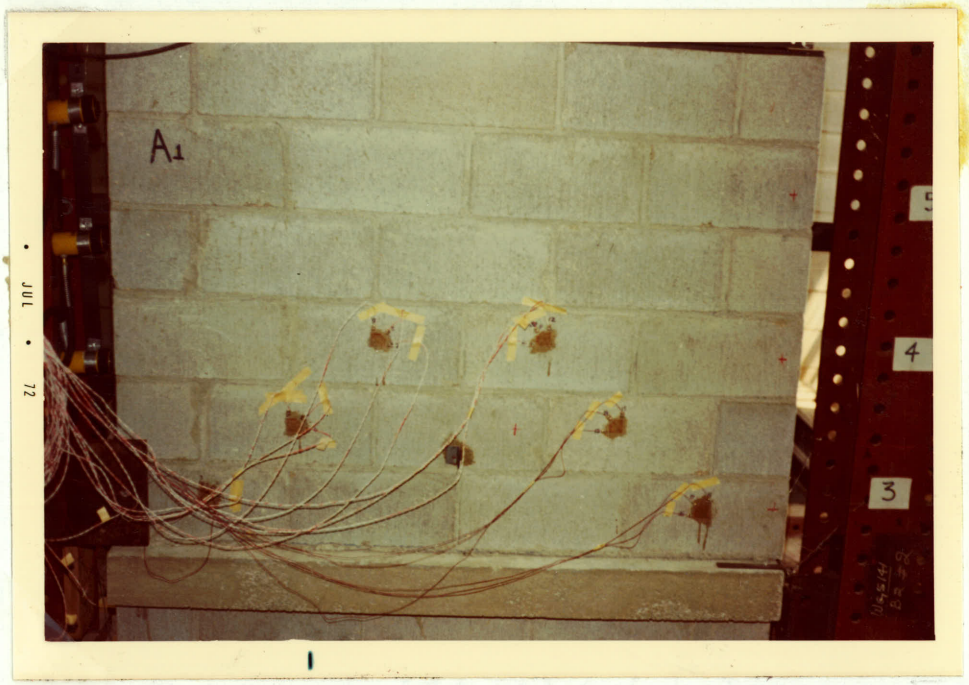


Photo 8-1. Wall A_1 after failure in shear.

NOTE: Shifting of the wall to the right.

Wall A_2

a) Stress

The failure of Wall A_2 was in shear, as expected from the analytical consideration. The shear stress at failure was

$$\frac{23{,}400\,\text{lb.}}{190.5\,\text{in}^2} = 123.0 \text{ psi}$$

and the friction contributed $0.75 \times 82.5 = 62$ psi, i.e. the mortar contribution appears to be $123.0 - 62 = 61.0$ psi, which is close to the shear strength of mortar as found for Wall A_1. The expected shearing stress at failure was 135.0 psi, and the average shearing stress from finite element analysis was 182 psi.

Photo 8-2 shows the wall ready to be tested. Photo 8-3 shows the failure of Wall A_2.

b) Loads at Failure.

The wall failed at a horizontal load of 23,400 lb. which is 2,300 lb. more than the calculated 21,100 lb. in Chapter II.

c) Deflections

The measured deflections are given in Table 8-8 and plotted for selected loads in Figure 8-3. On the same figure, the deflections obtained from finite element analysis are plotted for comparison. The deflections of the finite element analysis and the loading are given in Tables 8-6 and 8-7. Figure 8-4 shows load deflection relation for a point 5" above the slab in the free of stresses side of the wall.

TABLE 8-6. LOADING FOR FINITE ELEMENT ANALYSIS

WALL A_2

NODE	PX(KIPS)	PY(KIPS)
1	3.7100	1.7900
2	3.7100	0.0000
3	3.7100	0.0000
4	3.7100	0.0000
5	3.7100	0.0000
6	3.7100	0.0000
7	3.7100	0.0000
15	0.0000	1.7900
29	0.0000	1.7900
43	0.0000	1.7900
57	0.0000	1.7900
71	0.0000	1.7900
85	0.0000	1.7900
99	0.0000	1.7900
113	0.0000	1.7900

TABLE 8-7. DEFLECTIONS FROM FINITE ELEMENT ANALYSIS.

WALL A_2

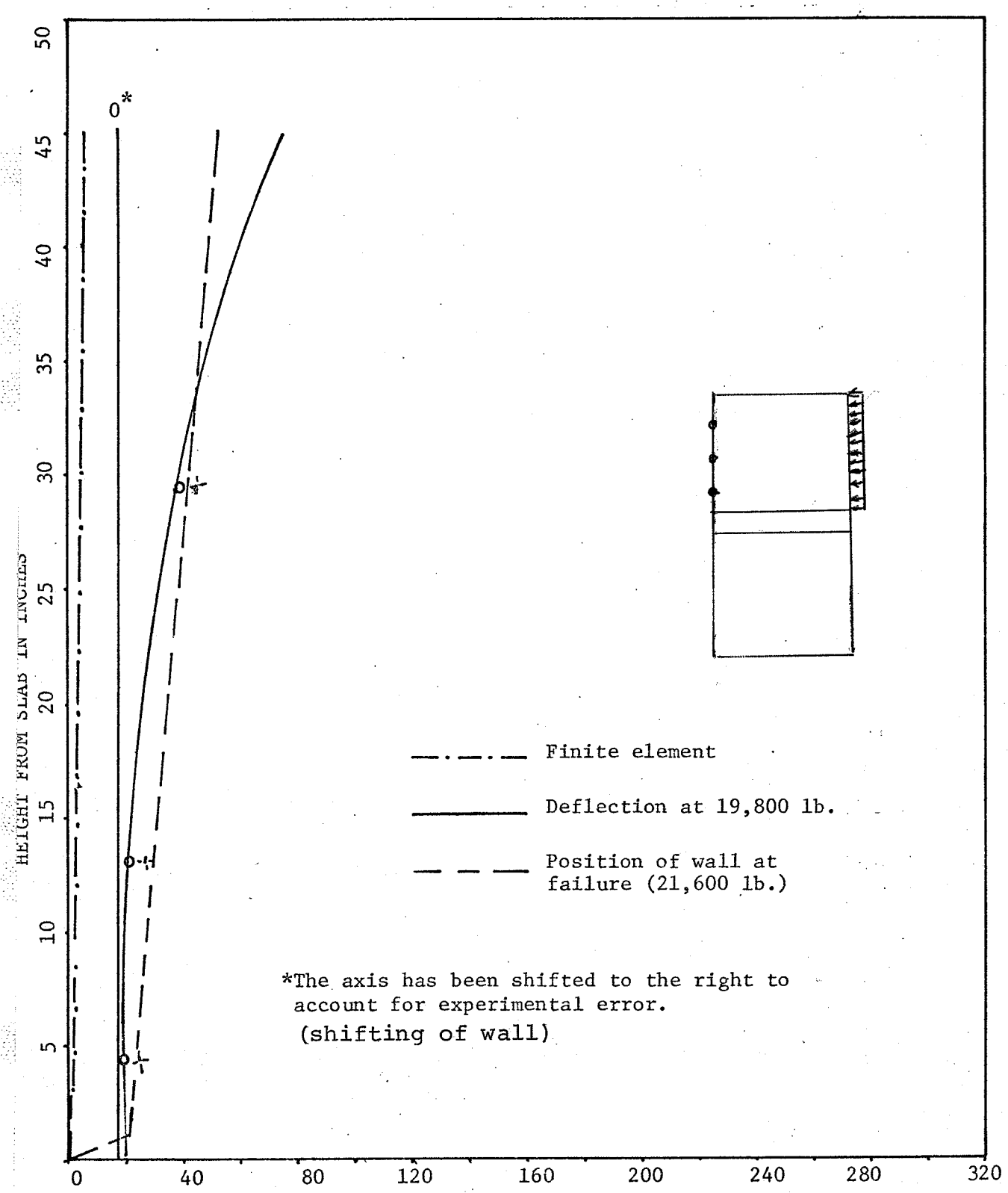
Nodal Displacements (in)

NODE	X	Y
1	0.0049	-0.0064
2	0.0044	-0.0063
3	0.0038	-0.0060
4	0.0032	-0.0054
5	0.0026	-0.0043
6	0.0021	-0.0026
7	0.0022	-0.0000

TABLE 8-8. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR ROTATION AND HORIZONTAL FORCES CORREDTED FOR ROLLING RESISTANCE.

WALL A2

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6
3,600	0.0000	0.0000	0.0000	0.0000
5,400	0.0029	0.0069	0.0103	0.0071
7,800	0.0237	0.0073	0.0121	0.0117
9,000	0.0040	0.0065	0.0108	0.0116
10,800	0.0090	0.0102	0.0120	0.0215
12,600	0.0111	0.0135	0.0132	0.0235
14,400	0.0138	0.0189	0.0141	0.0361
16,200	0.0124	0.0197	0.0182	0.0382
18,000	0.0191	0.0211	0.0259	0.0375
19,800	0.0218	0.0221	0.0391	0.0340
21,600	0.0228	0.0225	0.0548	0.0336



HORIZONTAL DEFLECTIONS IN $1X10^{-3}$ INCHES

FIGURE 8-3. DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL A_2

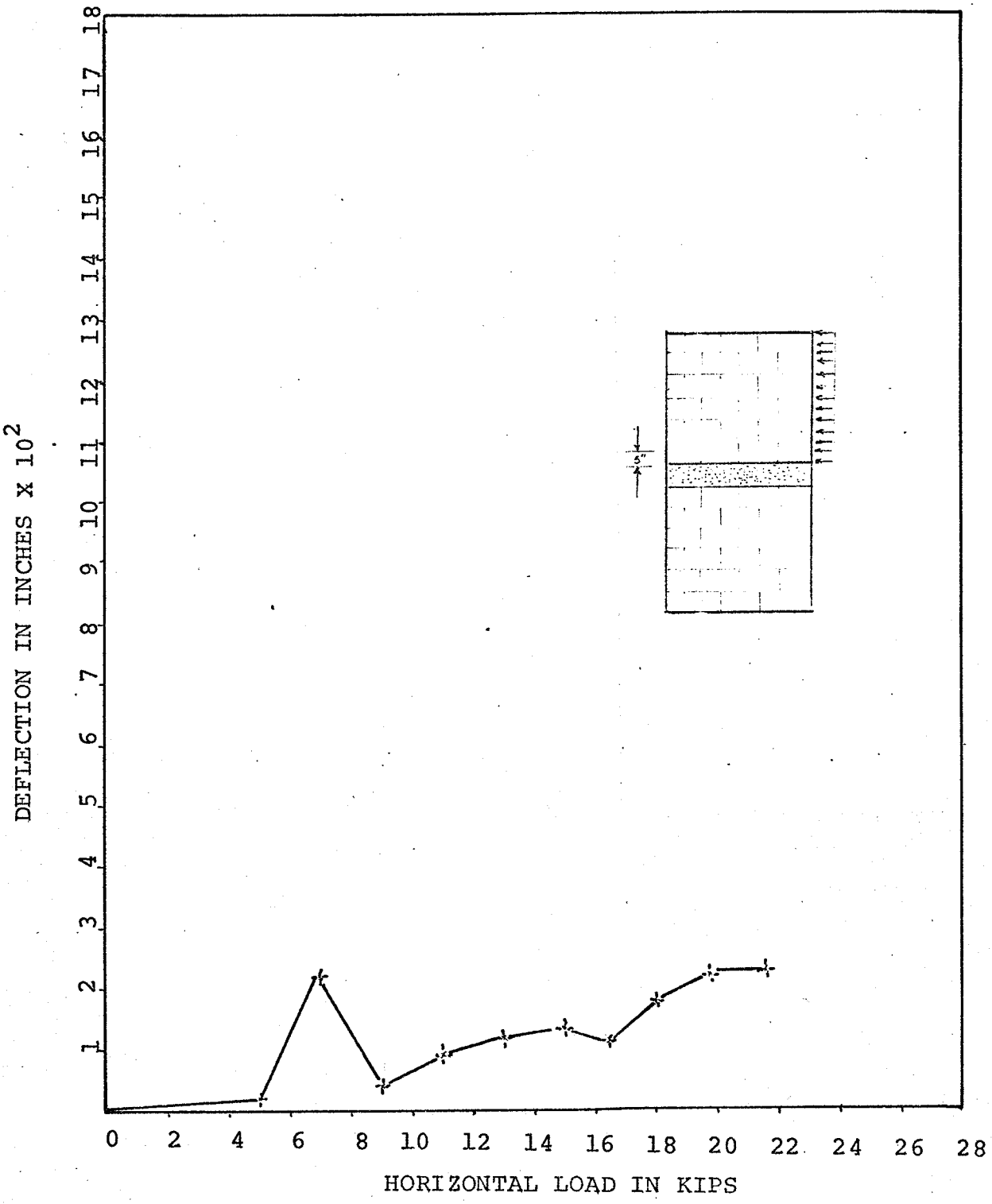


FIGURE 8-4. LOAD-DEFLECTION CURVE FOR A POINT 5" ABOVE THE SLAB ON THE FREE OF STRESSES SIDE. WALL A_2

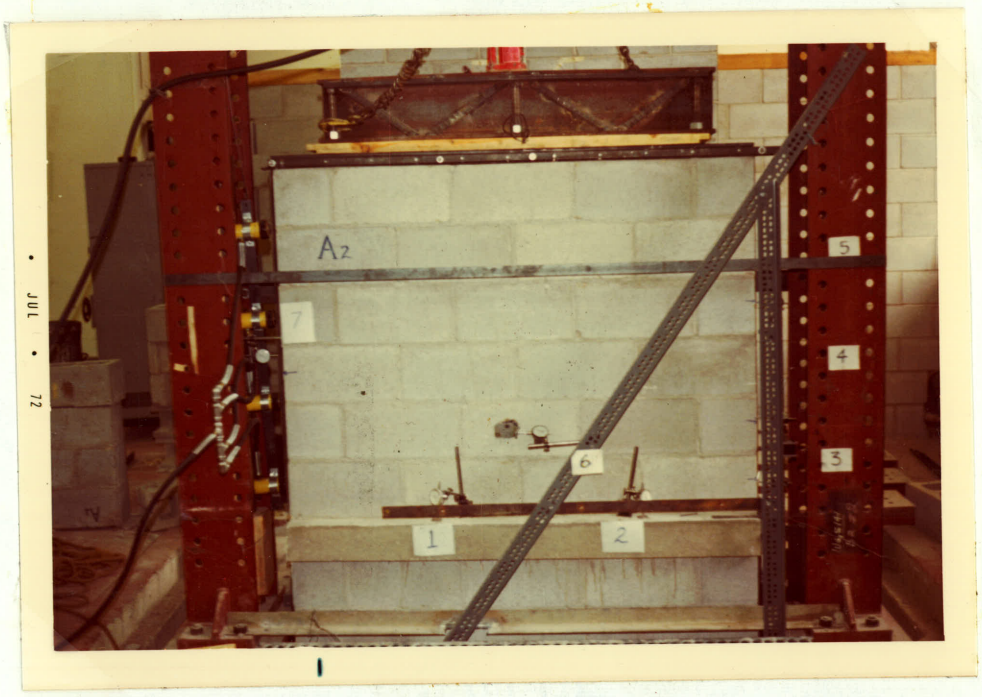
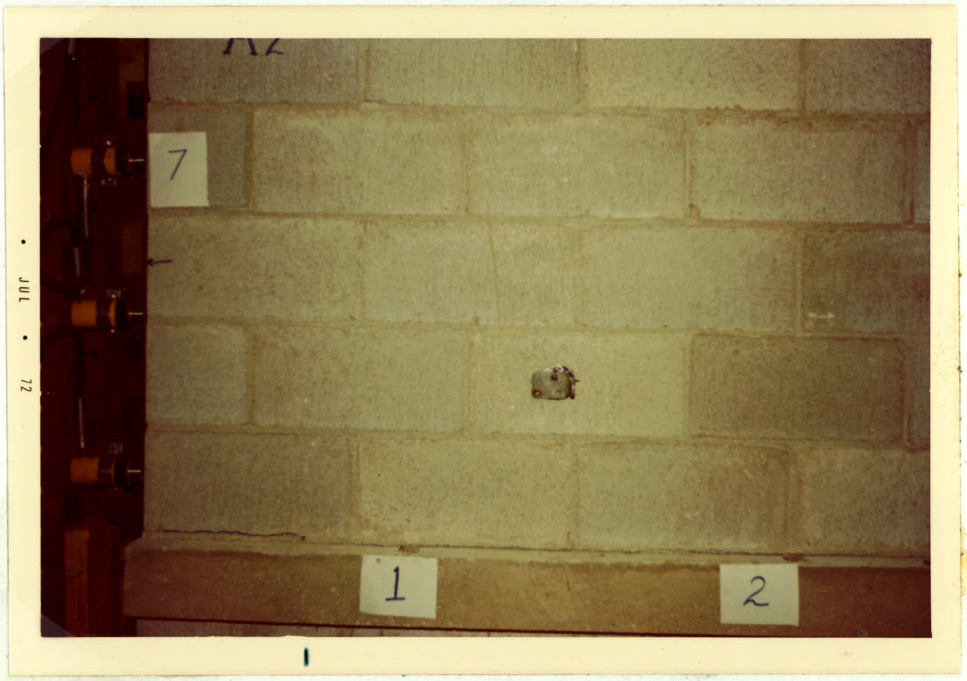


Photo 8-2. Wall A_2 placed on testing apparatus.



Note shear failure.

Photo 8-3. Shear failure of Wall A_2.

Wall A_3

a) Stresses

The mode of failure for this test was expected to be shear or due to diagonal tension. The wall failed in shear at 37,400 lb. The shear stress was

$$\frac{32,400 \text{lb.}}{190.5 \text{in}^2} = 170 \text{ psi}$$

The vertical stress was 149 psi which gives a frictional contribution to the shear resisting ability of the wall of 149 x .75 = 112 psi. The mortar contribution was

$$190 - 112 = 58 \text{ psi}$$

The calculated shearing stress was 157.00 psi and the average shearing stress from finite element analysis was 173 psi.

b) Load at Failure

The wall failed at 32,400 lb. which is 2,340 lb. more than the expected load of 30,060 lb.

c) Deflections

The corrected-for-rotation deflection measurements are given in Table 8-11 and plotted in Figure 8-5. In the same figure the deflections as obtained from finite element analysis are plotted. Tables 8-9 and 8-10 give the loading of the nodes and their deflections from the finite element analysis method. Figure 8-6 shows the load deflections curve for a point 5" above the slab on the unloaded side of the wall.

TABLE 8-9. LOADING FOR FINITE ELEMENT ANALYSIS

WALL A_3

NODE	PX(KIPS)	PY(KIPS)
1	5.1500	3.1400
2	5.1500	0.0000
3	5.1500	0.0000
4	5.1500	0.0000
5	5.1500	0.0000
6	5.1500	0.0000
7	5.1500	0.0000
15	0.0000	3.1400
29	0.0000	3.1400
43	0.0000	3.1400
57	0.0000	3.1400
71	0.0000	3.1400
85	0.0000	3.1400
99	0.0000	3.1400
113	0.0000	3.1400

TABLE 8-10. DEFLECTIONS FROM FINITE ELEMENT ANALYSIS.

WALL A_3

Nodal Displacements (in)

NODE	X	Y
1	0.0053	-0.0064
2	0.0048	-0.0063
3	0.0042	-0.0060
4	0.0036	-0.0054
5	0.0030	-0.0043
6	0.0025	-0.0026
7	0.0025	-0.0000

TABLE 8-11. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR ROTATION AND HORIZONTAL FORCES CORRECTED FOR ROLLING RESISTANCE

WALL A3

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6
3,600	0.0000	0.0000	0.0000	0.0000
5,400	0.0053	0.0058	0.0114	0.0075
7,800	0.0061	0.0076	0.0117	0.0124
9,000	0.0047	0.0245	0.0220	0.0215
10,800	0.0043	0.0257	0.0402	0.0339
12,600	0.0031	0.0318	0.0516	0.0398
14,400	0.0057	0.0504	0.0821	0.0540
16,200	0.0081	0.0630	0.1089	0.0640
18,000	0.0063	0.0655	0.0849	0.0728
19,800	0.0057	0.0730	0.0022	0.0787
21,600	0.0073	0.0379	0.0049	0.0607
23,400	0.0077	0.0261	0.0140	0.0483
25,500	0.0088	0.0232	0.0084	0.0532
27,000	0.0094	0.0233	0.0289	0.0464
28,800	0.0100	0.0317	0.0689	0.0695
31,600	0.1365	0.1621	0.1759	0.1053
32,400	0.1795	0.2577	0.2589	0.3251
34,200	0.2235	0.3157	0.3249	0.4111

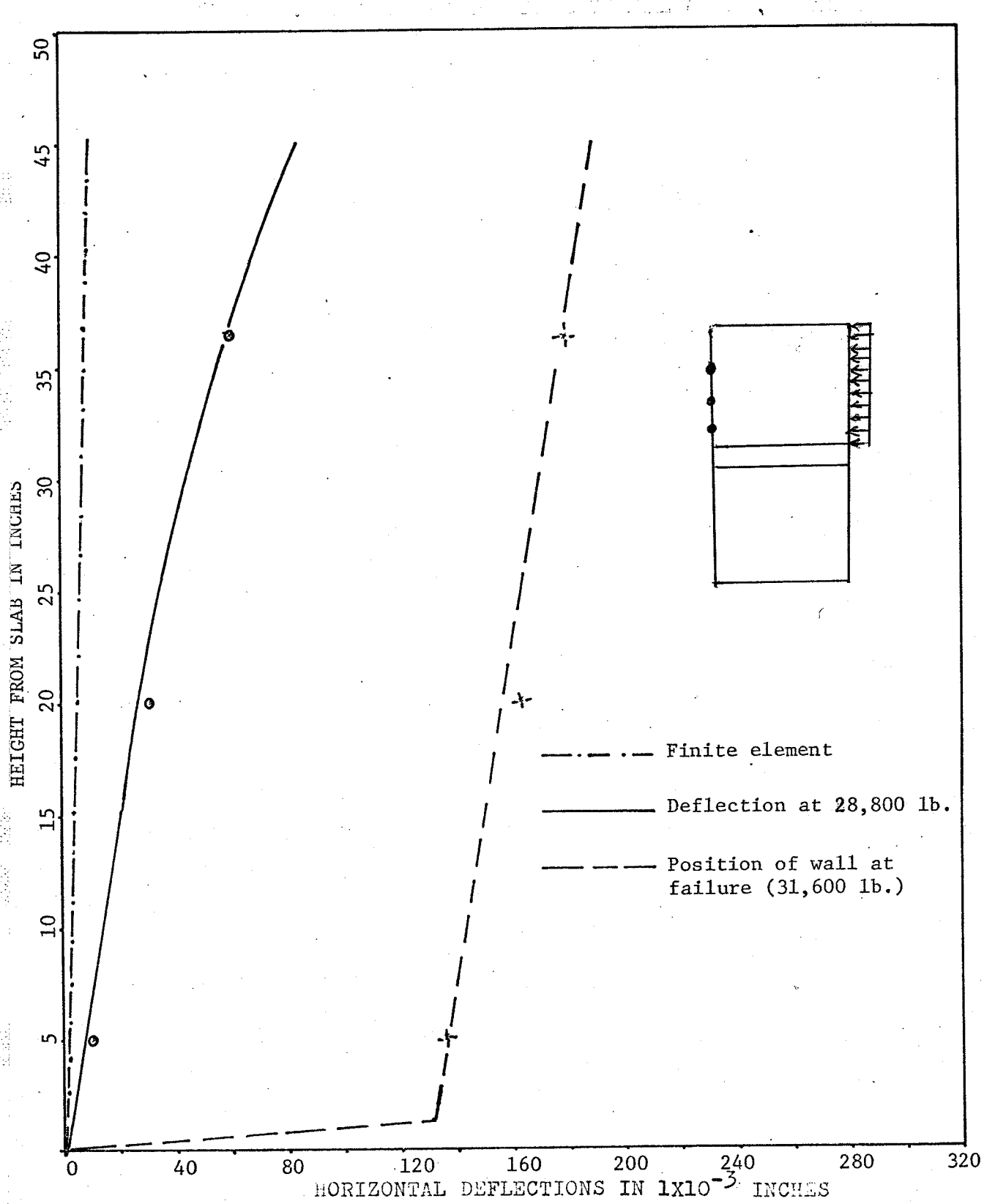


FIGURE 8-5. DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL A

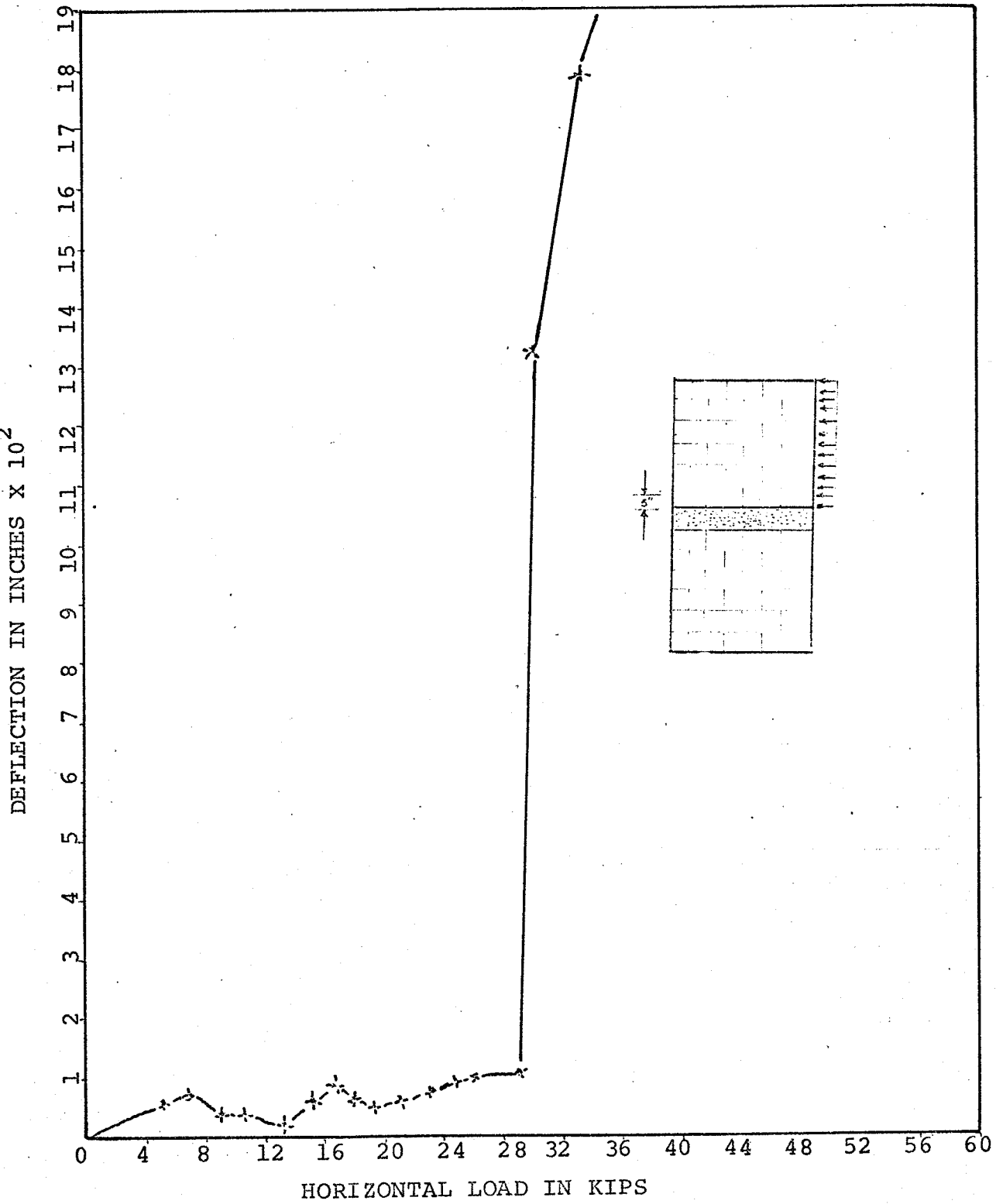


FIGURE 8-6. LOAD-DEFLECTION CURVE FOR A POINT 5" ABOVE THE SLAB ON THE FREE OF STRESSES SIDE. WALL A_3

Wall A_4

a) Stresses

The wall failed in the manner shown in Photo 8-4. The joint remained unharmed from the loading. The shearing stresses at the joint were

$$\frac{39,600 \text{lb}}{190.5 \text{in}^2} = 208 \text{ psi}$$

The wall was expected to fail when a shearing stress of 176.00 psi in the joint was reached. The average shearing stress from this finite element analysis was found to be 155 psi.

b) Loads at Failure

Failure occurred at 39,600 and the wall was still able to carry an additional load of 46,800 - 39,600 = 7,200 lb. i.e. 1.18 of the load at failure.

Looking at Photo 8-4, it is observed that a crack occurred in the first 1/4 of the first blocks. Photo 8-5 shows that cracks progressed into the second block.

Vertical load 39.95^k was applied on the wall for which the expected load at failure was 33,540 lb. and failure was expected to be due to diagonal tension. The wall failed at 39,600 lb. i.e. 6000 lb. more than expected. Photo 8-7 shows the wall after failure. The wall was loaded further and at 46,800 lb.it failed - exploding.

c) Deflections

The deflections are given on Tables 8-13 and 8-14.

Table 8-13 gives the deflections from the finite element analysis and 8-14, the experimentally found ones.

Table 8-12 gives the load/node in the finite element analysis.

The deflections plotted on Figure 8-9 show load-deflection relation for a point 5" above the slab on the free of stresses side of Wall A_4.

TABLE 8-12. LOADING FOR FINITE ELEMENT ANALYSIS

WALL A_4

NODE	PX(KIPS)	PY(KIPS)
1	7.3500	4.4800
2	7.3500	0.0000
3	7.3500	0.0000
4	7.3500	0.0000
5	7.3500	0.0000
6	7.3500	0.0000
7	7.3500	0.0000
15	0.0000	4.4800
29	0.0000	4.4800
43	0.0000	4.4800
57	0.0000	4.4800
71	0.0000	4.4800
85	0.0000	4.4800
99	0.0000	4.4800
113	0.0000	4.4800

TABLE 8-13. DEFLECTIONS FROM FINITE ELEMENT ANALYSIS.

WALL A_4

Nodal Displacements (in)

NODE	X	Y
1	0.0060	-0.0063
2	0.0055	-0.0063
3	0.0049	-0.0060
4	0.0042	-0.0054
5	0.0035	-0.0043
6	0.0030	-0.0027
7	0.0030	-0.0000

TABLE 8-14. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR ROTATION AND HORIZONTAL FORCES CORRECTED FOR ROLLING RESISTANCE

WALL A4

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6
3,600	0.0000	0.0000	0.0000	0.0000
5,400	0.0002	0.0004	0.0013	0.0004
7,800	0.0085	0.0106	0.0117	0.0050
9,000	0.0184	0.0226	0.0356	0.0168
10,800	0.0187	0.0176	0.0385	0.0254
12,600	0.0232	0.0192	0.0407	0.0317
14,400	0.0239	0.0194	0.0432	0.0388
16,200	0.0322	0.0277	0.0497	0.0435
18,000	0.0308	0.0235	0.0374	0.0426
19,800	0.0359	0.0329	0.0551	0.0499
21,600	0.0378	0.0355	0.0584	0.0536
23,400	0.0406	0.0371	0.0603	0.0583
25,500	0.0437	0.0394	0.0652	0.0614
27,000	0.0453	0.0453	0.0670	0.0646
28,800	0.0509	0.0503	0.0729	0.0688
31,600	0.0547	0.0539	0.0761	0.0735
32,400	0.0611	0.0609	0.0806	0.0763
34,200	0.0659	0.0657	0.0858	0.0839
36,000	0.0749	0.0756	0.0956	0.0918
37,800	0.0815	0.0815	0.1024	0.0970
39,800	0.0984	0.0934	0.1232	0.1128
41,400	0.1251	0.1335	0.1483	0.1302
43,200	0.1387	0.1587	0.1727	0.1535
45,000	0.1554	0.1619	0.1902	0.1659

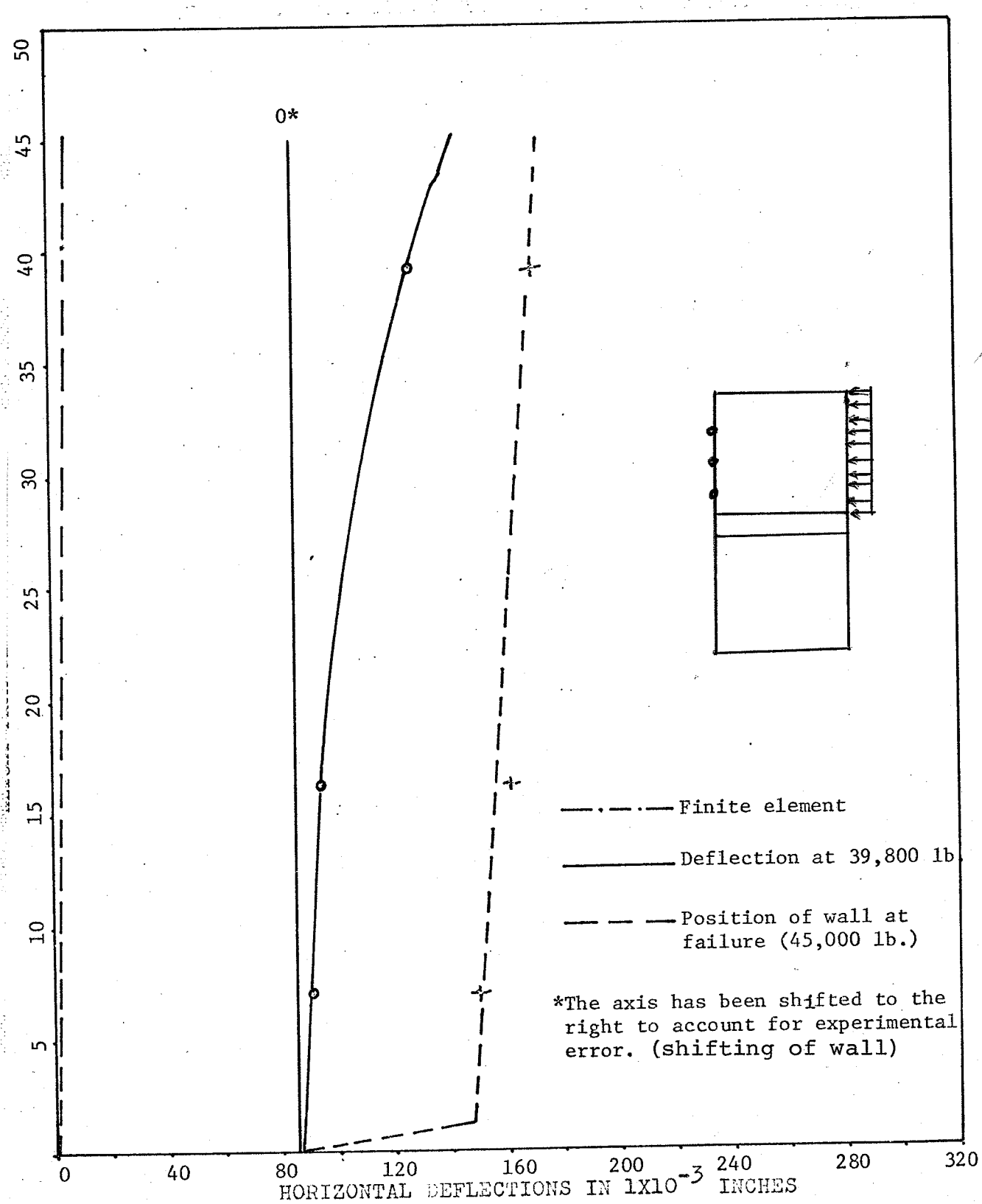


FIGURE 8-7. DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL A_4

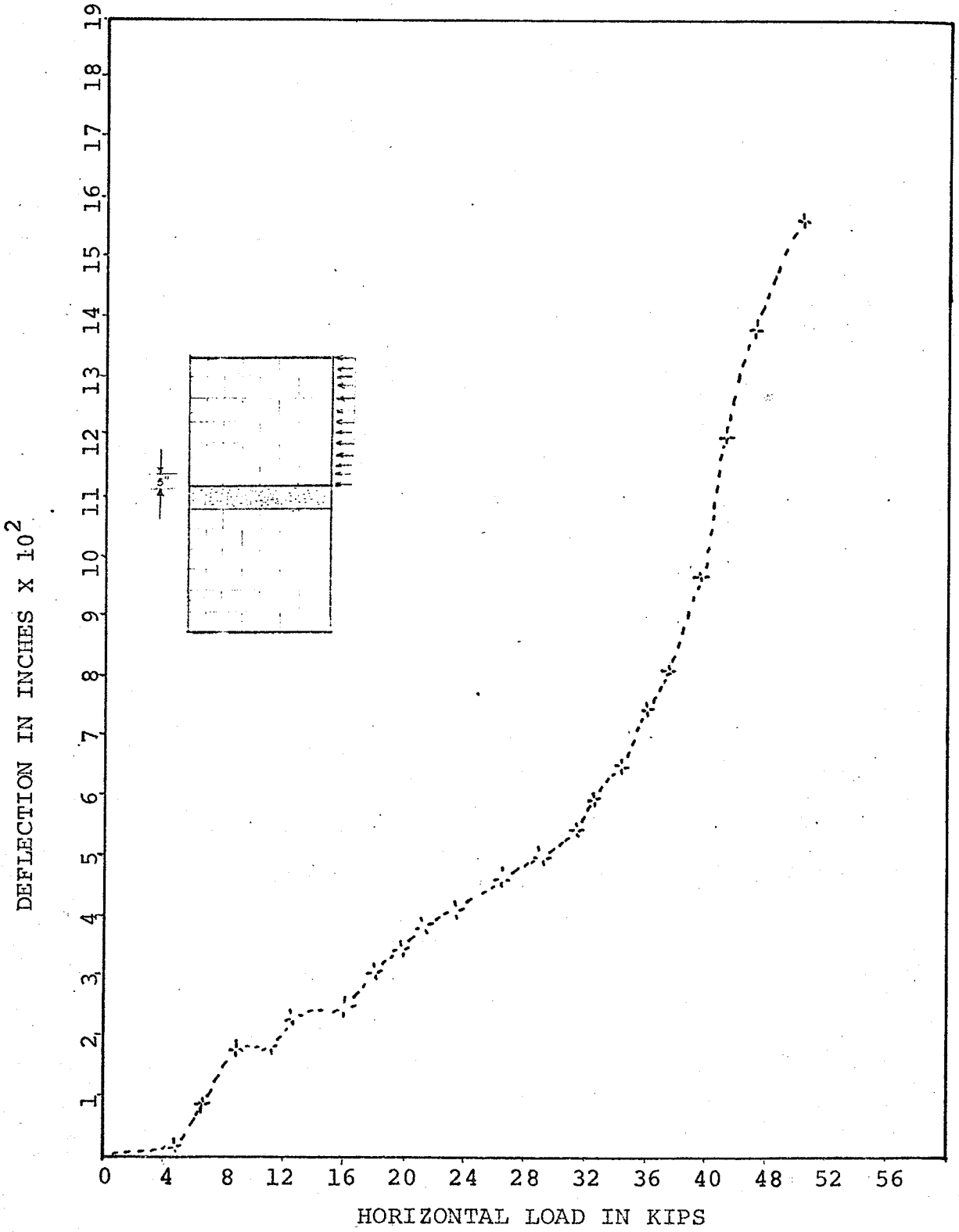


FIGURE 8-8. LOAD-DEFLECTION CURVE FOR A POINT 5" ABOVE THE SLAB ON THE FREE OF STRESSES SIDE. WALL A_4

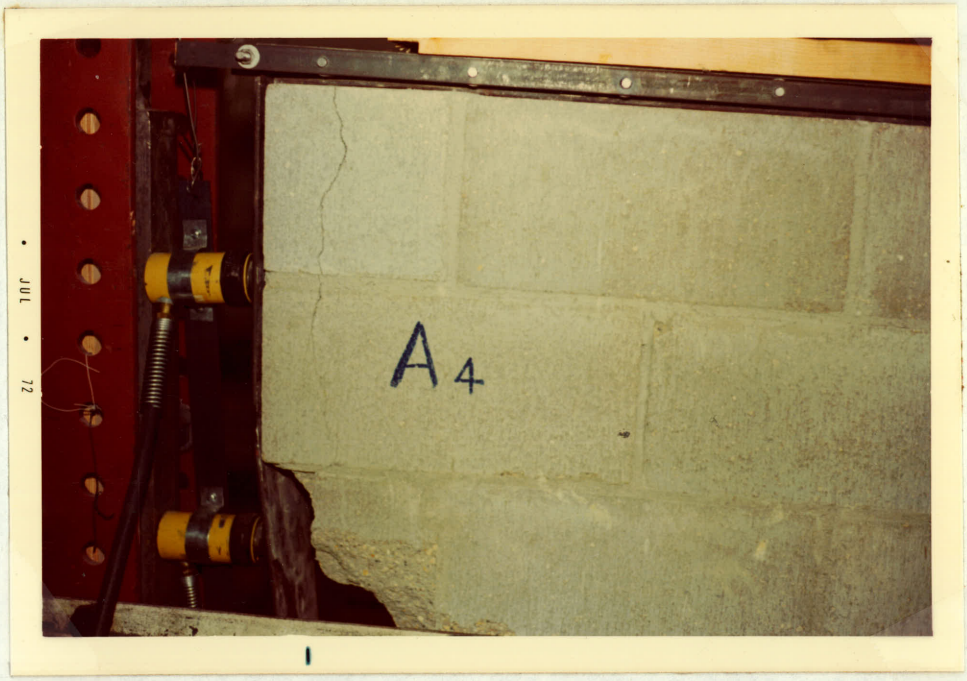


Photo 8-4. Vertical crack on Wall A_4.

Photo 8-5. Wall A_4 after explosive failure.

Photo 8-6 View of Wall A_4 showing severe cracking.

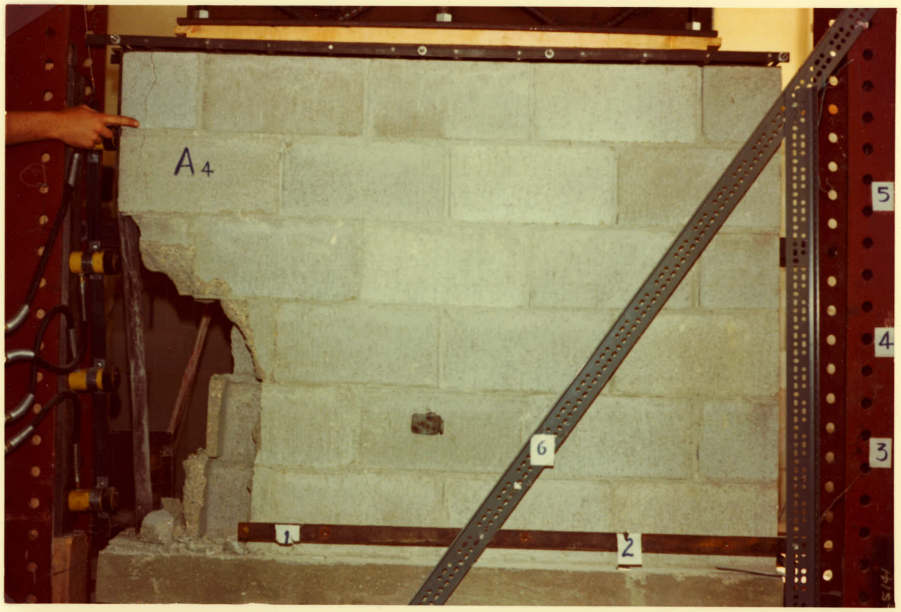


Photo 8-7. Final condition of Wall A_4 after loaded to 46.8^k horizontally.

Wall A_5

a) Stresses

Wall A_5 failed in a similar manner to Wall A_4. The shearing stress at failure was

$$\frac{41,400 lb}{190.5 in^2} = 218 \text{ psi}$$

The vertical stress was

$$\frac{51.4 lb}{190.5 in^2} \times 1000 = 270 \text{ psi}$$

The mortar contribution in resisting the 218 psi shearing stress was 218 - 270 x .75 = 218 - 202 = 16 psi. (Calculated)

The wall was expected to fail in diagonal tension when the shearing stress load reached the value of 192.00 psi. The wall analyzed by finite element method and using the loads at failure from the experimental investigation showed that the average shearing stress under this loading would be 158 psi.

b) Loads at Failure

The wall failed at 41,400 lb. horizontal load. The failure is shown in Photo 8-8. Photo 8-9 shows one of the blocks after failure. The wall was expected to fail in diagonal tension at 36,643 lb. This load is 4,757 lb. less than the actual load at failure.

The same procedure in all respects was used in testing this wall. The vertical load was 51.4^k, and cracks occurred in the same manner as in Wall A_4 and at the same load. The wall

was loaded beyond this point and a load of 58,000 lb. in the horizontal direction was reached when small cracks, shown in Photo 8-8, developed in the lower section. Load carrying capacity started dropping, but due to large deformation, the piston travel was exceeded forcing the test to be stopped. When the wall was taken apart, cracks were found as far as three blocks from the loaded face. No signs of failure were observed at the mortar joint between the slab and the wall.

c) Deflections

The deflections of this wall are plotted for selected loads in Figure 8-9 and for other loads the measured deflections are given in Table 8-17. Figure 8-10 shows the load-deflection curve for a point 5" above the slab in the stress free side of the wall.

TABLE 8-15. LOADING FOR FINITE ELEMENT ANALYSIS

WALL A_5

NODE	PX(KIPS)	PY(KIPS)
1	6.6700	5.7000
2	6.6700	0.0000
3	6.6700	0.0000
4	6.6700	0.0000
5	6.6700	0.0000
6	6.6700	0.0000
7	6.6700	0.0000
15	0.0000	5.7000
29	0.0000	5.7000
43	0.0000	5.7000
57	0.0000	5.7000
71	0.0000	5.7000
85	0.0000	5.7000
99	0.0000	5.7000
113	0.0000	5.7000

TABLE 8-16. DEFLECTIONS FROM FINITE ELEMENT ANALYSIS

WALL A_5

Nodal Displacements (in)

NODE	X	Y
1	0.0057	-0.0062
2	0.0052	-0.0062
3	0.0046	-0.0059
4	0.0040	-0.0053
5	0.0033	-0.0043
6	0.0028	-0.0026
7	0.0028	-0.0000

TABLE 8-17. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR ROTATION AND HORIZONTAL FORCES CORRECTED FOR ROLLING RESISTANCE

WALL A5

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5
3,600	0.0000	0.0000	0.0000
5,400	0.0008	0.0003	0.0006
7,800	0.0013	0.0004	0.0026
9,000	0.0018	0.0020	0.0077
10,800	0.0020	0.0027	0.0097
12,600	0.0023	0.0033	0.0102
14,400	0.0024	0.0041	0.0117
16,200	0.0023	0.0047	0.0143
18,000	0.0047	0.0052	0.0150
19,800	0.0056	0.0062	0.0164
21,600	0.0141	0.0166	0.0180
23,400	0.0165	0.0183	0.0196
25,500	0.0159	0.0197	0.0202
27,000	0.0194	0.0200	0.0231
28,800	0.0273	0.0212	0.0217
31,600	0.0293	0.0330	0.0345
32,400	0.0379	0.0407	0.0447
34,200	0.0453	0.0600	0.0613
36,000	0.0532	0.0778	0.0706
37,800	0.0601	0.0861	0.1015
39,800	0.1029	0.1200	0.1296
41,400	0.1095	0.1306	0.1380

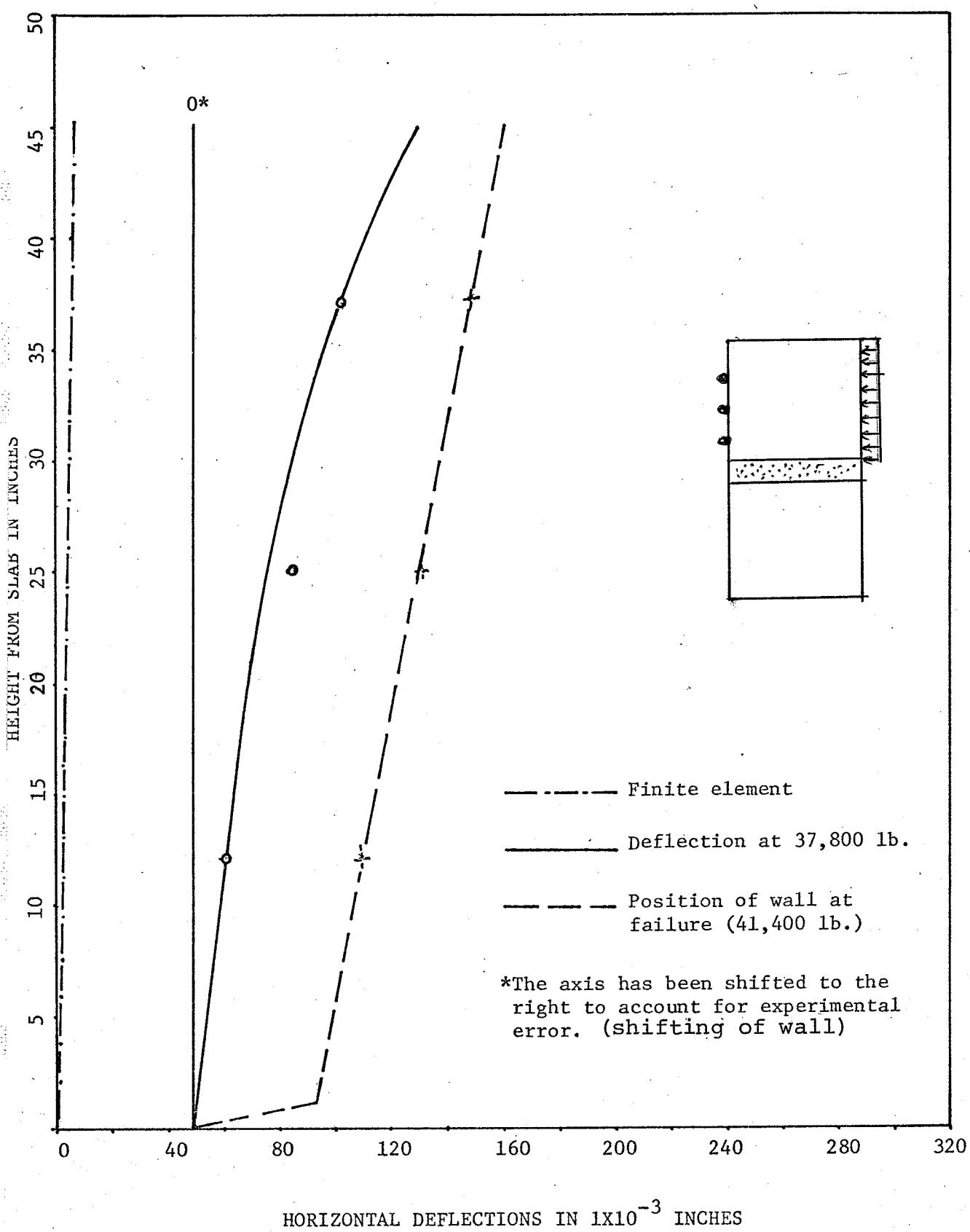


FIGURE 8-9. DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL A_5

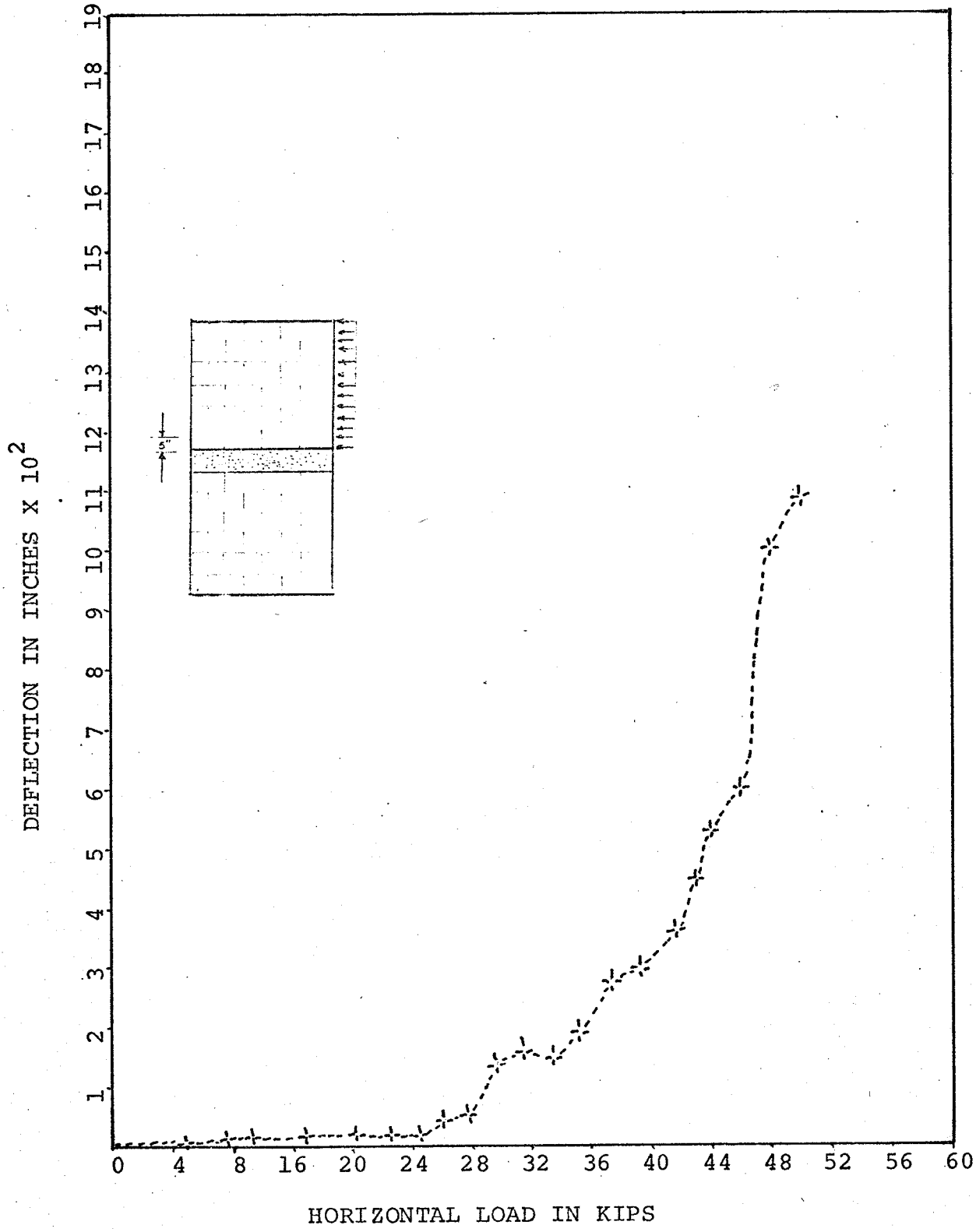


FIGURE 8-10. LOAD-DEFLECTION CURVE FOR A POINT 5" ABOVE THE SLAB ON THE FREE OF STRESSES SIDE. WALL A_5

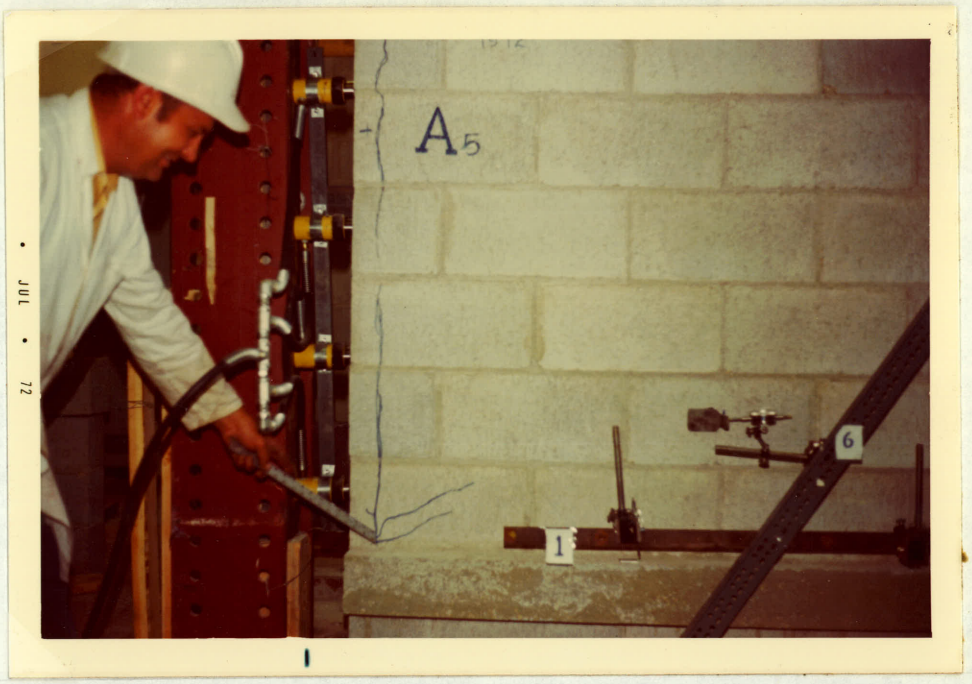


Photo 8-8. Wall A_5 after failure. Note diagonal cracks and vertical crack.

Photo 8-9. Third vertical block from bottom left corner after failure.

Testing of Group B

This group involved five walls similar to those of Group A. Shear strength of mortar is 54.5 psi, referring to the 28-day strength, but this strength was expected to be lower at the time of the test due to the adverse effect of the additive in masonry cement. To investigate the effect of having mortar with a low shear strength, this group was tested with vertical loads identical to those of A_3, A_4 and A_5 for walls B_3, B_4 and B_5 and A_1 for B_1 and B_2. The contact area between blocks and slab is 190.5 in^2.

TABLE 8-18. LOADING OF GROUP B WITH LOADS AT FAILURE

Wall	Total Vertical Load in kips	Vertical Stress in psi	Total Horizontal Load at Failure in kips	Shear Stress in psi
B_1	2.33	12.3	5,400	28.3
B_2	2.33	12.3	5,400	28.3
B_3	28.3	149	34,200	180
B_4	39.95	210	39,800	221
B_5	51.4	270	36,000	188

The results are analyzed in a manner similar to the procedure followed in Group A. The results from the analytical investigation are also discussed.

Wall B_1

a) Stresses

Wall B_1 failed in shear at a shearing stress of

$$\frac{5400\ lb}{190.5 in^2} = 28.3\ psi$$

The vertical stress was 12.3 psi and therefore the mortar shear strength was

$$Vo = 28.3 - .75 \times 12.3 = 19.2\ psi$$

The wall was expected to fail when a shearing stress of 105.00 psi was reached.

b) Loads at Failure

The failure occurred at 5,400 lb. horizontal load. This load is 4,350 lb. less than the expected load of 10,350 lb.

c) Deflections

Measurements of deflections are given in the form of tables and a graph in the following pages. Table 8-19 gives the deflections which are plotted in Figure 8-11. Load deflection curve is not plotted for Wall B_1 because of the limited amount of deflection data.

TABLE 8-19. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR ROTATION AND HORIZONTAL FORCES CORRECTED FOR ROLLING RESISTANCE

WALL B1

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6
3,600	0.0000	0.0000	0.0000	0.0000
5,400	0.0060	0.0076	0.0400	0.0077

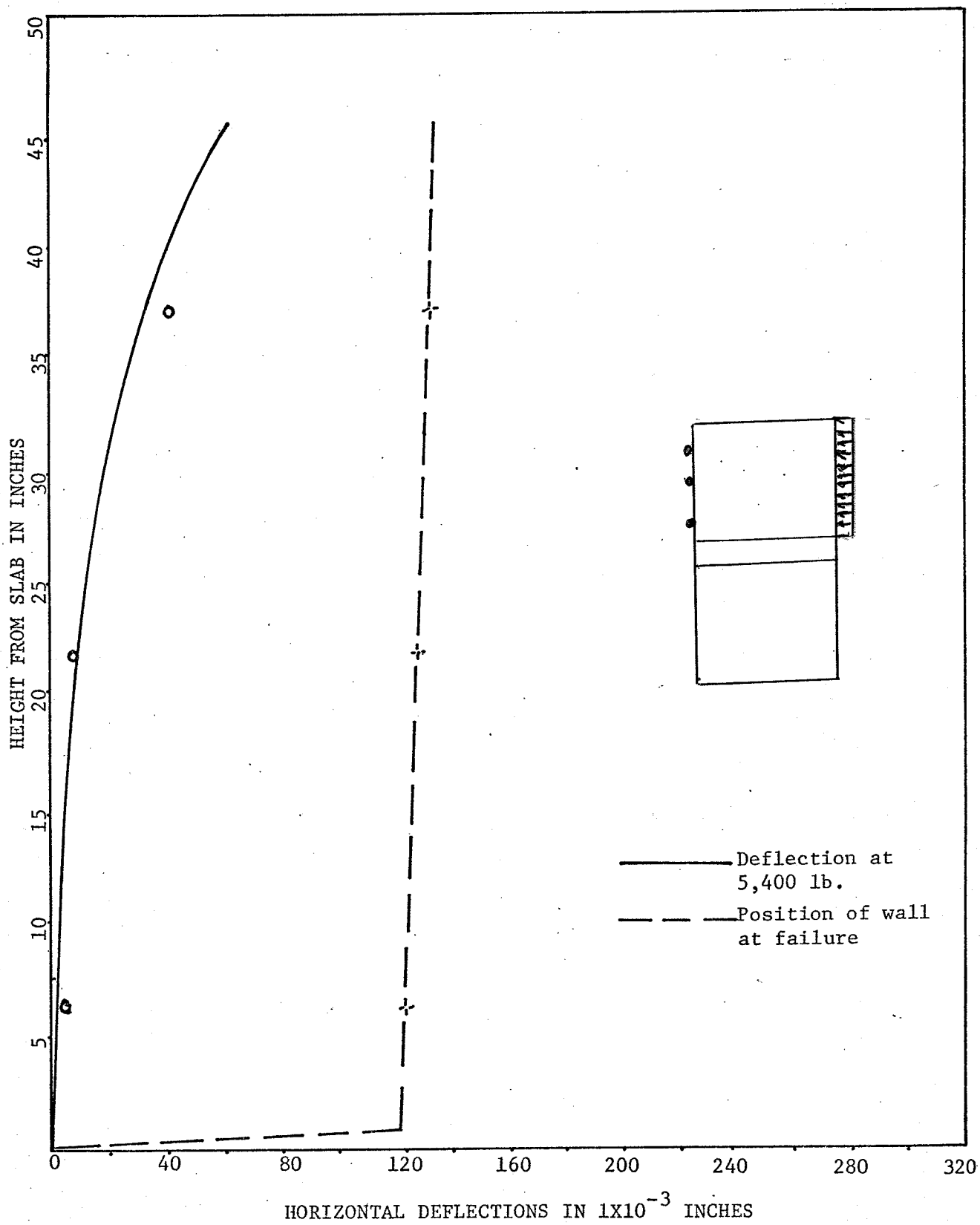


FIGURE 8-11 DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL B_1

Wall B_2

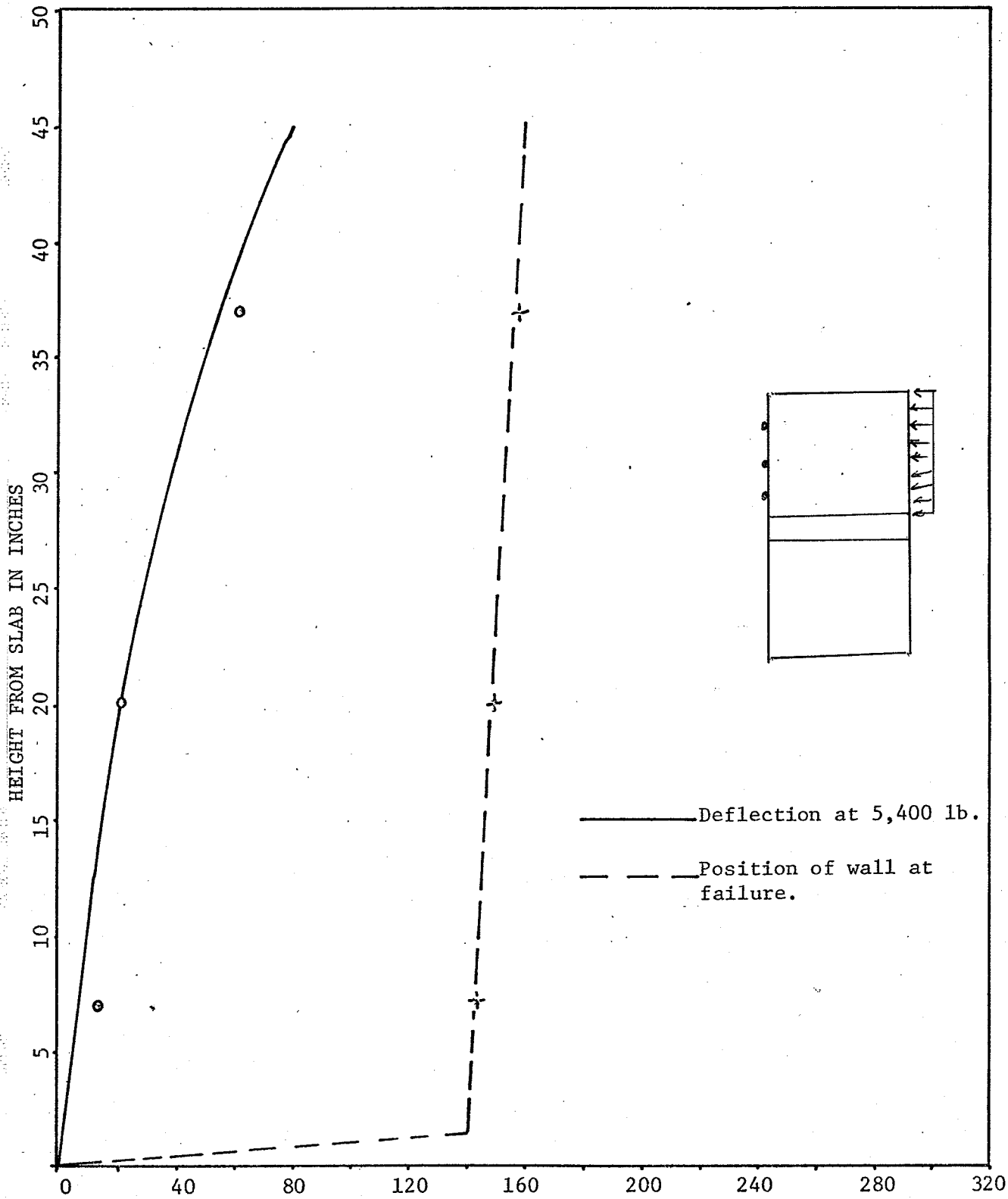
Wall B_2 was tested at the same vertical stress as Wall B_1 and failed at exactly the same load. The comments and calculations made for Wall B_1 apply for Wall B_2. The reason for carrying out this test was to verify the adverse effect of polyvinyl acetate as an additive, when masonry cement is used in the mortar.

Table 8-20 gives deflections and Figure 8-12 is a plot of the deflections.

TABLE 8-20. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR ROTATION AND HORIZONTAL FORCES CORRECTED FOR ROLLING RESISTANCE

WALL B_2

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6
3,600	0.0000	0.0000	0.0000	0.0000
5,400	0.0340	0.0211	0.0932	0.1457



HORIZONTAL DEFLECTIONS IN 1×10^{-3} INCHES

FIGURE 8-12. DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL B_2

Wall B_3

a) Stresses - Strains

The vertical stress was 149 psi and the wall failed in shear at $\frac{34,200 \text{ lb}}{190.5 \text{ in}^2} = 180$ psi

The frictional contribution was

$$0.75 \times \frac{48.3^k}{190.5 \text{in}^2} = 148.9 \text{ psi}$$

The mortar therefore contributed

$$Vo = \frac{34,200}{190.5 \text{in}^2} - 148.9 = 179.5 - 148.9 = 30.7 \text{ psi}$$

The wall was expected to fail at a shearing stress of 46.00 psi. (Adverse effect of additive - See Appendix A.)

Wall B_3 had strain gauges attached, and was tested using the same procedure used for the other walls. The results from the strain gauges are given in Appendix D. The strains from strain gauge A, in the x-direction, are plotted in Figure 8-13. (See Figure 7-1 and Photo 7-2.)

b) Loads at Failure

The wall failed at 34,200 lb. horizontal load. This load is larger by 4,140 lb. than the expected 30,060 lb. load.

c) Deflections

Table 8-21 gives the deflections as measured. The deflections for selected loads are plotted on Figure 8-14.

Figure 8-15 shows a load-deflection relation for a point 5" above the free of stresses side of the wall.

TABLE 8-21. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR TOTATION AND HORIZONTAL FORCES CORRECTED FOR ROLLING RESISTANCE

WALL B3

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6
3,600	0.0000	0.0000	0.0000	0.0000
5,400	0.0095	0.0065	0.0048	0.0044
7,800	0.0236	0.0224	0.0216	0.0187
9,000	0.0401	0.0369	0.0361	0.0345
10,800	0.0646	0.0694	0.0626	0.0632
12,600	0.0869	0.0881	0.0834	0.0848
14,400	0.1176	0.1194	0.1063	0.1051
16,200	0.1240	0.1290	0.1303	0.1326
18,000	0.1307	0.1645	0.1701	0.1655
19,800	0.1804	0.1866	0.1892	0.1837
21,600	0.2091	0.1905	0.1958	0.2044
23,400	0.2203	0.2227	0.2383	0.2344
25,500	0.2287	0.2323	0.2412	0.2379
27,000	0.2452	0.2598	0.2624	0.2561
28,800	0.2481	0.2560	0.2640	0.2833
31,600	0.2551	0.2629	0.2728	0.2934
32,400	0.2688	0.2812	0.2935	0.2910
34,200	0.3000	0.3120	0.3363	0.3070
36,000	0.7410	0.8040	0.9956	0.6873

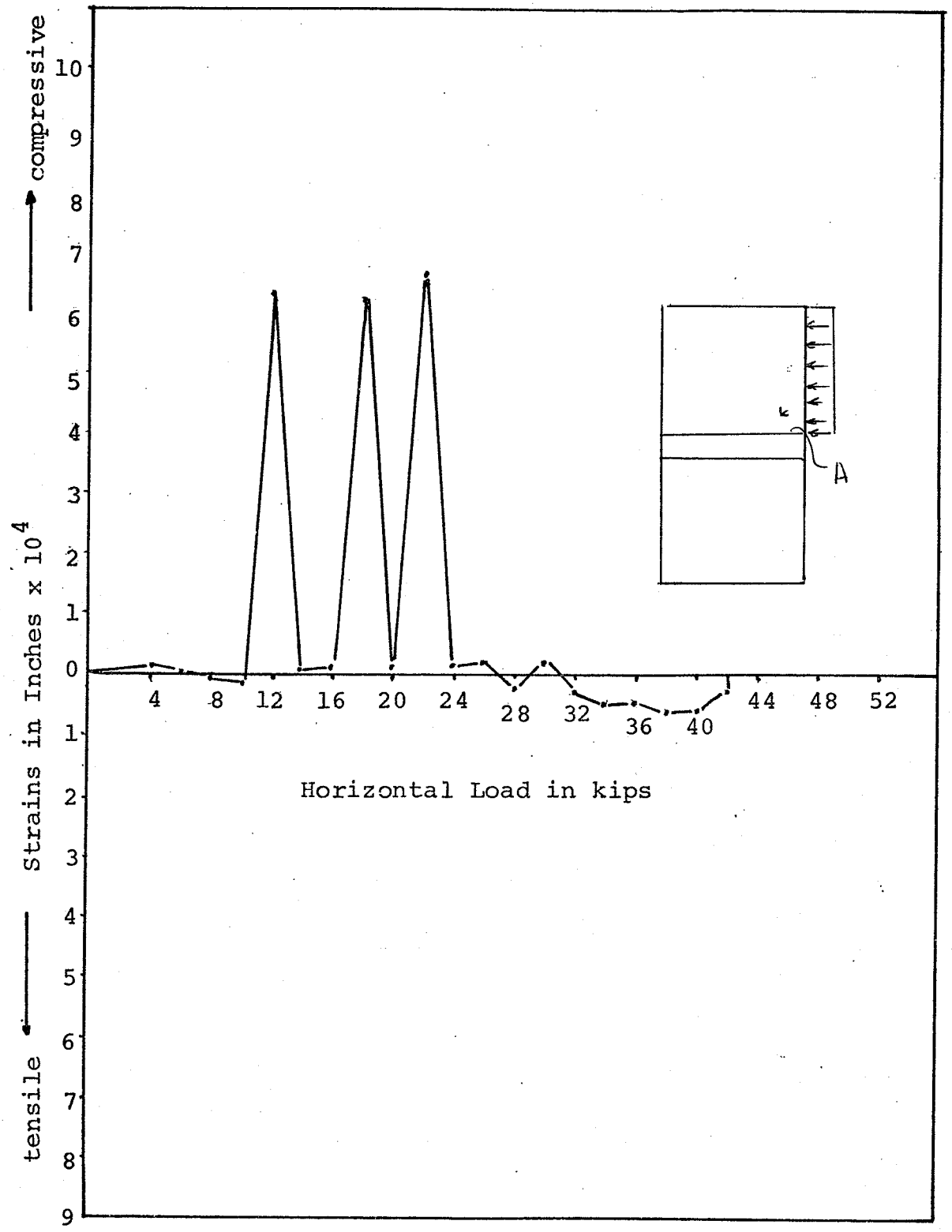


FIGURE 8-13. LOAD-STRAIN RELATIONS

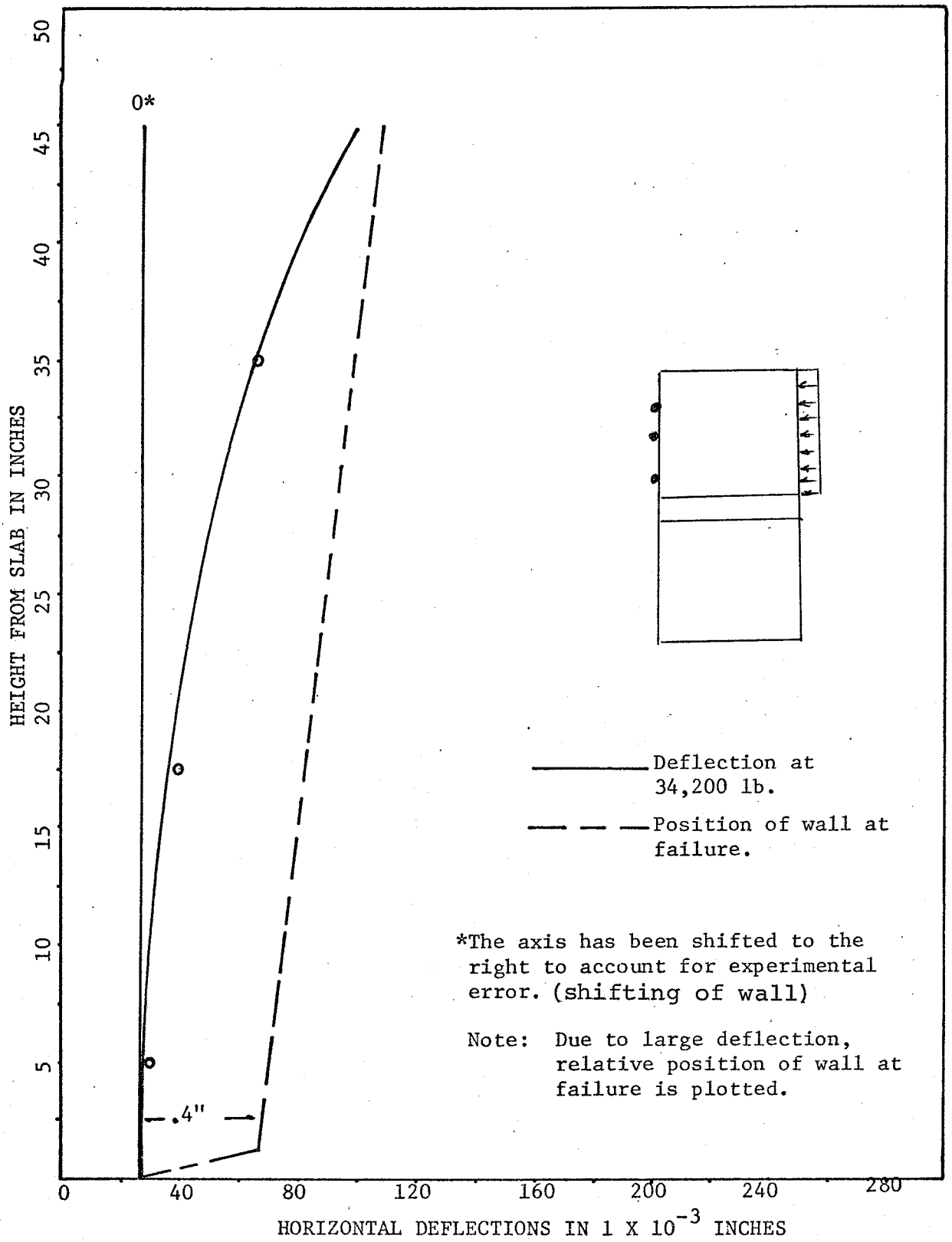


FIGURE 8-14. DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL B_3

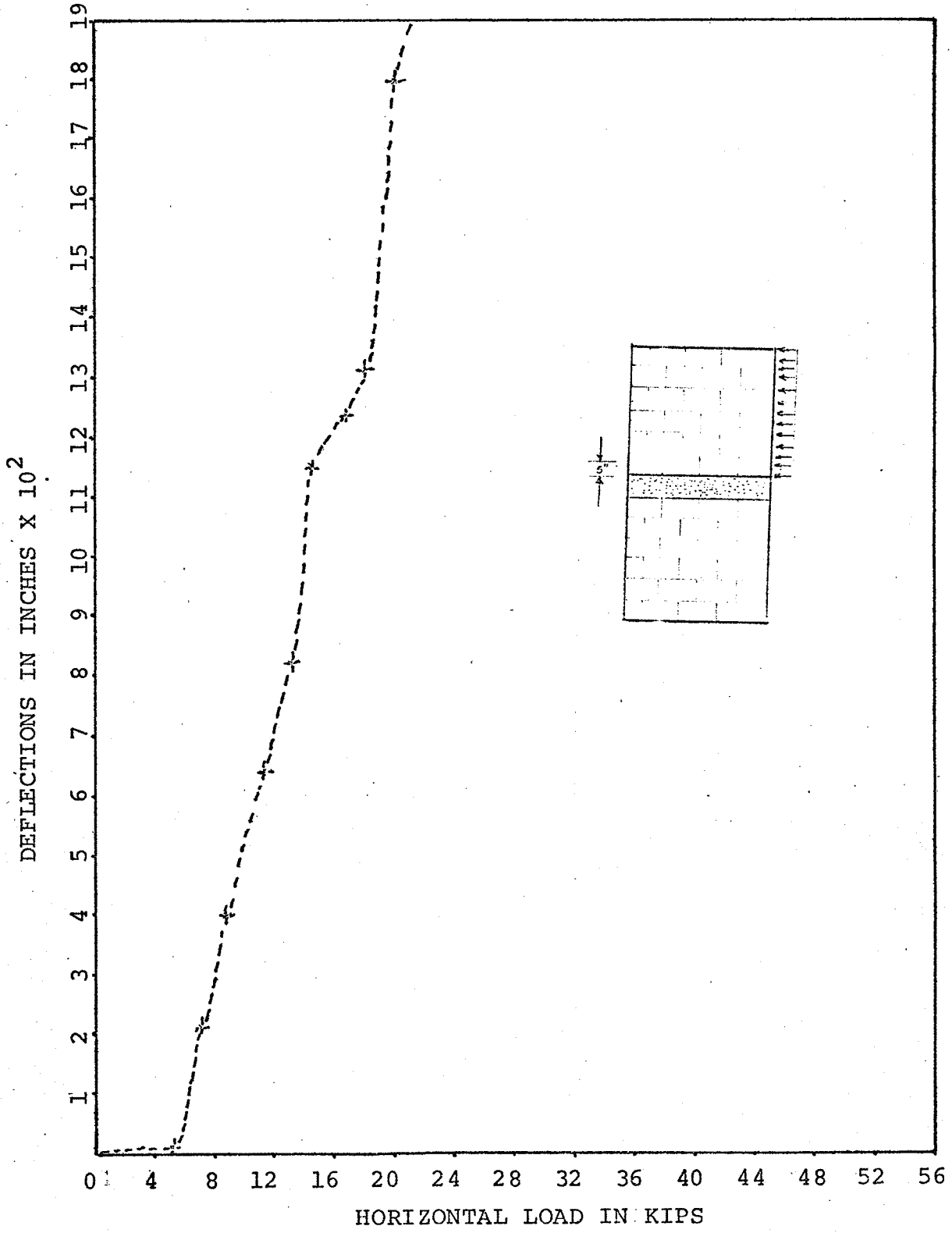


FIGURE 8-15. LOAD-DEFLECTION CURVE FOR A POINT 5" ABOVE THE SLAB ON THE FREE OF STRESSES SIDE. WALL B_3

Wall B_4

a) Stresses

Wall B_4 failed as shown in Photo 8-10. The shearing stress at the connection was

$$\frac{89,800 \text{ lb}}{190.5 \text{ in}^2} = 210 \text{ psi}$$

The wall was expected to fail at 176.00 psi shearing stress, by diagonal tension.

Photo 8-11 shows another view of the failure.

b) Loads at Failure

The load at failure was 39,800 lb. which is 5,260 lb. larger than the expected 33,540 lb. load.

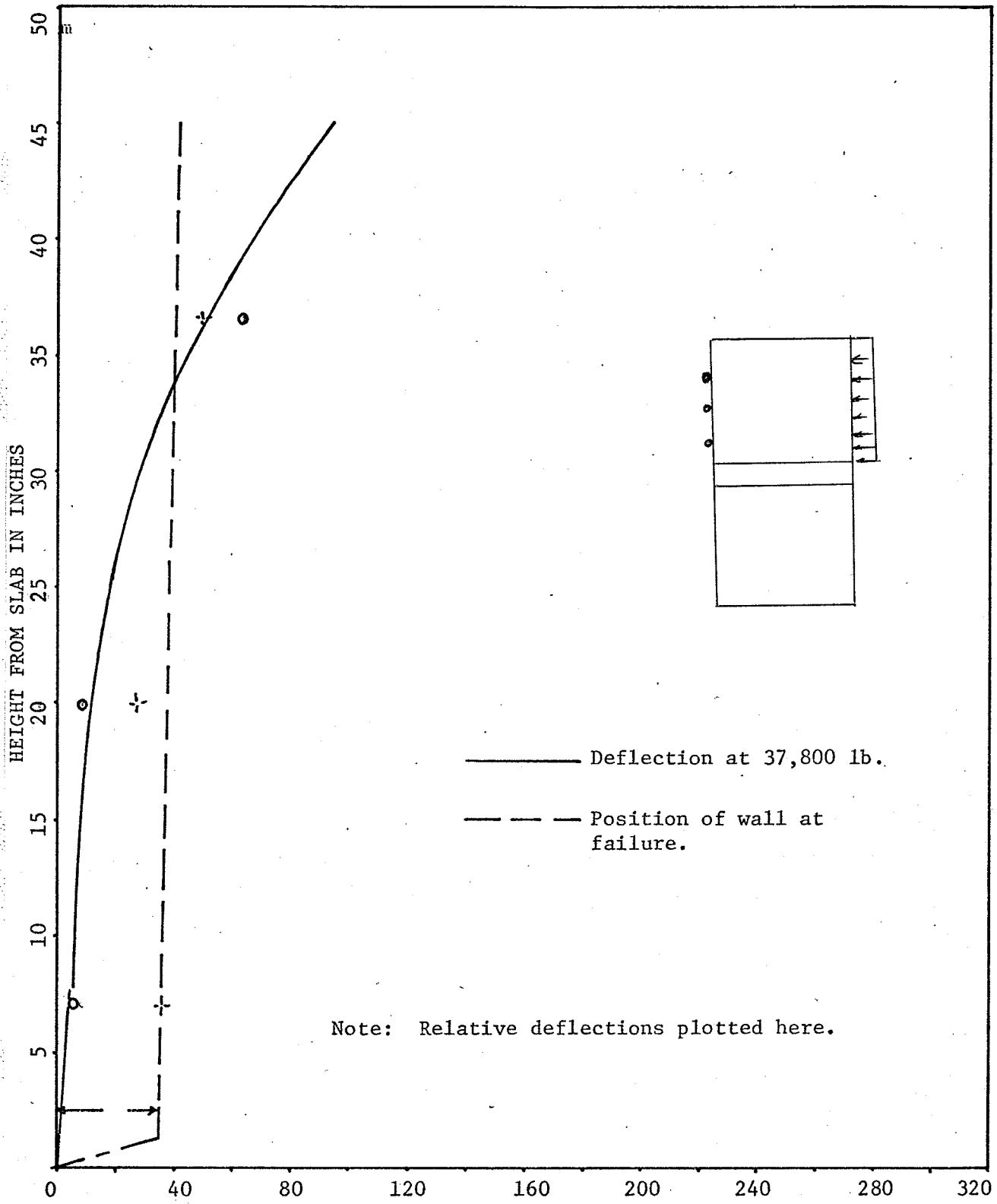
c) Deflections

The deflections as measured are plotted for selected loads in Figure 8-16. The deflections for other loads are given on Table 8-22. Figure 8-17 shows the load-deflection curve for a point 5" above the slab on the unloaded side of the wall.

TABLE 8-22. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR TOTATION AND HORIZONTAL FORCES CORRECTED FOR ROLLING RESISTANCE

WALL B4

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6	
3,600	0.0000	0.0000	0.0000	0.0000	
5,400	0.0057	0.0051	0.0059	0.0150	
7,800	0.0196	0.0219	0.0240	0.0322	
9,000	0.0313	0.0340	0.0369	0.0442	
10,800	0.0462	0.0518	0.0600	0.0624	
12,600	0.0566	0.0608	0.0645	0.0741	
14,400	0.0867	0.0922	0.1038	0.1206	
16,200	0.1004	0.1194	0.1237	0.1367	
18,000	0.1060	0.1090	0.1109	0.1283	
19,800	0.1237	0.1241	0.1264	0.1459	
21,600	0.1322	0.1357	0.1391	0.1588	
23,400	0.1436	0.1569	0.1645	0.1673	
25,500	0.1537	0.1642	0.1768	0.1856	
27,000	0.1631	0.1885	0.2007	0.2085	
28,800	0.1642	0.1737	0.2031	0.2154	
31,600	0.1725	0.1796	0.2121	0.2227	
32,400	0.1780	0.1791	0.2204	0.2269	
34,200	0.1829	0.1879	0.2285	0.2428	
36,000	0.1931	0.1993	0.2493	0.2583	
37,800	0.2065	0.2075	0.2657	0.3162	
39,800	0.6386	0.6279	0.5520	0.5573	Failure



HORIZONTAL DEFLECTIONS IN $1X10^{-3}$ INCHES

FIGURE 8-16 DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL B_4

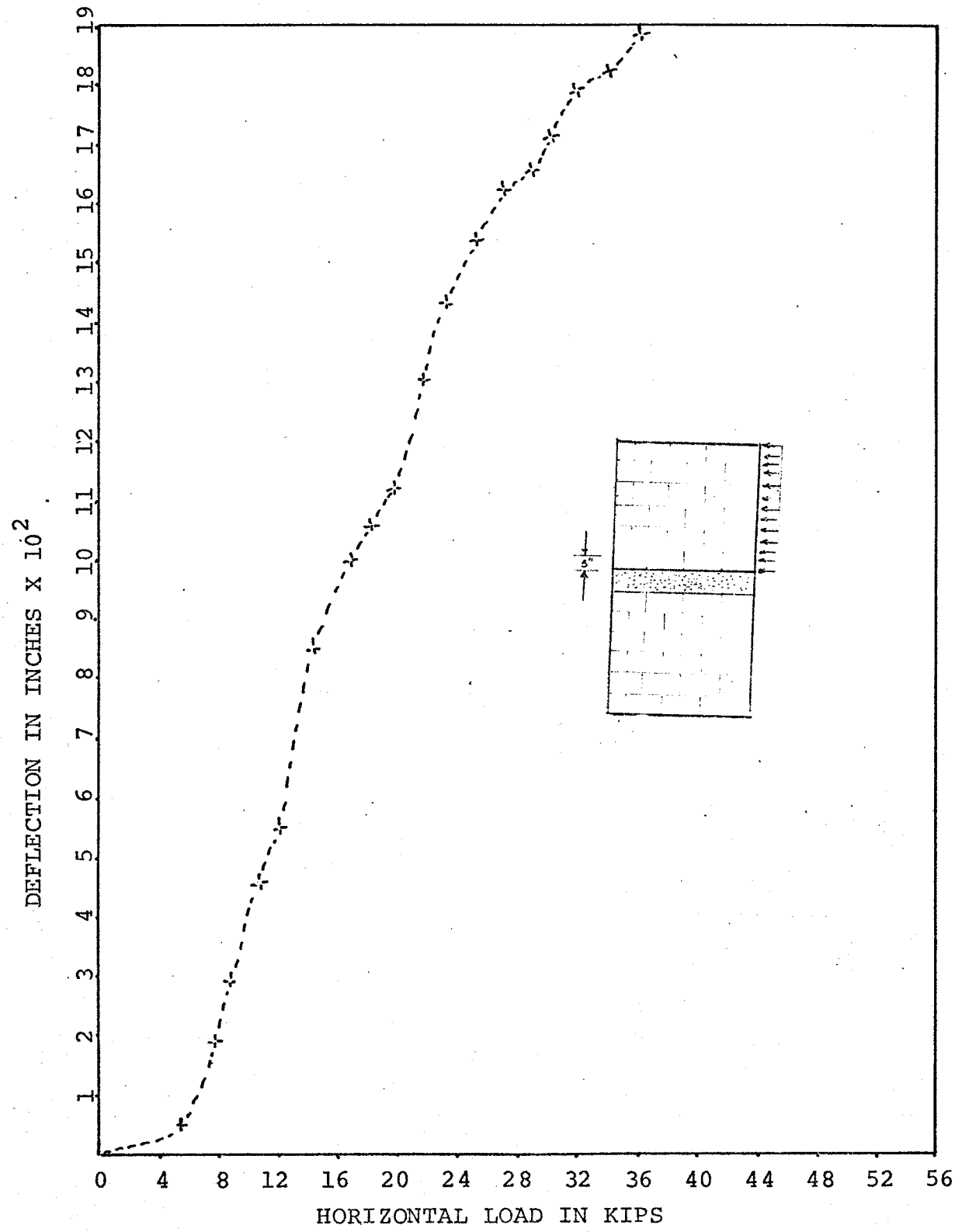


FIGURE 8-17. LOAD-DEFLECTION CURVE FOR A POINT 5" ABOVE THE SLAB ON THE FREE OF STRESSES SIDE. WALL B_4

Photo 8-10. Failure of Wall B_4.

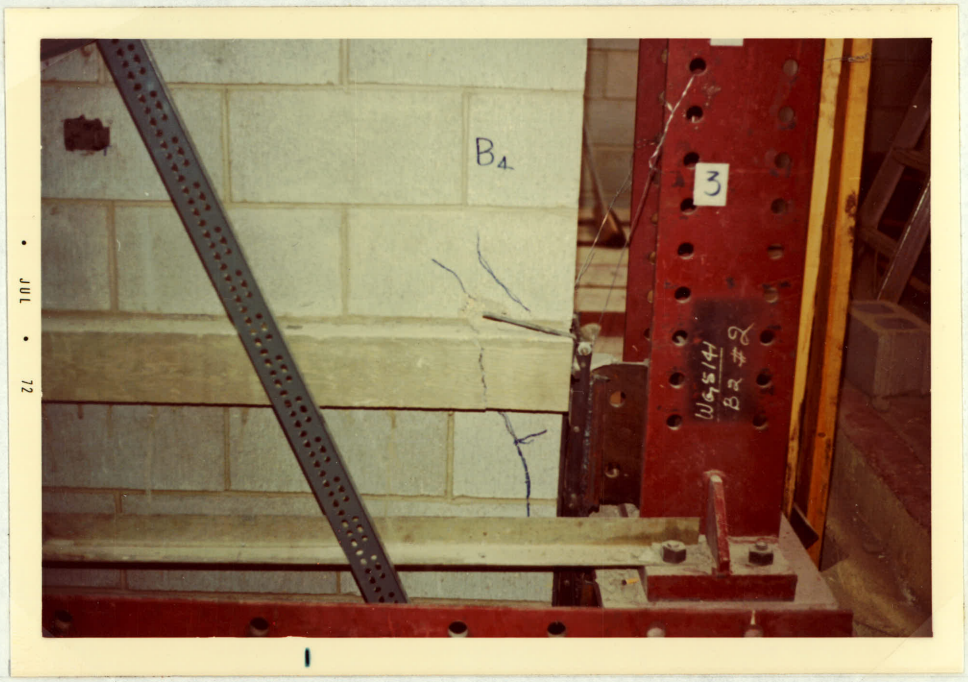


Photo 8-11. View of Wall B_4.

Wall B_5

a) Stresses

When Wall B_5 failed by cracking as shown in Photo 8-12. the connection between slab and wall remained intact. The shearing stress at the connection was

$$\frac{36{,}000\ \text{lb}}{190.5\ \text{in}^2} = 188\ \text{psi}$$

which is almost identical to the predicted 192.00 psi. The contribution of the mortar was

$188 - 0.75 \times 270 = 188 - 203 = 15$ psi

270 psi being the vertical stress. The mortar contributed nothing in resisting the load.

b) Loads at Failure

The wall failed at 36,000 lb. which is almost the same as the predicted 36,600 lb.

c) Deflections

The measured deflections are given on Table 8-23 and a deflection curve for the wall, at 36,000 lb. horizontal load, is plotted on Figure 8-18. Figure 8-19 shows the load-deflection curve for the same point as for the other walls.

TABLE 8-23. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR ROTATION AND HORIZONTAL FORCES CORRECTED FOR ROLLING RESISTANCE

WALL B5

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6
3,600	0.0000	0.0000	0.0000	0.0000
5,400	0.0023	0.0030	0.0046	0.0035
7,800	0.0073	0.0091	0.0098	0.0089
9,000	0.0082	0.0110	0.0116	0.0074
10,800	0.0161	0.0184	0.0198	0.0182
12,600	0.0195	0.0196	0.0309	0.0205
14,400	0.0134	0.0237	0.0358	0.0215
16,200	0.0152	0.0248	0.0346	0.0244
18,000	0.0163	0.0289	0.0369	0.0258
19,800	0.0173	0.0248	0.0332	0.0262
21,600	0.0180	0.0322	0.0419	0.0251
23,400	0.0181	0.0284	0.0322	0.0230
25,500	0.0194	0.0156	0.0320	0.0224
27,000	0.0192	0.0148	0.0287	0.0193
28,800	0.0189	0.0189	0.0254	0.0172
31,600	0.0223	0.0230	0.0283	0.0151
32,400	0.0230	0.0252	0.0310	0.0120
34,200	0.0227	0.0266	0.0367	0.0100
36,000	0.0238	0.0333	0.0852	0.0240

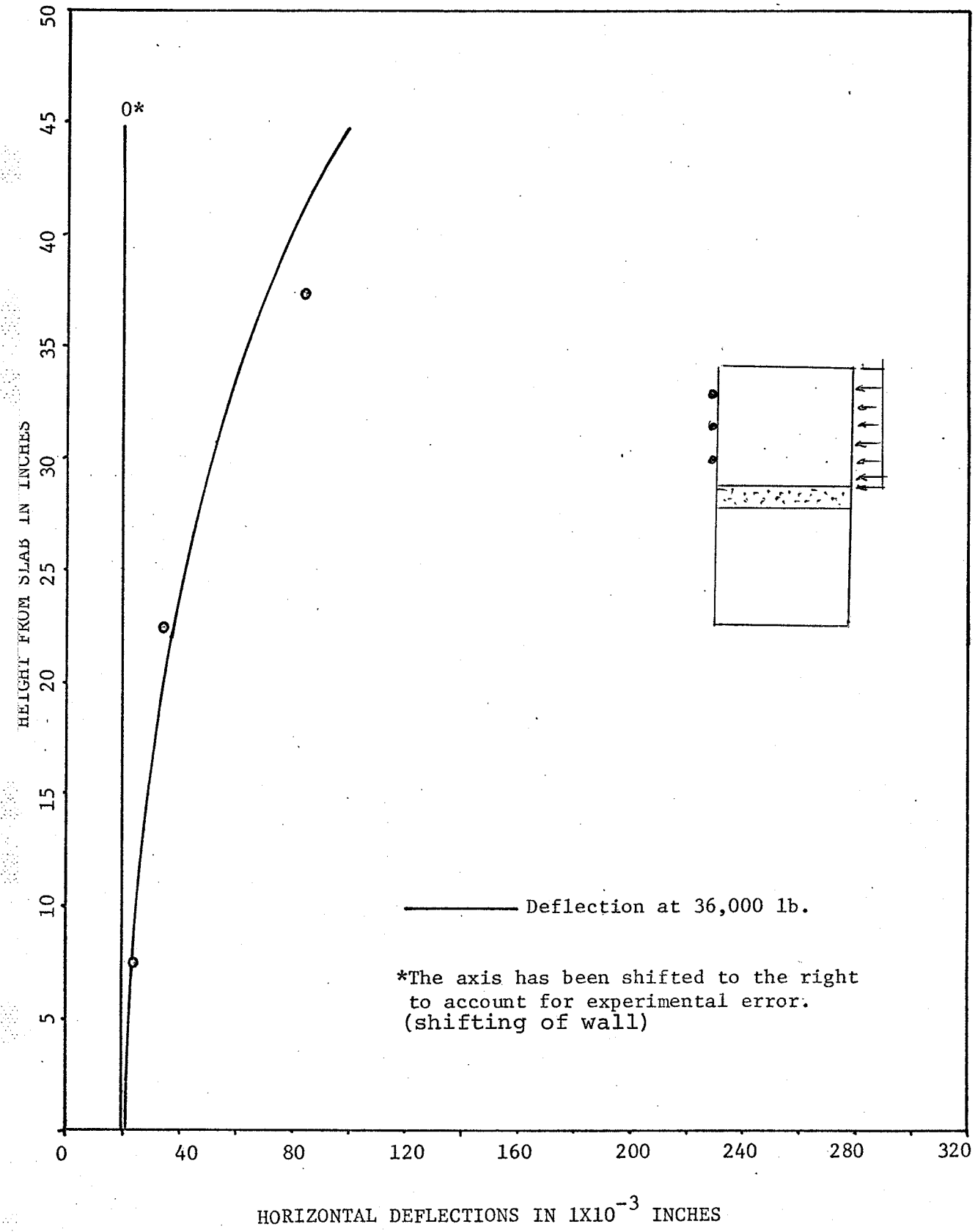


FIGURE 8-18. DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL B_5

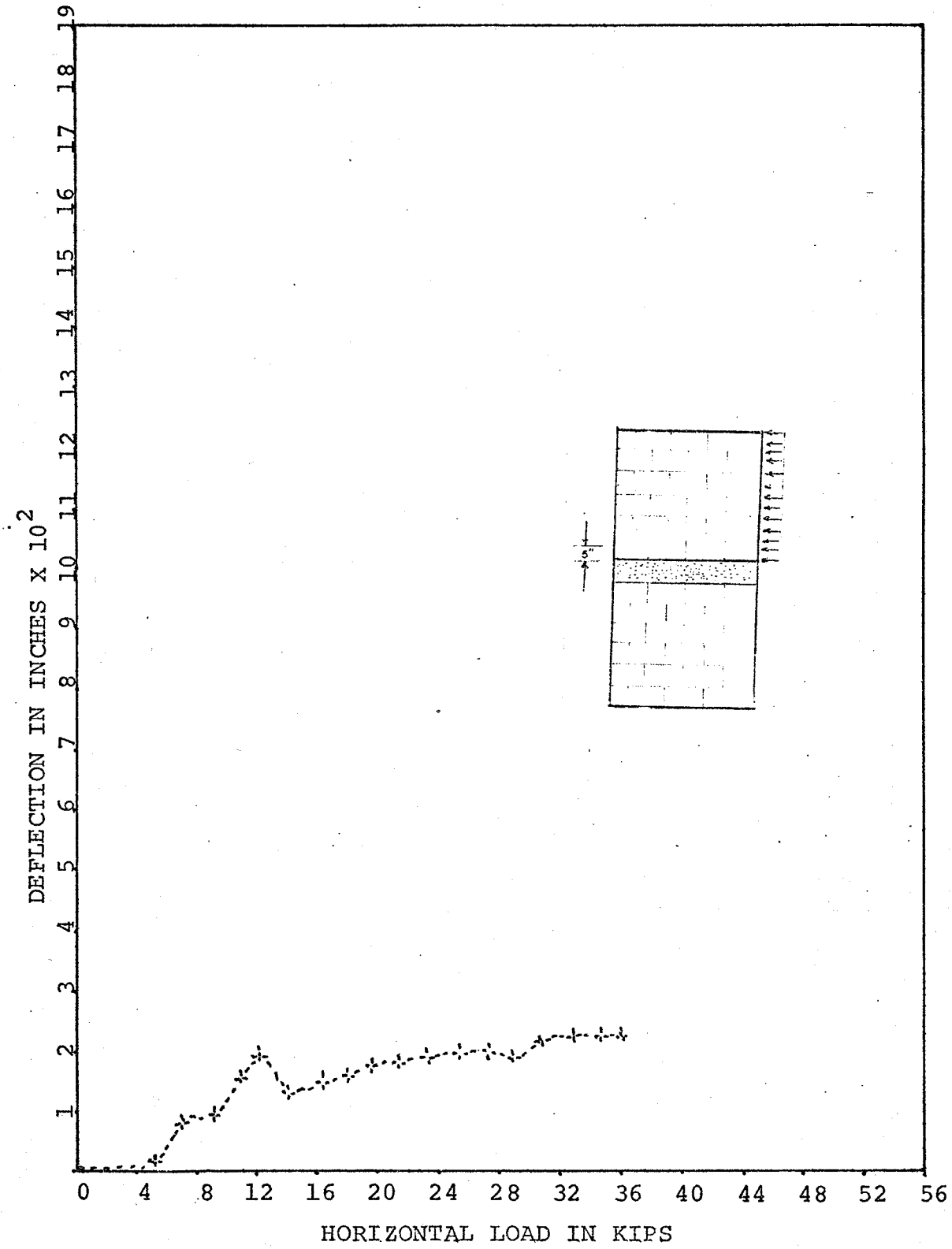


FIGURE 8-19. LOAD-DEFLECTION CURVE FOR A POINT 5" ABOVE THE SLAB ON THE FREE OF STRESSES SIDE. WALL B_5

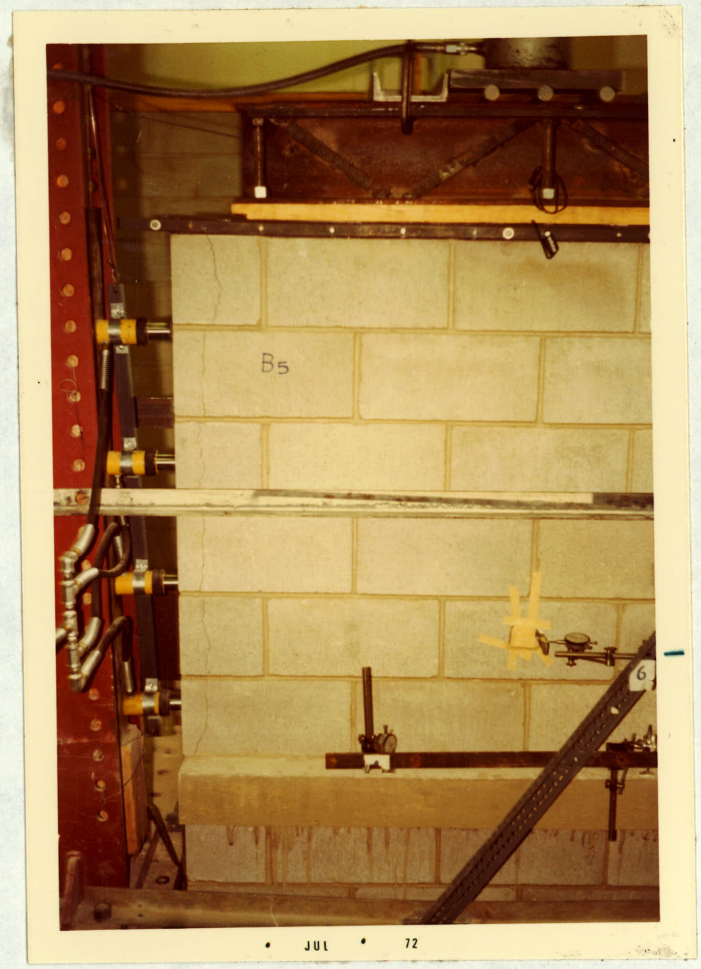


Photo 8-12 Wall B_5 after failure.

Testing of Group C

The shear strength of the mortar at 28 days was 158 psi. The hollow wall was tested at zero vertical load. Wall C_2 was tested at 2.33 vertical load. The shear resisting area in the case of Wall C_2 was the total cross-sectional area of the wall, i.e. 7.625 x 63.5 = 485 in^2, and for Wall C_1 it was 190.5 in^2. The other three walls, C_3, C_4 and C_5 were not tested due to limitation of time and capacity of loading frame.

TABLE 8-24. LOADING OF GROUP C AND TEST RESULTS

Wall	Total Vertical Load in kips	Vertical Stress in psi	Total Horizontal Load at Failure in kips	Shear Stress at Failure in psi
C_1	0	0	34,600	182
C_2	2.33	12.3	39,600	82

Wall C_1

a) Stresses

The shearing stress at failure was $\frac{34{,}600\ lb}{190.5 in^2} = 182$ psi. The wall was expected to fail at 158 psi shearing stress which is 0.87 the actual shear at failure. The failure was a clear shear failure. Photo 8-13 shows the wall after failure.

b) Load at Failure

The wall failed at 34,600 lb. horizontal load. The vertical load was 0.0 lb.

c) Deflections

The deflections are given on Table 8-25 and the deflection for 12,600 lb. horizontal load is plotted on Figure 8-20. Figure 8-21 shows the load-deflection curve for the same point as for all other test specimens.

TABLE 8-25. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR ROTATION AND HORIZONTAL FORCES CORRECTED FOR ROLLING RESISTANCE

WALL C1

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6
3,600	0.0000	0.0000	0.0000	0.0000
5,400	0.0139	0.0250	0.0173	0.0012
7,800	0.0327	0.0427	0.0419	0.0143
9,000	0.0432	0.0523	0.0619	0.0250
10,800	0.0525	0.1037	0.1896	0.0387
12,600	0.0583	0.1004	0.1053	0.0447
14,400	0.0706	0.1824	0.2213	0.0592
16,200	0.0808	0.2034	0.2373	0.0707
18,000	0.0851	0.1964	0.2548	0.0721
19,800	0.0891	0.1977	0.2648	0.0724
21,600	0.0949	0.2020	0.3763	0.0763
23,400	0.1004	0.2067	0.3873	0.0780
25,500	0.1055	0.2114	0.3966	0.0833
27,000	0.3388	0.3550	0.4721	0.3655
28,800	0.3469	0.3692	0.4721	0.3751
31,600	0.3479	0.3822	0.4794	0.3898
32,400	0.3498	0.3826	0.4873	0.4067
34,200	0.3580	0.3927	0.3886	0.4379
36,000	0.3597	0.4016	0.4899	0.4719

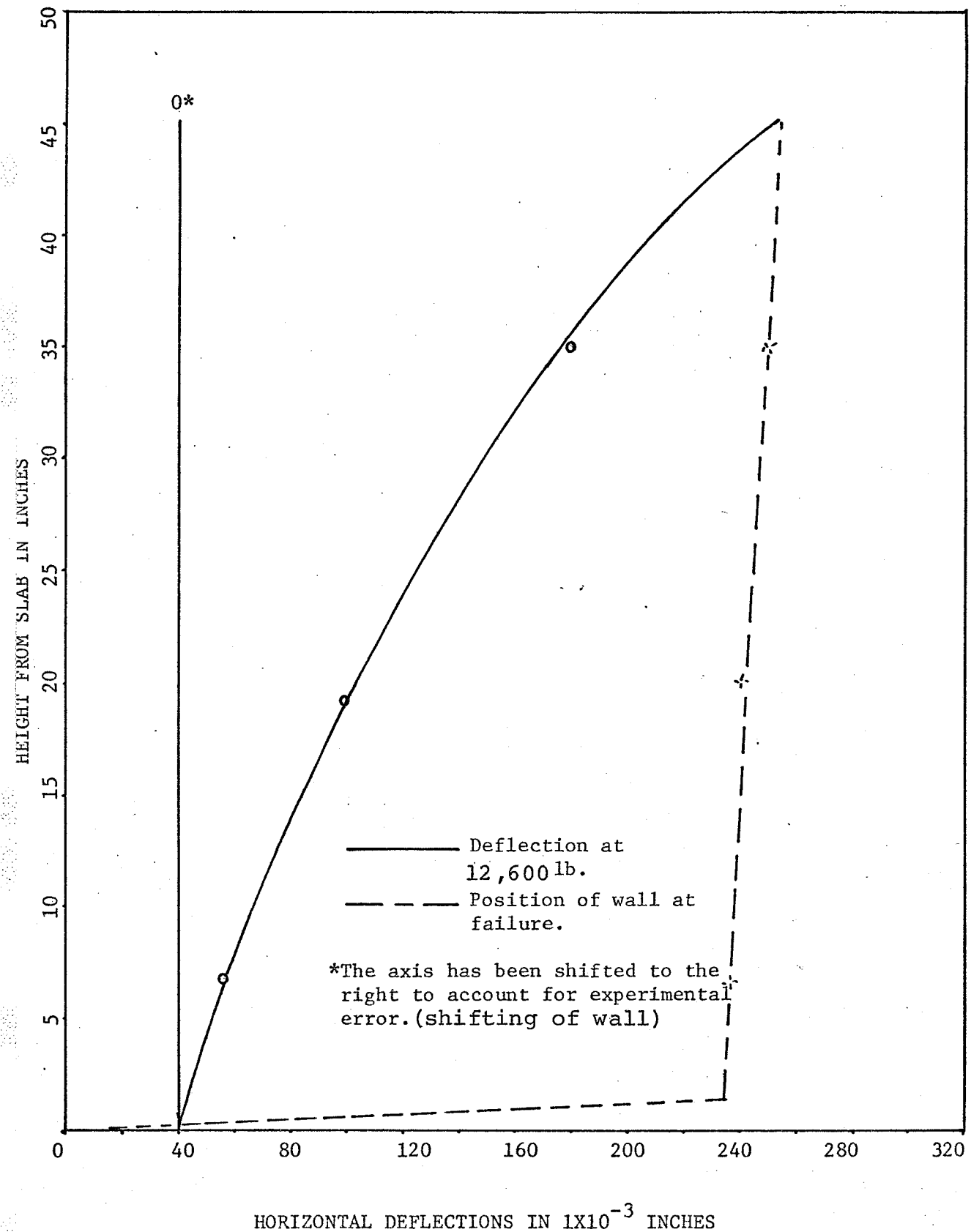


FIGURE 8-20. DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL C_1

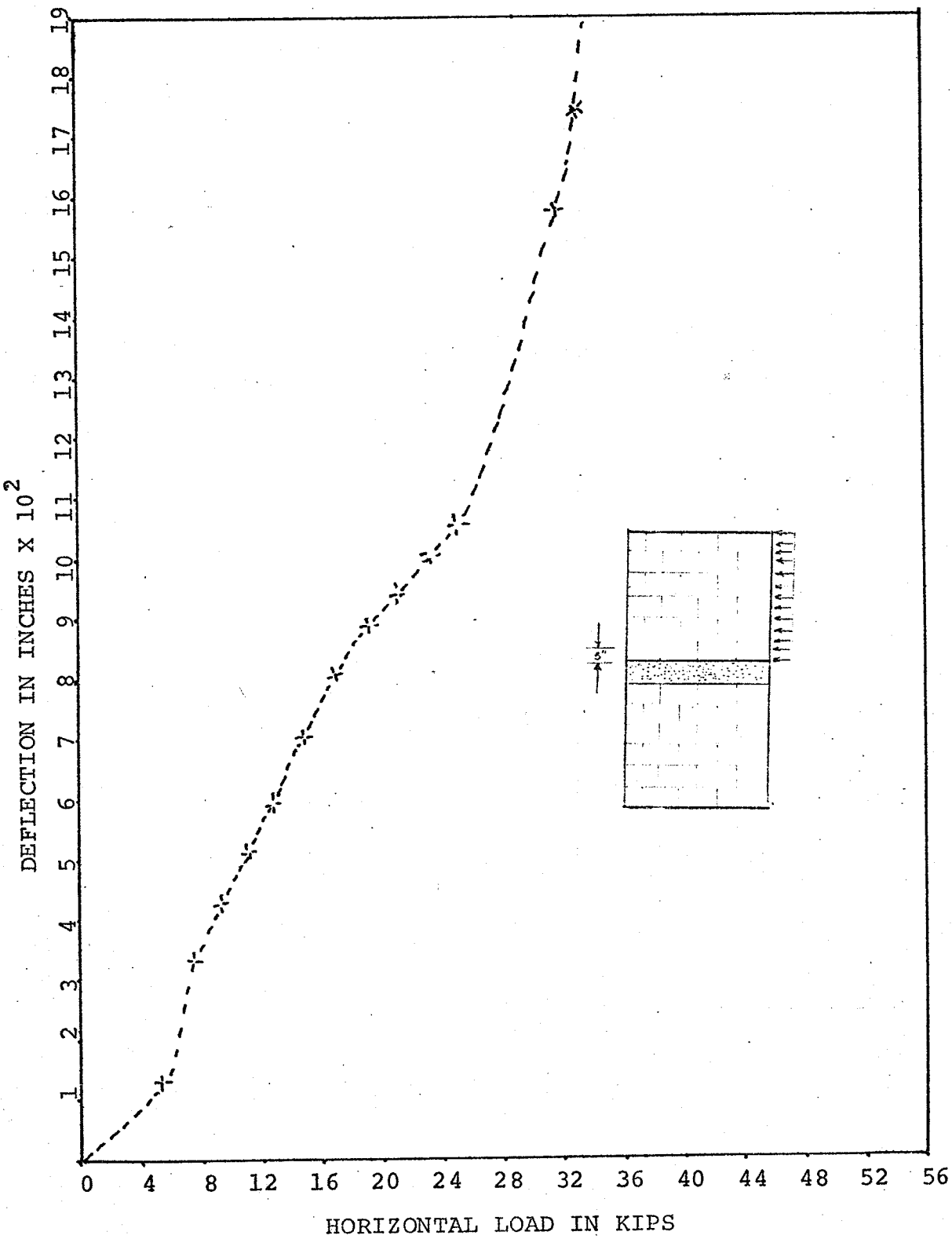


FIGURE 8-21. LOAD-DEFLECTION CURVE FOR A POINT 5" ABOVE THE SLAB ON THE FREE OF STRESSES SIDE. WALL C_1

Photo 8-13. Clear shear failure view of Wall C_1 at the joint (horizontal load 30,000 lb.)

Wall C_2

a) Stresses

This is the only filled wall tested. It failed by shear at a shearing stress of $\frac{39,600 \text{ lb}}{485 \text{ in}^2} = 82$ psi. Looking at Photos 8-14 and 8-15, a crack through the slab can be seen. The last portion of the slab remained cemented on the upper part. The failure of this wall is similar to that of Wall B_4. The crack in the slab occurred first. The remaining joint section could not carry the load and failed in shear. The capacity of the wall was relatively low. The bonding between slab and the wall was poor. The poor bonding is attributed to the fact that mortar fell in during the construction of the wall which prevented good bonding between slab and wall. Also, the fact that the slab was smooth finished may have had some effect in the poor bonding. No special precautions were taken in order to ensure good connection, for example, the surface was dry and no cement slurry was applied.

b) Load at Failure

The wall failed at 39,600 lb. horizontal load which is 11,400 lb. less than the expected load.

c) Deflections

The deflections of this wall as measured are given in Table 8-26. A graph of the deflections is given in Figure 8-22 for the deflections just before failure occurred. A load-deflection curve is shown in Figure 8-23.

TABLE 8-26. HORIZONTAL DISPLACEMENTS IN INCHES, CORRECTED FOR ROTATION AND HORIZONTAL FORCES CORRECTED FOR ROLLING RESISTANCE

WALL C2

Horizontal Load in Lbs.	Dial 3	Dial 4	Dial 5	Dial 6
3,600	0.0000	0.0000	0.0000	0.0000
5,400	0.0038	0.0041	0.0039	0.0077
7,800	0.0168	0.0167	0.0173	0.0219
9,000	0.0288	0.0275	0.0276	0.0361
10,800	0.0387	0.0372	0.0389	0.0472
12,600	0.0551	0.0540	0.0535	0.0613
14,400	0.0661	0.0647	0.0701	0.0725
16,200	0.0744	0.0705	0.0746	0.0772
18,000	0.0775	0.0744	0.0868	0.0837
19,800	0.0885	0.0861	0.0962	0.0899
21,600	0.0977	0.0979	0.0994	0.0994
23,400	0.1071	0.1005	0.1213	0.1186
25,500	0.1120	0.1201	0.1393	0.1418
27,000	0.1420	0.1348	0.1472	0.1542
28,800	0.1546	0.1483	0.1242	0.1640
31,600	0.1667	0.1500	0.2551	0.1724
32,400	0.3283	0.4423	0.4941	0.4014
34,200	0.3433	0.4433	0.4952	0.4028
36,000	0.3483	0.4437	0.4971	0.4030
37,800	0.3503	0.4496	0.5046	0.4041
39,800	0.3508	0.4816	0.5868	0.4047

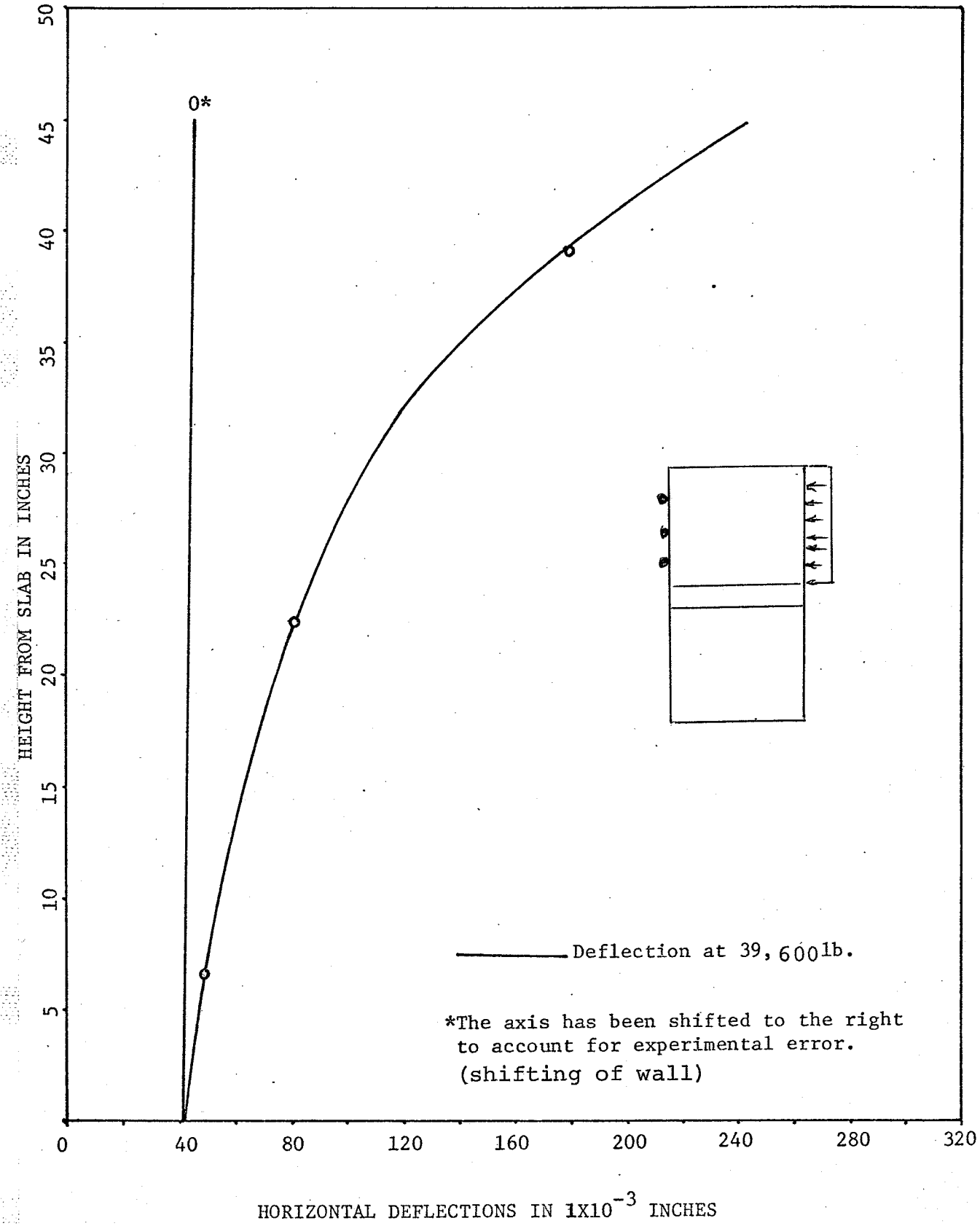


FIGURE 8-22. DEFLECTIONS DUE TO HORIZONTAL FORCES. WALL C_2

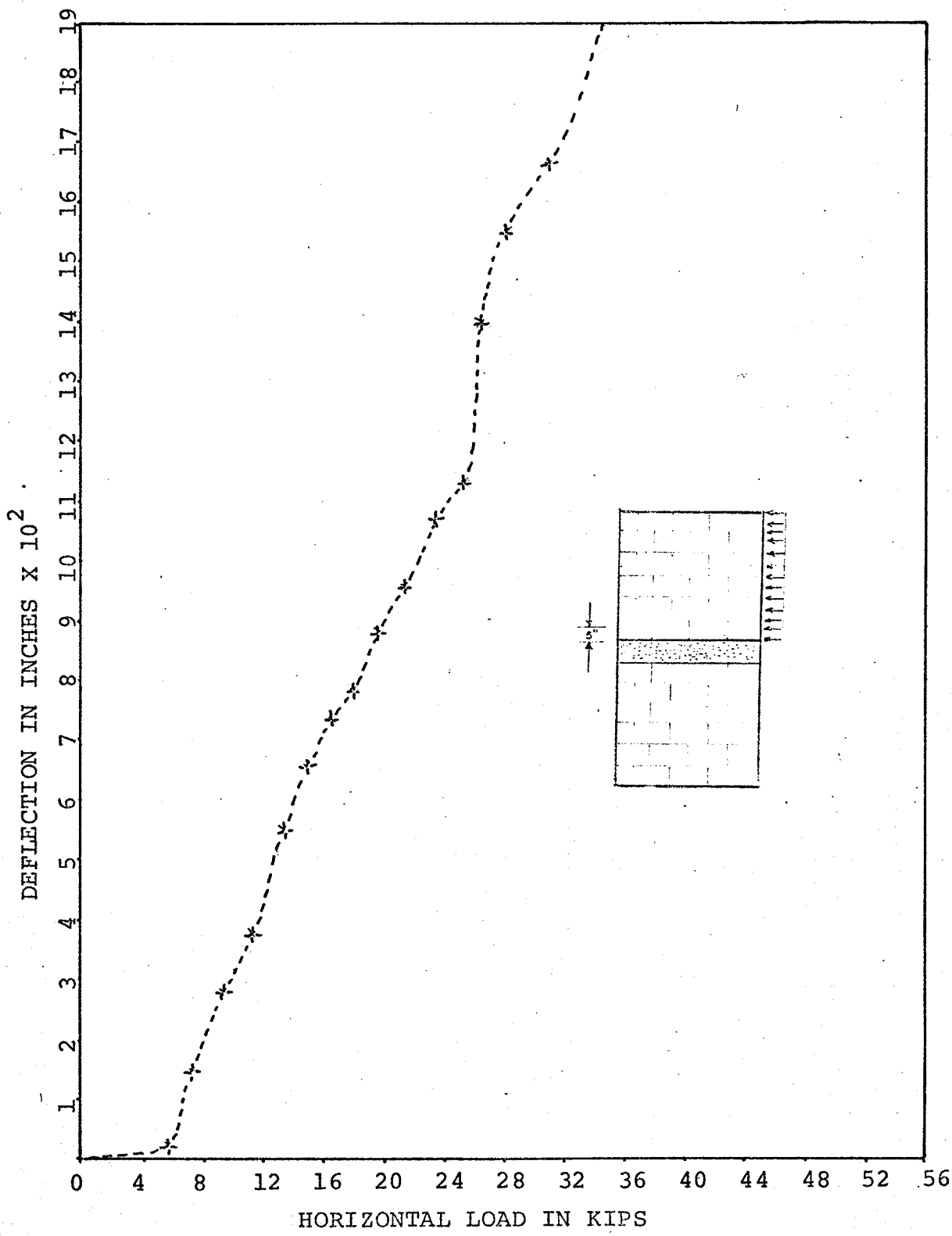


FIGURE 8-23. LOAD-DEFLECTION CURVE FOR A POINT 5" ABOVE THE SLAB ON THE FREE OF STRESSES SIDE. WALL C_2

Photo 8-14 Failure of Wall C_2.

Photo 8-15. Close-up of failure, Wall C_2.

Note the crack through the slab and block under the slab.

CHAPTER IX

Discussion of Results

a) General

The shear strength of the wall was calculated from the formulae given in Chapter II. The results of these calculations are given on Table 8-1. The ultimate shear stress calculated from the suggested formulae (Chapter II) agrees well with the experimental results. Practically no increase in the shear strength of the walls was noticed after the precompression was increased from 150 psi to 270 psi. However, the small increase in shear strength with an increase in precompression above 150 psi occurred because failure is governed by the ultimate strength of the blocks. For practical purposes, the shear strength after a certain precompression can be assumed constant. See Figure 9-6. The failures of the walls came at loads very close to those calculated from the analytical work. However, the mode of failure at high precompressive stresses was different than expected. The results and the failure modes are discussed further in this chapter.

Although some special problems such as large deflections of loading frame and local failures were spotted, the experiments can be regarded as successful. However, the conclusions given in this chapter should be investigated further.

b) Analysis of Results

From the experiments carried out it was found that two types of failure occured:

i) shear failure

ii) failure by cracking.

These two types of failures will now be discussed.

Shear Failures

Walls A_1, A_2, A_3, B_1, B_2, B_3, C_1 and C_2 failed in shear. The results obtained from the analytical work gave very good indication of the capacity of the joint. (The relation $V = Vo + \mu\sigma y$ with $\mu = 0.75$ was used in the analysis.[5]) However, analyzing the shear stress for Walls A_1, A_2 and A_3, it was observed that the contribution of the mortar was higher than the average shear stress found experimentally using the 3-block test.

At this point, it is possible to suggest that the 3-block shear test underestimates the strength of the mortar. The average strength of the mortar for the three shear failures of Walls A_1, A_2 and A_3 is 1/3 (66.3 61 + 58) = 60 psi, whereas the strength predicted from the 3-block experiment was 48.5 psi.

From these results the effective stress of the 3-block shear test can be found using a simple analogy procedure:

$$\frac{V_1}{A_1} = \frac{1}{K}\frac{Vb}{Ab}$$

where V_1 = total shearing load on the wall

A_1 = area of contact for that wall

Vb = total shearing load for 3-block test

K (Greek letter kappa) = stress reduction factor

Ab = nominal area of contact for the 3-block test

Since $\frac{V_1}{A_1}$ was found to be 60 psi and for the same mortar the 3-block test gave a shear strength of 48.5

$$K = \frac{48.5}{60} = 0.81$$

that is, the nominal stress (48.5 psi) calculated for the series of tests for mortar shear strengths should be multiplied by $\frac{1}{0.81}$ to give the effective stress. The author believes that the commonly used procedures in placing the mortar and the shear distribution in the mortar joint are such as to strengthen the point, and that by using a stress reduction factor, more realistic results are obtained.

Checking now the failure of Wall C_1, the ratio of shear strength to shearing stress at failure is $\frac{158}{182} = .87$ which is close to the stress reduction factor proposed earlier. This factor cannot be checked with the results from Walls B_1, B_2 and B_3 due to the adverse effect of the additive.

The distribution of shear in the 3-block test is now discussed in more detail. Figure 9-1 shows a shear distribution which is believed to be more realistic for the 3-block test.

The shear at any point is given by the relation

$$\tau xy = \frac{P}{I}\frac{1}{4}(c^2 - y^2)$$

(Timoshenko, Theory of Elasticity)

P = Load I = moment of inertia

The relation is derived from the relation $\tau xy = \frac{P}{I}\frac{1}{2}(c^2 - y^2)$ which gives the shear for a cantilevered beam loaded by a concentrated load at the free end, by assuming that the 3-block test is equivalent to two cantilevered beams loaded at the unsupported side with a load $\frac{P}{2}$.

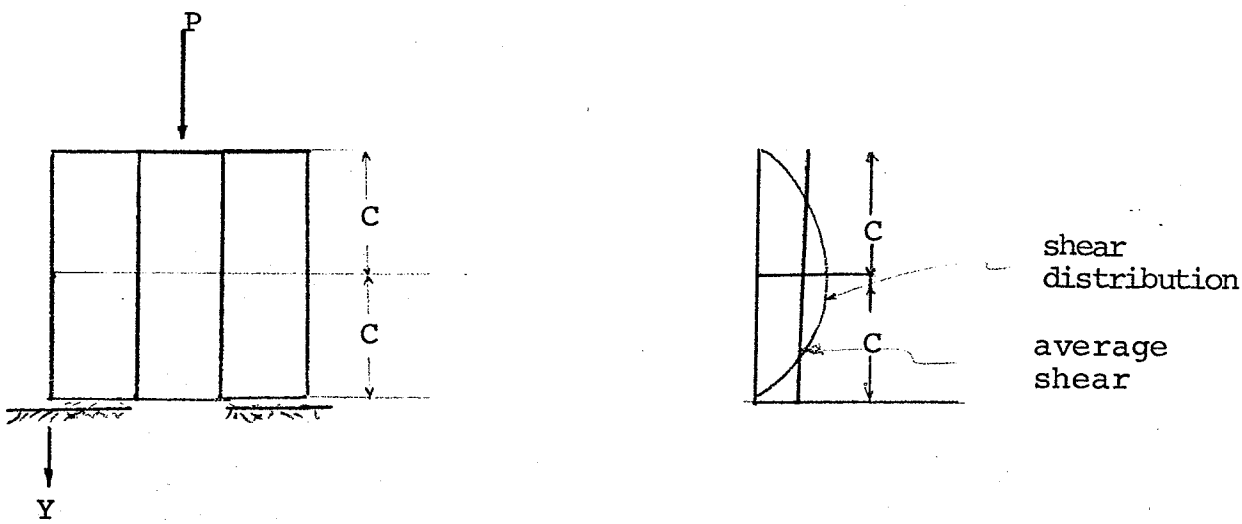


FIGURE 9-1. SHEAR DISTRIBUTION ON 3-BLOCK SHEAR STRENGTH TEST

Although in previous discussion the shear distribution is not taken as parabolic, the shear strength of the mortar at the 3-block shear strength test is larger than the average shear stress. This argement is demonstrated in Figure 9-1.

The capacity of the joint for the walls that failed in shear is greatly dependent on the mortar shear strength. Wall C_1 failed at a load 2.5 times more than Wall A_1, even though Wall A_1 had a vertical stress of 12.3 psi and Wall C_1 had no prestress.

Failure by Cracking

In the analytical work it was stated that tension failure will occur when the tensile stress would reach the value of 100 psi at which time the relation

$$100 = f\tau = \sqrt{\frac{\sigma y^2}{4} + \tau^2} - \frac{\sigma y}{2}$$

would hold and the crack would be at an angle given by the relation

$$\theta^o = \tfrac{1}{2} \tan^{-1} \frac{\tau}{\sigma y}$$

Walls A_4, A_5, B_4 and B_5 failed by cracking. Two possible explanations can be given to these failures.

1. Failure due to tensile stress;
2. Local failure due to bending.

The two possibilities are now examined.

1. Wall A_4

The vertical stress was 210 psi and the shearing stress at failure was 208. The angle at which the crack was expected is given by the relation

$$\theta^{o} = \tfrac{1}{2} \tan^{-1} \frac{\tau}{\sigma y} = \tfrac{1}{2} \tan^{-1} \frac{208}{210}$$

$$= \tfrac{1}{2} \tan^{-1} \; .495 \quad \text{and}$$

$$\theta = \tfrac{1}{2} (45^{o}) = 22.5^{o}$$

Looking at Photo 8-5 and the way the crack is shown, this is a very good explanation of the failure mode, for the crack must be at $90 + \theta = 113^{o}$ with the direction which is very close to the one obtained experimentally. From the two possible explanations of the failure, it is believed that the second, local failure due to bending, is the more probable. However, the fact remains that failure occurred at loads higher than expected.

The finite element analysis for this wall gave an angle for maximum tensile stress of -24 degrees at element 21 which is 114° with the horizontal because of the sign convention used in the program. The tensile stress is given from the relation

$$f\tau = \sqrt{\frac{\sigma y^{2}}{4} + \tau^{2}} - \frac{\sigma y}{2}$$

$$= \sqrt{\frac{210^{2}}{4} + 208^{2}} - \frac{210}{2} = 128 \text{ psi}$$

which is within reasonable limits from the one found experimentally. The same approach used for the other walls gives the following results.

<u>Wall A_5</u>

$$\theta = \tfrac{1}{2}\tan^{-1} \frac{208}{270} = \tfrac{1}{2}\tan^{-1} \; .75 = 18^{o}$$

The crack then should be at right angle with the direction

of the tensile stress, i.e. at $90 + 18 = 108^{\circ}$. Measuring the angle from Photo 8-8, we see a good agreement with the calculated value.

The tensile stress then can be calculated using the formulae

$$f\tau = \sqrt{\frac{\sigma^2 y}{4} + \tau^2} - \frac{\sigma y}{2}$$

$$= \sqrt{\frac{270^2}{4} + 218^2} - \frac{270}{2}$$

$$= \sqrt{18200 + 45700} - 135$$

$$= \sqrt{63900} - 135 = 252 - 135 = 117 \text{ psi}$$

which is very close to the tensile strength found experimentally.

Wall B_4

The angle which the crack made with the horizontal is 105 degrees, and the angle calculated in the theoretical analysis is $\Theta\ \frac{1}{2}\tan^{-1}\left(\frac{\tau}{\sigma y}\right) = \frac{1}{2}\tan^{-1}\frac{176}{210} = \frac{1}{2}\tan^{-1} .838$

$\Theta = 20.0^{\circ}$.

The crack then should be at $90^{\circ} + 20^{\circ} = 110^{\circ}$ with the horizontal which is almost as measured in the experiment.

The tensile stress at failure was according to the theoretical approach

$$f\tau = \sqrt{\frac{\sigma y^2}{4} + \tau^2} - \frac{\sigma y}{2} = \sqrt{\frac{210^2}{4} + 176^2} - 105$$

$$= 210 - 105 = 105 \text{ psi}$$

remarkably close to the tensile strength found in the tension test.

Wall B_5

Using the same assumption as previously used, the tensile stress on the wall can be found as

$$f\tau = \sqrt{\frac{270^2}{4} + 188^2} - \frac{270}{2}$$

$$= \sqrt{53900} - 135 = 232 - 135 = 97 \text{ psi}$$

2. A possible explanation of the failure is the following. The blocks failed due to bending at the center of the loaded face and after this had occurred, the side of the blocks failed. Assume that the blocks are loaded as shown in Fig. 9-2.

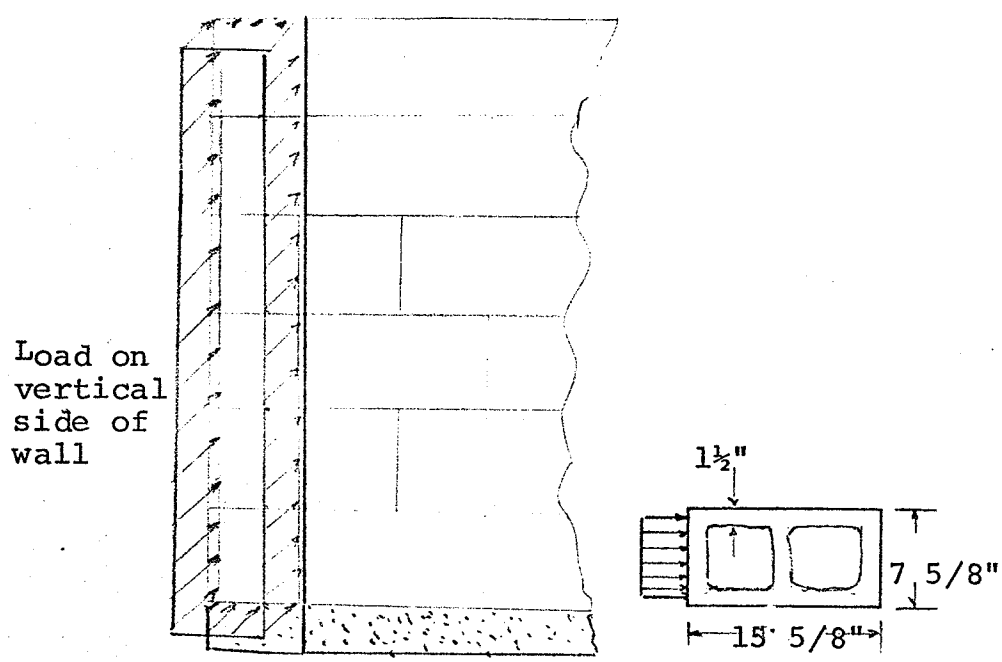


FIGURE 9-2. ASSUMED BEHAVIOUR OF END SECTION

For Wall A_4 the total load was 39,600 kips, i.e. $\frac{39,600}{48} = .825$ k/in. This load uniformly distributed over the 7.725" of the cross-section of the wall will give a moment of

$$\frac{WL^2}{12} = \frac{.825 \times 4.725^2}{7.725 \times 12} = .1985 \text{ k/in.}$$

The stresses due to this bending are

$$\sigma y = \frac{My}{2}$$

$$= \frac{.1985 \times 175 \times 12}{1.5^3} = .53 \text{ ksi}$$

The assumptions made here are the following:

a) thickness 1.5"

b) distance between supports 7.725 - 2 x 1.5 = 4.625"
1.5 being the thickness of the sides of the block

c) the assumed beam is fixed at the ends.

The results of this calculation show high tensile strength but the thickness of the block varies from 2" on the upper part to 1.25" on the lower - therefore lower stresses on the upper part existed. The fact that the failure was sudden strengthens this point. Also the bending resistance of the steel plate is not taken into consideration here. A more accurate analysis is, therefore, required.

A portion of the blocks shown schematically in Figure 9-3 was analyzed by the computer using the finite element technique.

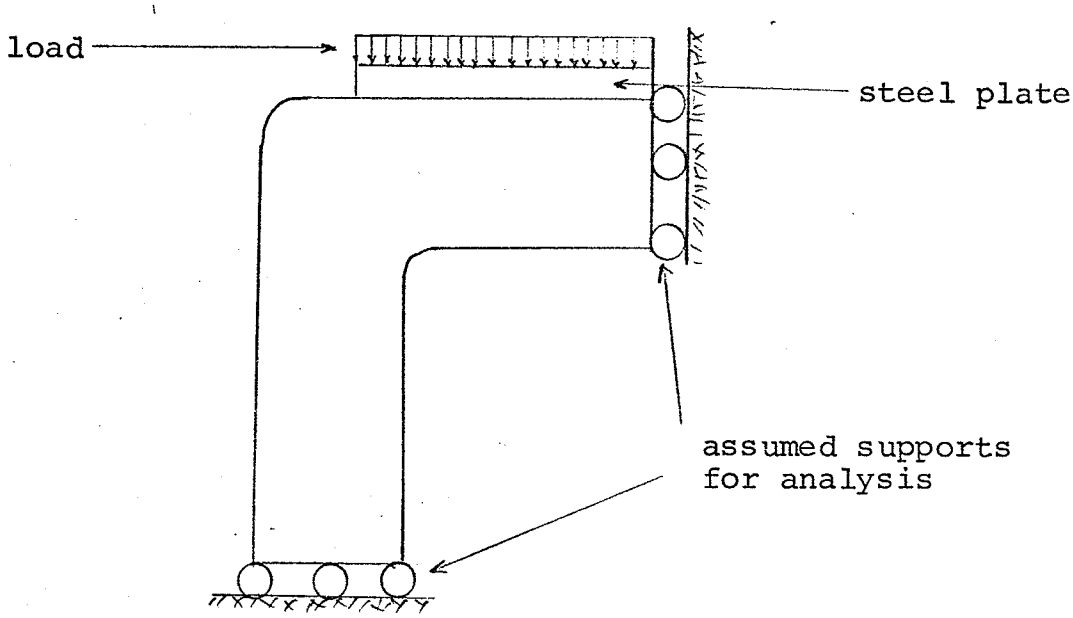


FIGURE 9-3. ANALYZED BLOCK PORTION

The results of this analysis are given in Table 9-1. Figure 9-4 shows the elements and nodes into which the section was subdivided for the analysis.

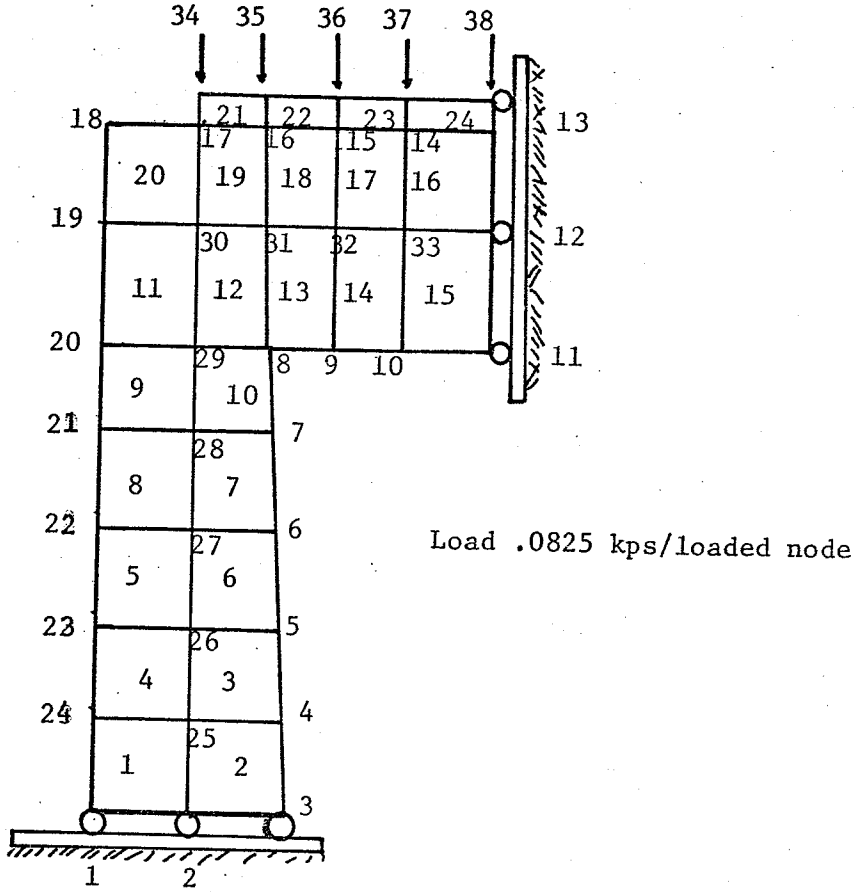


FIGURE 9-4. ELEMENT AND NODES ANALYZED.

TABLE 9-1. STRESSES AT SELECTED NODES.

Node	X-Stress	Y-Stress	Max-Stress	Direction
10	0.0868	-0.0654	0.1309	-25.3714
11	0.0516	-0.0533	0.1426	-34.2756
12	0.0259	-0.0122	0.1540	-41.2822
20	-0.0193	-0.2052	0.0243	23.5536
21	-0.3458	-0.2666	0.0455	-41.7696
22	-0.5555	-0.1247	0.0024	-25.5205
23	-0.6139	-0.0801	-0.0752	-5.4430

From this analysis it is now most probably that the walls which failed in this manner failed in the following sequence. The moment produced by the applied load caused high stresses (tensile) on the inside face of the block (Point A). At a certain point the section failed in tension, which caused large tensile stresses on the outside of the block (Points B and B^1). As the load increased, the block deflected as shown in Photo 9-1 with resulting tensile failure at the other end of this and the following block. Figure 9-5 illustrates sequence of failures.

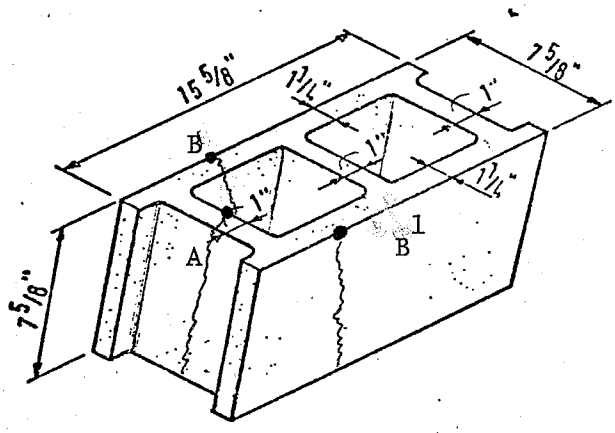


FIGURE 9-5. FAILURE STAGES OF LOADED BLOCKS

The failure of Wall B_4 is believed to be a failure due to diagonal tension. The tensile stress at this failure was calculated to be 105 psi, which means that when the local failures occurred, the walls were at impending failure by diagonal tension. Although this is to be verified by further investigation, as recommended in the section "Recommendations", from this study it is found that the walls, even with the local failures, can take a load as large as that calculated by the analysis, using the relation

$$f\tau = \sqrt{\frac{\sigma y^2}{4} + \tau^2} - \frac{\sigma y}{2}$$

It is observed from the experiments that the shear stress of the mortar is not of great importance when the vertical stresses are high. In this case, failure will occur by other shear mechanisms, so the strength of the mortar is not utilized.

Examining the deflection curves of the walls, one can very clearly see that the walls built using mortar produced by normal cement and polyvinyl acetate deflected more, before failure, than the other walls. This is due to the plasticizing effect of the additive.

Let us now look into the code requirements and allowable shearing stress for masonry constructed shear walls, and how the allowable shear compares with the experimentally found capacity.

The 1970 Canadian National Building Code[2] states "The maximum horizontal shear stress in a shear wall, Vsw, shall

not exceed the value Vsw = Vm + -.3 fcs

where Vm = the allowable applicable shear strength

fcs = compressive stress due to vertical loads."

Article 4.3.3.33.4 of the Supplement No. 4 to the National Building Code of Canada.

A plot of this relation is given in Figure 9-6. In the same figure, the experimental results of Group A are plotted. From this graph, it is observed that the allowable shear as given in the Code gives a low safety factor at low vertical stresses and this factor increases with increasing vertical stress. It is believed by the author that for concrete blocks and mortars produced using masonry cement or normal cement the frictional factor μ is at least 0.70, (the lowest found in literature was 0.72[6]). If μ is known with precision, then the frictional contribution in the resulting lateral forces may be predicted with great accuracy. The use of a low value of μ (0.30) implies that the ability of a masonry wall to carry lateral loads is underestimated. The San Fernando Earthquake showed this to be the case. Observing Figure 9-1, it is clear that after a vertical stress of approximately 190 psi, the shear capacity is almost constant. After this point shear failure is not possible, i.e. the wall will fail in another mode. However, the experimental data indicates that the allowable shear stress should not be greater than $4\sqrt{f^1c}$ psi in order to prevent local or diagonal tensile failures from occurring.

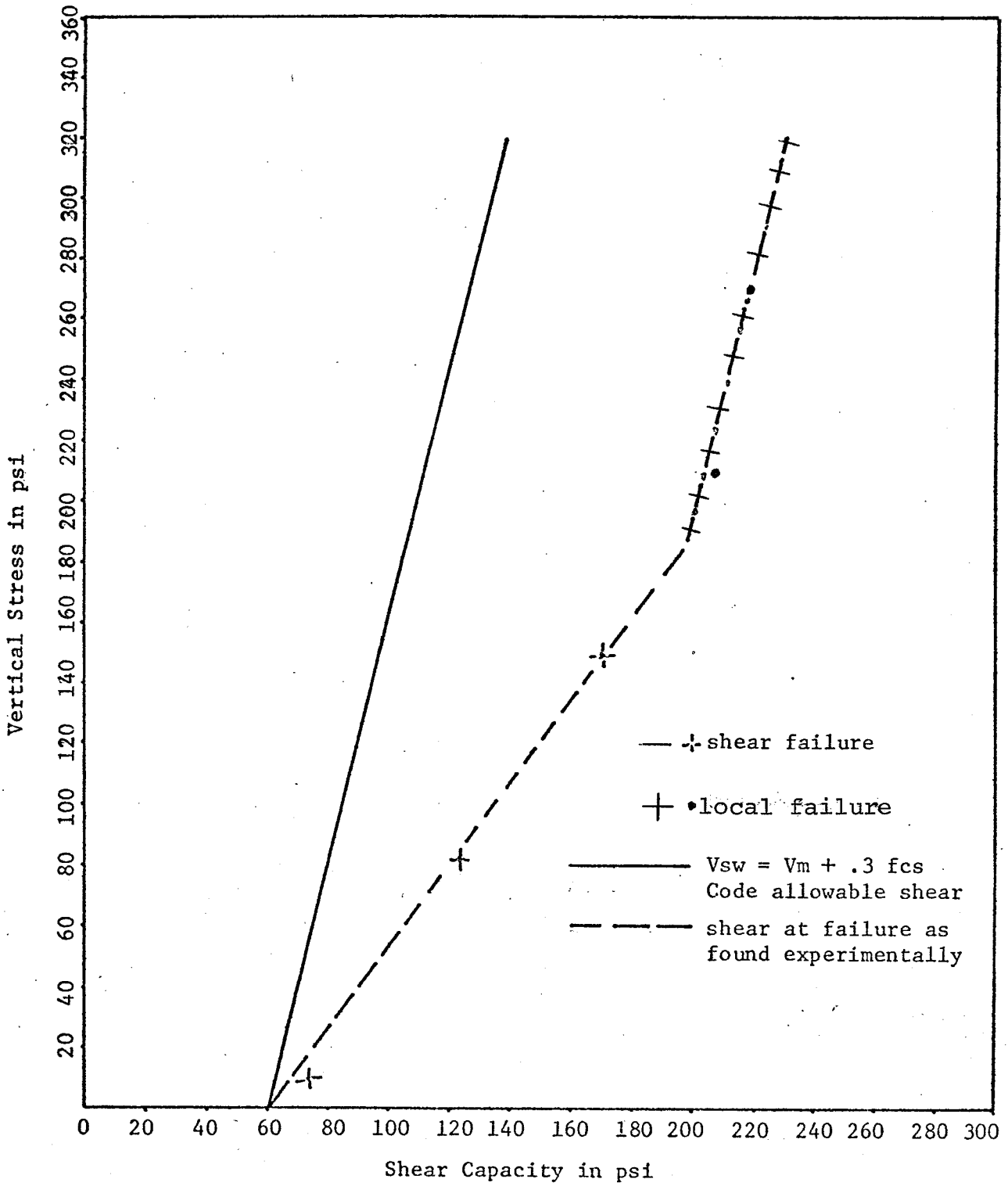


FIGURE 9-6. RELATION BETWEEN VERTICAL STRESS AND SHEAR CAPACITY

Let us now look into the San Fernando Earthquake. It is reported by Mr. Norman B. Green in the May 1972 ACI issue,[3] that the load imposed on the structures was in the order of 0.83g. Masonry constructed buildings performed exceptionally well. This is explained by the fact that the ability of the wall sections to carry this load is a function of the vertical stress and the friction factor between blocks and mortar which can very easily be as large as 0.85.[7] In other words, the strength of the walls in shear was almost equal to the horizontal force (excluding the contribution of the mortar). The great damage on relatively light buildings in the area, and the condition of the heavier buildings show that the structures were at the edge of failure and that if the intensity of the earthquake was in the order of .90g, greater damage was unavoidable. Photos 9-2, 9-3 and 9-4 show three views of the San Fernando Earthquake. Note the similarity of the failure with those of Walls A_4 and A_5, although the loading on Walls A_4 and A_5 is not exactly the same as that induced on the buildings shown in Photos 9-2, 3 and 4. The failure shown in Photo 9-4 is believed to be a compressive one.

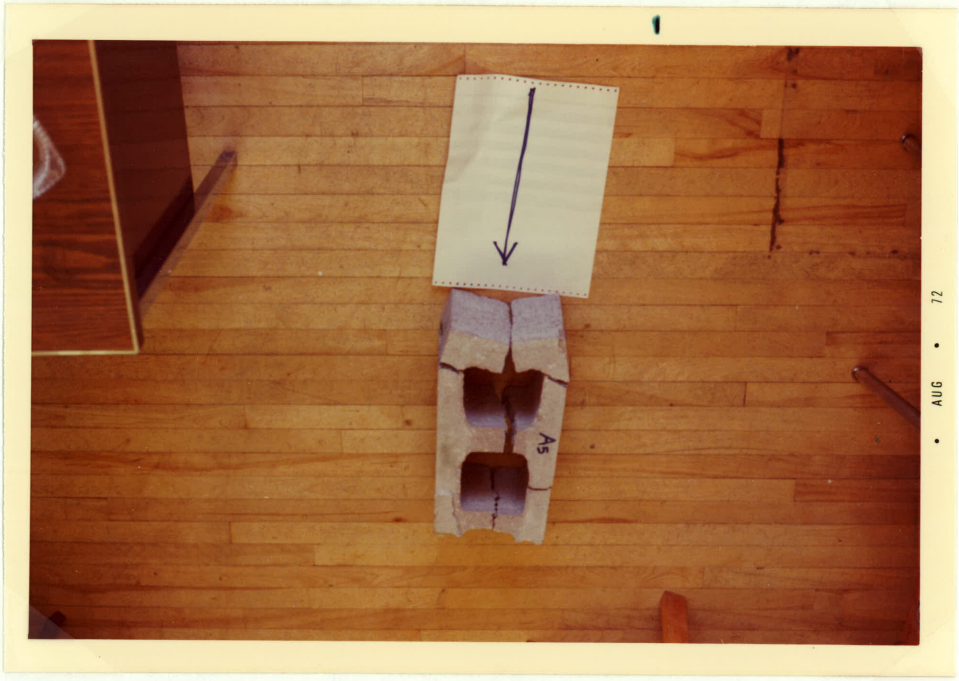


Photo 9-1. Failure mechanisms of loaded block.

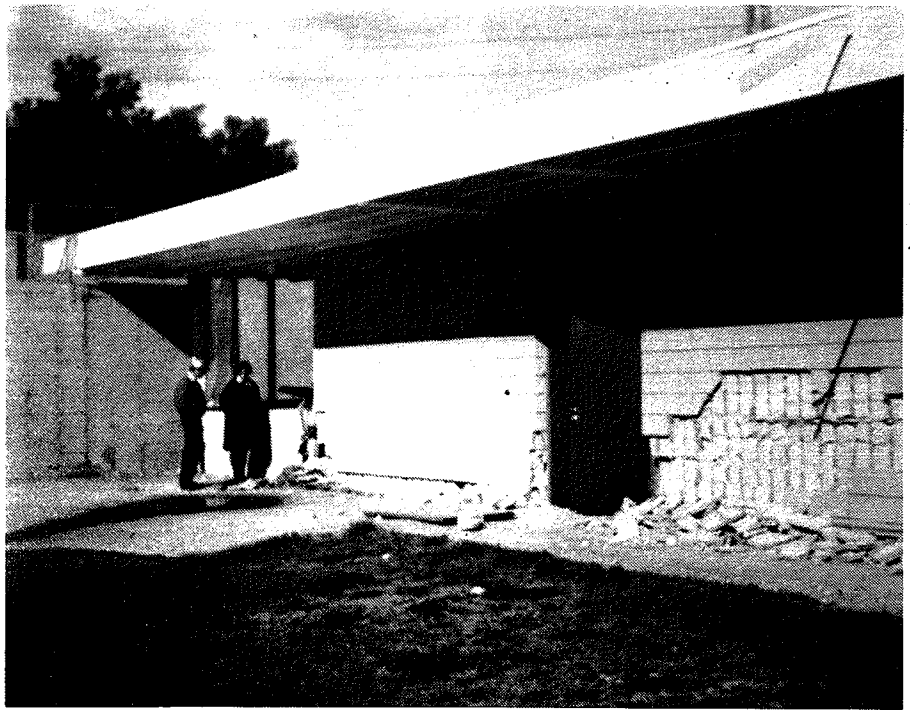


Photo 9-2. Damage to light building in San Fernando Earthquake.

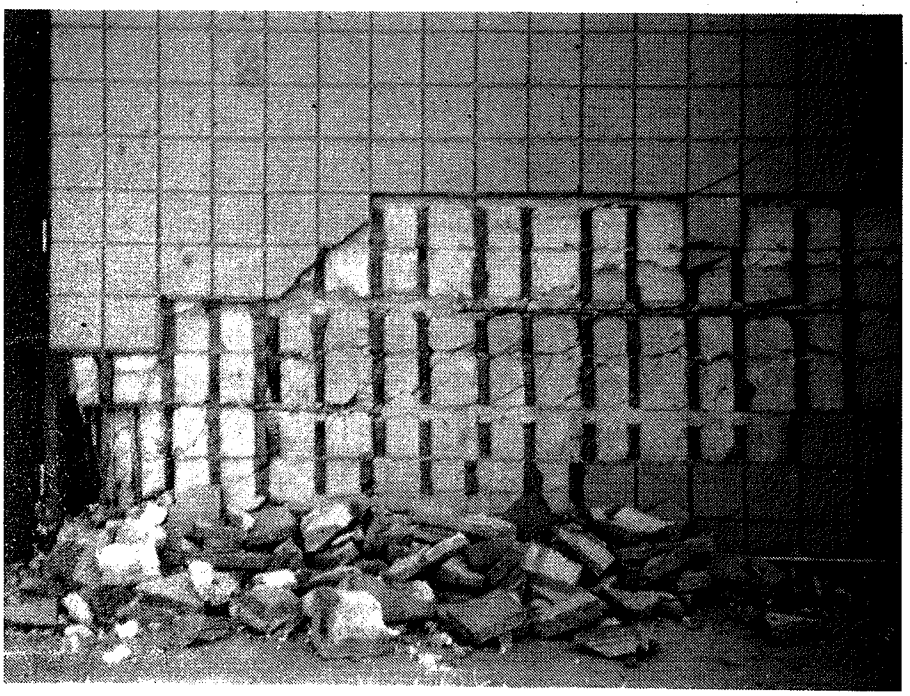


Photo 9-3. View of earthquake damage. Note the separation between blocks and concrete fill.

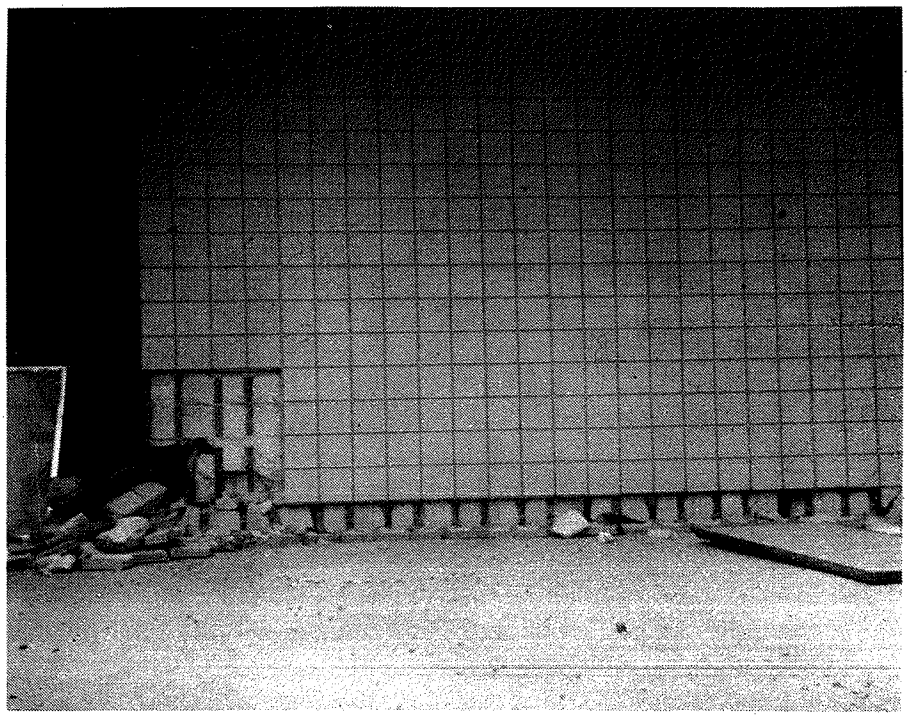


Photo 9-4. The damage in this picture is similar to the failure of Wall A_4.

Note: Although there is a similarity in Photos 9-4 and 8-7, the loading of the walls is not exactly the same.

In studying the strains obtained from the strain gauges, it is observed that at certain points the blocks are almost free of strain, or the strains are decreased despite the increase in the horizontal load. The same observation can be made with deflections and with the strains that can be found using the reading of dials 6 and 5.

It is believed that most deformations take place in the joints, and the spring-like behaviour mentioned before in this study seems to be a valid one. However, a detailed analysis of these results is beyond the scope of this study.

An explanation of the behaviour is given. If both the mortar and the blocks follow Hooke's law, then the total deformation for one block and one joint is

$$\Delta T = \Delta T_B + \Delta T_j \quad \text{when}$$

ΔT = total deformation

ΔT_B = deformation of block

ΔTj = deformation of mortar joint

If the modulus of elasticity of the blocks is E_B and that of the mortar is E_j then the block and the joint have different deformations (due to E and size). When the capacity of the joint is exceeded then the joint deforms much further beyond the strain in the block with the effect of reducing the stresses in the block.

The finite element analysis in masonry constructed walls can be of great usefulness. The results obtained in analyzing Group A (deflections and stresses) show that the analysis gives very high stresses and deflections which are a fraction of the ones measured. It is believed that this is because of the modulus of elasticity used in the analysis. It was assumed that E = 3000 ksi. This is not the case. Let us try to find a relation between E_B and Ej where E_B = modulus of elasticity of block and Ej = modulus of elasticity of mortar.

It was stated previously that

$$\Delta T = \Delta T_B + \Delta Tj$$

$$\Delta Tj = \varepsilon j Tj$$ where εj = strain in joint

Tj = thickness of joint

Using Hooke's law

$$\sigma = E\varepsilon \quad \text{then} \quad \Delta Tj = \frac{\sigma}{Ej} Tj$$

If Em = modulus of elasticity of masonry wall and T = equivalent thickness of the wall and T_B = thickness of the block then

$$\frac{\sigma}{Em} T = \frac{\sigma}{E_B} T_B + \frac{\sigma}{Ej} Tj$$

from which the modulus of elasticity can be found by setting

$$\frac{Tj}{Tb} = \gamma \text{ and } \frac{Ej}{Eb} = \beta$$

$$\frac{Em}{Eb} = \frac{1 + \gamma}{1 + \gamma/\beta}$$

This indicates that the modulus of elasticity of the

masonry constructed wall is a function of many factors that vary from wall to wall. And it is believed to be the reason for the results of the finite element analysis and the strain gauges being difficult to relate.

CHAPTER X

Conclusions

1. The shear strength of the mortar is very important when low vertical stresses exist. For high vertical stresses, the contribution of the mortar is small compared with the frictional contribution.

2. Mortar produced by normal cement and polyvinyl acetate has greater flexibility and larger deflections are measured before failure. This is an important aspect since greater energy can be absorbed by larger deformations.

3. The shear capacity of unreinforced masonry walls can be obtained using the relation

$$V = Vo + \mu\sigma y$$

4. Failure of the wall will not occur until the relation

$$f\tau = \sqrt{\frac{\sigma y^2}{4} + V^2} - \frac{\sigma y}{2}$$

is at least 100 psi.

5. The code allowable shear formula for unreinforced masonry walls

$$Vsw = Vm + 0.3\ fcs$$

underestimates the shear capacity by at least .4 fcs, since it uses 0.3 fcs instead of at least 0.7 fcs.

6. The maximum shear that can be carried out before local or general failure in tension occurs, is at least $4\sqrt{f'c}$,

provided that vertical stress is high enough (over 180 psi). However, more test data is needed.

7. Finite element analysis gives very high stresses, and is useful for finding average stresses. However, deflections are much less than the measured ones. The modulus of elasticity used in this analysis appears to be very high.

8. Strain gauges as means of calculating strains and stresses on the lbocks give results not easily related to those obtained by finite element because of the effects of local variations of material property and lack of homogeneity of wall.

9. No conclusions can be drawn for filled walls because only one was tested.

CHAPTER XI

Recommendations

1. High shear strength mortar should be used when the vertical loads are small. Normal cement with 10% polyvinyl acetate gives a shear strength of 150 psi at 28 days and is recommended for use after further studies.

2. When mortar with high shear strength is used, and where shear stresses are high, it is recommended that grooves be cut out of the block to better utilize the strength of the mortar.

3. It is recommended that the shear strength of the wall is obtained using the least of the values obtained from

$$V = Vo + \mu\sigma Y \quad \text{with } \mu = 0.70 \qquad (1)$$

$$f\tau = \sqrt{\frac{\sigma y^2}{4} + V^2} - \frac{\sigma y}{2} \quad \text{with } f\tau = 2\sqrt{f'c} \qquad (2)$$

$$V \leqslant 4\sqrt{f'c} \qquad (3)$$

4. It is recommended that allowable shear for masonry constructed unreinforced shear walls and connections with slabs be increased if they are similar to those tested. The amount of increase should be determined by further testing.

5. It is recommended that a study be carried out using the finite element approach to find the appropriate elastic constants for unreinforced masonry walls, and a further study to determine the usefulness of strain gauges as a

means of understanding the movements of the blocks within the wall, since it is believed, from the results obtained from the limited investigation in this area, that movements take place within the wall.

6. It is recommended that in similar tests, the loaded face block is filled to avoid local failures, or if not filled, two plates (1" x 1.5") should be placed on the sides of the loaded face in order that the applied load is transferred to the wall without causing bending of the end section. Also the plate on which the loads are applied should be 1 inch thick so that bending of the plate would be largely eliminated.

7. It is also recommended that a large number of tests be carried out in order to propose changes in the allowable shear stress on similar walls.

8. It is recommended that the effect of vertical reinforcement in the shear strength capacity of the wall be investigated.

9. It is recommended that the shear capacity of filled walls with and without vertical and horizontal reinforcement be investigated.

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APPENDICES

TABLE OF CONTENTS

APPENDIX A

	Page
Title Page	140
List of Tables	141
INTRODUCTION	142
MORTARS AND TEST METHODS	143
MORTAR I	144
MORTAR II	145
MORTAR III	146
CONCLUSIONS AND RECOMMENDATIONS	149

APPENDIX A

MORTARS AND CONCRETE DATA

LIST OF TABLES

APPENDIX A

		Page
A-1	Shear Strength of Mortar	144
A-2	Shear Strength of Mortar	145
A-3	Shear Strength of Mortar with Normal Cement, 5% Additive	147
A-4	Shear Strength of Mortar with Normal Cement, 10% Additive	147
A-5	Shear Strength of Mortar with Normal Cement, 15% Additive	148
A-6	Shear Strength of Mortar with Normal Cement, 20% Additive	148
A-7	Compressive Strength of Mortar at 28 Days	151
A-8	Compressive Strength of Contrete Used in Slabs and Filling the Walls	152
A-9	Compressive Strength of Blocks	153
A-10	Seive Analysis of Sand	154

INTRODUCTION

The mortar in masonry construction is used to bind the individual masonry units together into a single structure whose strength characteristics can be predicted, and also to provide a seal which will protect the joint from penetration by air and moisture. Many problems such as cracking and rain penetration have been directly attributed to the use of unsuitable mortar. The mortar is also required to bond with steel reinforcement, ties and anchor bolts so that they perform integrally with the masonry. There are many types of mortars each with different characteristics. The most common mortars are the following five types M, S, N, O, K with average compressive strengths respectively at 28 days of 2500, 1800, 750, 350 and 75 psi. (Sven Sahlin, Structural Masonry, 1971.)

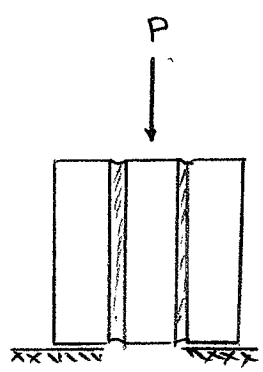
The most important property of hardened mortar is bond, which holds the masonry units together. The materials now available have a very poor bond strength. It has been proved experimentally that mortar having a compressive strength of 2500 psi could develop a bond strength of only 50 to 100 psi. Stresses causing bond failure develop when masonry is subjected to eccentric gravity loads, loads from winds and other lateral forces.

MORTARS

In this study, three types of mortars were used. The water-cement ratio was the same in all three, with the only difference being a polyvinyl acetate emulsion which was used as an additive. The weight of the additive was subtraved from the amount of water to keep w/c ratio constant and equal to 0.55.

TEST METHOD

The specimens were tested by applying direct force in the middle block while the other two blocks rested as shown in the figure below.



Bond or Shear capacity at failure = $\frac{P}{2A}$ where A = area of contact

$= 7\frac{5}{8} \times 3\frac{5}{8} = 27.75 \text{ in}^2$

P = load at failure

The size of the blocks was 2" x 4" x 8". (Nominal)

Mortar 1.

Materials: 1:4 masonry cement to sand

w/c = 0.55

no additive

TABLE A-1. SHEAR STRENGTH OF MORTAR I.

Age in Days	Load at Failure in lb.	Average Load in lb.	Average Shear Strength in psi
7	2200 1650 3740	2530	46
14	3100 3550 900	2512	45
28	3120 2500 1500	2706	48.5

Mortar 2.

Materials: 1:4 masonry cement to sand

w/c =0.55

10% by cement weight additive

TABLE A-2. SHEAR STRENGTH OF MORTAR II.

Age in Days	Load at Failure in lb.	Average Load in lb.	Average Shear Strength in psi
7	1800		
	2950	2833	51.0
	3750		
14	7400		
	4650	6333	113.5
	6950		
28	2760		
	3380		
	2890		
	3290		
	2160	3190	54.5
	3830		
	3480		
	1900		

Mortar 3.

Materials: 1:4 cement to sand

w/c = 0.55

5, 10, 15, 20% of additive was tried.

The shear strength of mortars produced using the above combination of materials is given on the following tables. From previous experiments conducted by the author, it was found that the shear strength of the mortar increased with increasing amount of additive However, the w/c ratio has to be decreased at higher additive content. The mortars produced were on the wet side, but in order to have consistancy the w/c ratio was kept at 0.55. The presented results are conservative since the quality of the blocks was poor, resulting in some cases of block failure.

TABLE A-3. SHEAR STRENGTH OF MORTAR WITH NORMAL CEMENT, 5 PERCENT ADDITIVE

Age in Days	Load at Failure in lb.	Average Load in lb.	Average Shear Strength in psi
7	2500		
	6000	4600	82.5
	3300		
14	7400		
	6500	6166	110
28	7000		
	6700	6666	120
	6300		

TABLE A-4. SHEAR STRENGTH OF MORTAR WITH NORMAL CEMENT, 10 PERCENT ADDITIVE

Age in Days	Load at Failure in lb.	Average Load in lb.	Average Shear Strength in psi
7	6640		
	4360	4793	86
	3380		
14	6500		
	9200	8420	151
	9560		
28	7500		
	9700	8833	158
	9300		

TABLE A-5. SHEAR STRENGTH OF MORTAR WITH NORMAL CEMENT, 15 PERCENT ADDITIVE

Age in Days	Load at Failure in lb.	Average Load in lb.	Average Shear Strength in psi
7	8320		
	5700	6373	114
	5100		
14	10000		
	8700	11400	205
	15500		
28	9250		
	9500	9316	167
	9200		

TABLE A-6. SHEAR STRENGTH OF MORTAR WITH NORMAL CEMENT, 20 PERCENT ADDITIVE

Age in Days	Load at Failure in lb.	Average Load in lb.	Average Shear Strength in psi
7	4500		
	5520	5473	98.5
	6400		
14	13000		
	9500	11000	197.5
	10500		
28	8750		
	12750	11250	202
	12250		

CONCLUSIONS AND RECOMMENDATIONS

From the series of experimental investigations it is believed that:

1. Addition of polyvinyl acetate, a product of the Swift Chemical Company with the trade number 52-81, increases the bond strength of mortar produced using normal cement.

2. Workability of the mortar with additive is good. The mortar was found to be in good condition, as far as workability is concerned after sitting untouched for two hours.

3. Water seems to have no effect on the hardened mortar. However, it is recommended that further and more extensive investigation be made regarding such aspects as: optimum additive and cement ratio, heat effects, fire resistance, moisture penetration, etc.

4. The additive decreases the shear strength of mortars produced, using masonry cement.

5. Polyvinyl acetate reduces the amount of water required to produce a workable mortar, which is an advantage for strength since the lower the water content the higher the strength.

6. Additive increases compressive strength by approximately 45%.

Recommendations

1. The use of polyvinyl acetate is recommended only as an additive with normal cement to increase shear strength. However, addition of the material in mortars produced using masonry cement must be avoided for the shear strength decreases with time after approximately 16 days.

2. Further investigation into the properties of mortar produced using normal cement and polyvinyl acetate is recommended.

TABLE A-7. COMPRESSIVE STRENGTH OF MORTARS AT 28 DAYS

imin	Masonry Cement No Additive		Masonry Cement 10% Additive		Normal Cement 10% Additive		Comments
	Load in lb.	Average Strength psi	Load in lb.	Average Strength psi	Load in lb.	Average Strength psi	
	19000		26300		27100		
	15000		23800		23800		
	15500		22900		23100		
	18500		25100		22100		
	14300		22000		21800		
	20800	2150 psi	20250	3320 psi	24900	3340	
	11000		25050		23000		
	8000		23800		21600		
	17900		21900		23900		
	19000		22600		23000		
	11000						
	19000						
	17000						
	13950						

The test cylinders were 3 inches in diameter and 6 incher in height.

$$\text{Area} = \Pi r^2 = 3.\frac{14 \times 3^2}{4} = 7.05\text{in}^2$$

TABLE A-8. COMPRESSIVE STRENGTH OF CONCRETE USED IN SLABS AND FILLING THE WALLS

Cylinder	Load in lb. at	Average Strength in psi
1	118000	
2	100000	
3	121000	
4	104000	
5	103000	2660
6	68000	
7	88000	
8	113000	
9	98500	
10	103500	

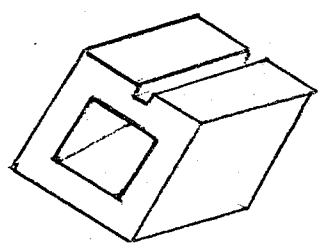
The test cylinders were 6 inches in diameter by 12 inches high.

$$\text{Area} = \pi r^2 = \frac{3.14 \times 6^2}{4} = 28.3 \text{ in}^2$$

TABLE A-9. COMPRESSIVE STRENGTH OF BLOCKS

No.	Area in in^2	Load in kips	P/A k/in^2	Average Strength in lb/in^2
1	34.58	103	2.98	
2	- " -	104	3.02	
3	- " -	110	3.2	2480
4	- " -	53	1.54	
5	- " -	58	1.68	
6	- " -	86	2.5	

Dimensions of tested block



Groove 3/4 x 3/4

Contact area = 7 1/2 x 7 1/2 - 4.1 x 5.1 - 3/4 x 3/4

= 34.58 in^2

TABLE A-10. SIEVE ANALYSIS OF SAND

Sieve	Wt. Retained	% Retained	% Retained Accumulative	% Passing Accumulative
1½	0	0	0	100
3/4	0	0	0	100
3/8	0	0	0	100
4	38	3.96	3.9	96.1
8	111	10.6	14.5	85.5
16	149	15.5	30.0	70.0
30	246	25.6	55.6	44.4
50	266	27.8	83.4	16.6
100	130	13.6	97.0	3.0
200	16	1.7	98.7	1.3
Pass 200	4	.4	99.1	.9

$$\text{Fineness modulus} = \frac{\Sigma \text{ \% accumulative retained}}{100}$$

$$= \frac{1}{100}\,[3.9 + 14.5 + 30.0 + 55.6 + 83.4 + 83.4 + 97.0] = 2.84$$

Note: Sieve No. 200 is not included in the fineness modulus.

APPENDIX B

TENSILE STRENGTH OF BLOCKS

TABLE OF CONTENTS

APPENDIX B

		Page
Title Page		155
List of Tables		
B-1	Calculation of Tensile Strength of Blocks	158
List of Photographs		
B-1	Tensile Strength Specimen During Test	159
B-2	Specimen After Tensile Failure	159

Calculation of Tensile Strength of Blocks

To determine the tensile strength of the blocks, ten specimens of three blocks were prepared using normal cement and 10% polyvinal acetate as an additive. (Photo B-1 illustrates specimen). Holes were made using a drill, on both sides of the specimen and using two plates to distribute the load, tension tests were made as shown in Photo B-1.

All specimens failed in tension on the blocks, and the area of the failure surface is calculated by measuring. A sample calculation is shown below.

$$\text{area} = 7.6\ [\frac{2+1.8}{2} + \frac{1.3+1}{2}]$$

$$= 7.6 \times [\frac{3.8+2.3}{2}] = 7.6 \times [\frac{6.1}{2}] = 7.6 \times 3.05 = 23.2$$

$$\text{tensile strength} = \frac{2370}{23.2} = 102 \text{ psi}$$

Although the failure cannot be stated as being 100% due to tension, it is believed that it gives a good indication of what the tensile strength of the blocks was.

TABLE B-1. CALCULATION OF TENSILE STRENGTH OF BLOCKS

Specimen	Load at Failure	Area	Average Strength	Comments
1	2370	23.2		
2	2350	23.2		
3	2400	23.2		
4	2300	23.2	102	
5	2380	23.2		
6	2230	23.2		
7	2420	23.2		
8	2400	23.2		
9	2370	23.2		
10	2480	23.2		

The results agree remarkably well with the assumption made in the core of this thesis that the tensile strength of the concrete was $f_t = 2\sqrt{f'_c} = 2\sqrt{2480} \cong 100$ psi.

It is also worthwhile mentioning here that the tensile strength of the mortar is large enough to make the possibility of building wall sections and then lifting them into place, a consideration to be looked into further.

Photo B-1 Tensile strength specimen during test.

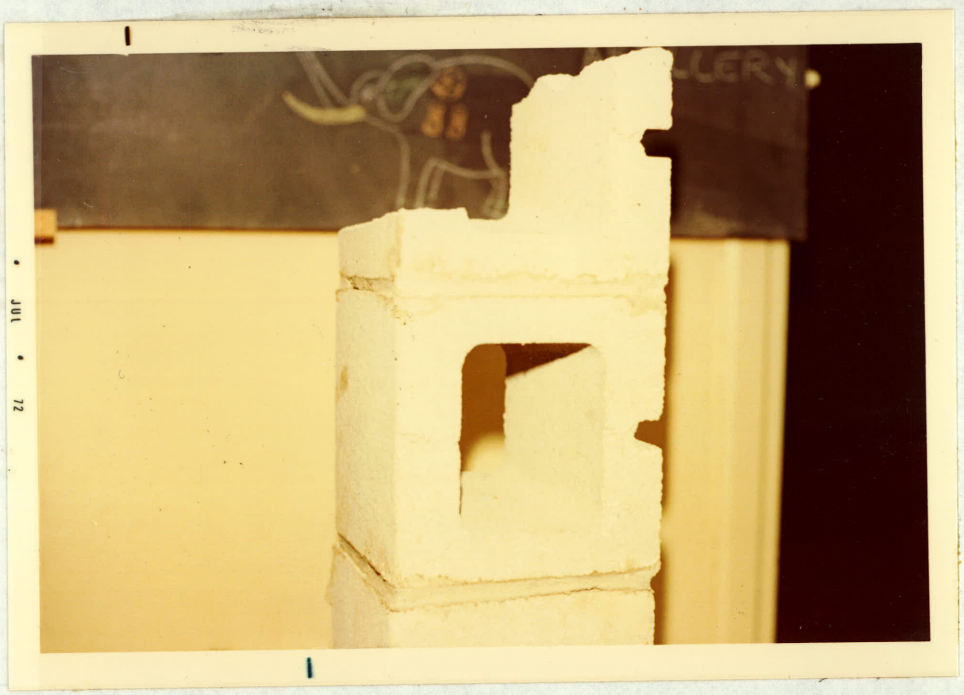


Photo B-2 Sepcimen after tensile failure.

APPENDIX C

ROLLING RESISTANCE

TABLE OF CONTENTS

APPENDIX C

		Page
Title Page		160
List of Tables		
C-1	Rolling Resistance	164
List of Figures		
C-1	Rolling Resistance Test	162
C-2	Rolling Resistancs versus Vertical Load	163

ROLLING RESISTANCE OF ROLLERS.

The horizontal loads must be corrected for the rolling resistance. For this purpose, the rollers used on the top of the wall were placed in the same manner as in the tests, using the same materials, under a vertical loading appartus and the rolling resistance at various vertical loads was found. The horizontal loads then were adjusted and thus the actual horizontal load on the wall was determined. Table C-1 and Figure C-2 give the results of this work.

To reduce the folling resistance as much as possible all plates were kept clean and the rollers were lubricated and cleaned after every test.

Figure C-1 illustrates qualitatively the procedure used in this test.

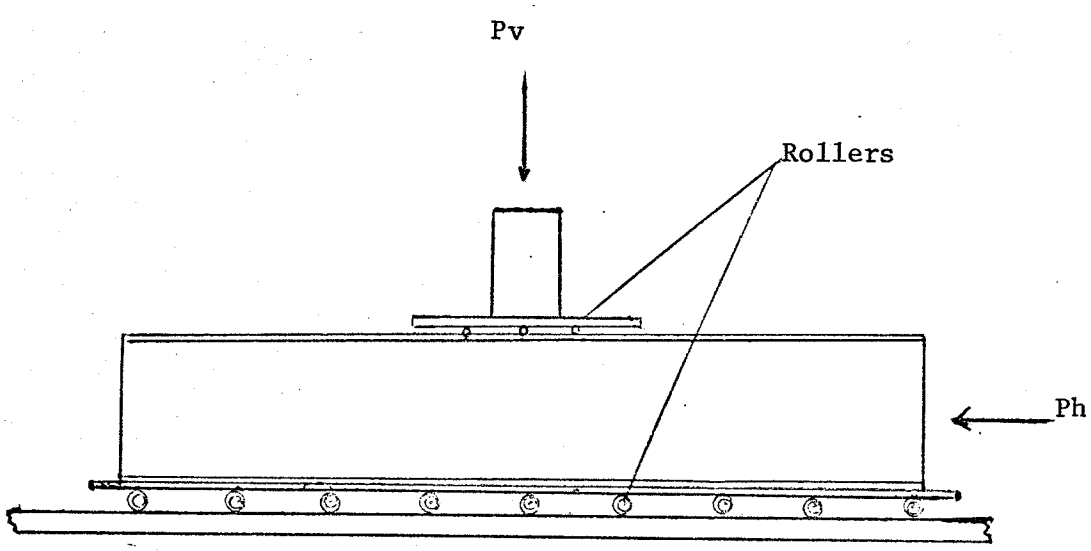


Figure C-1. Rolling Resistance Test

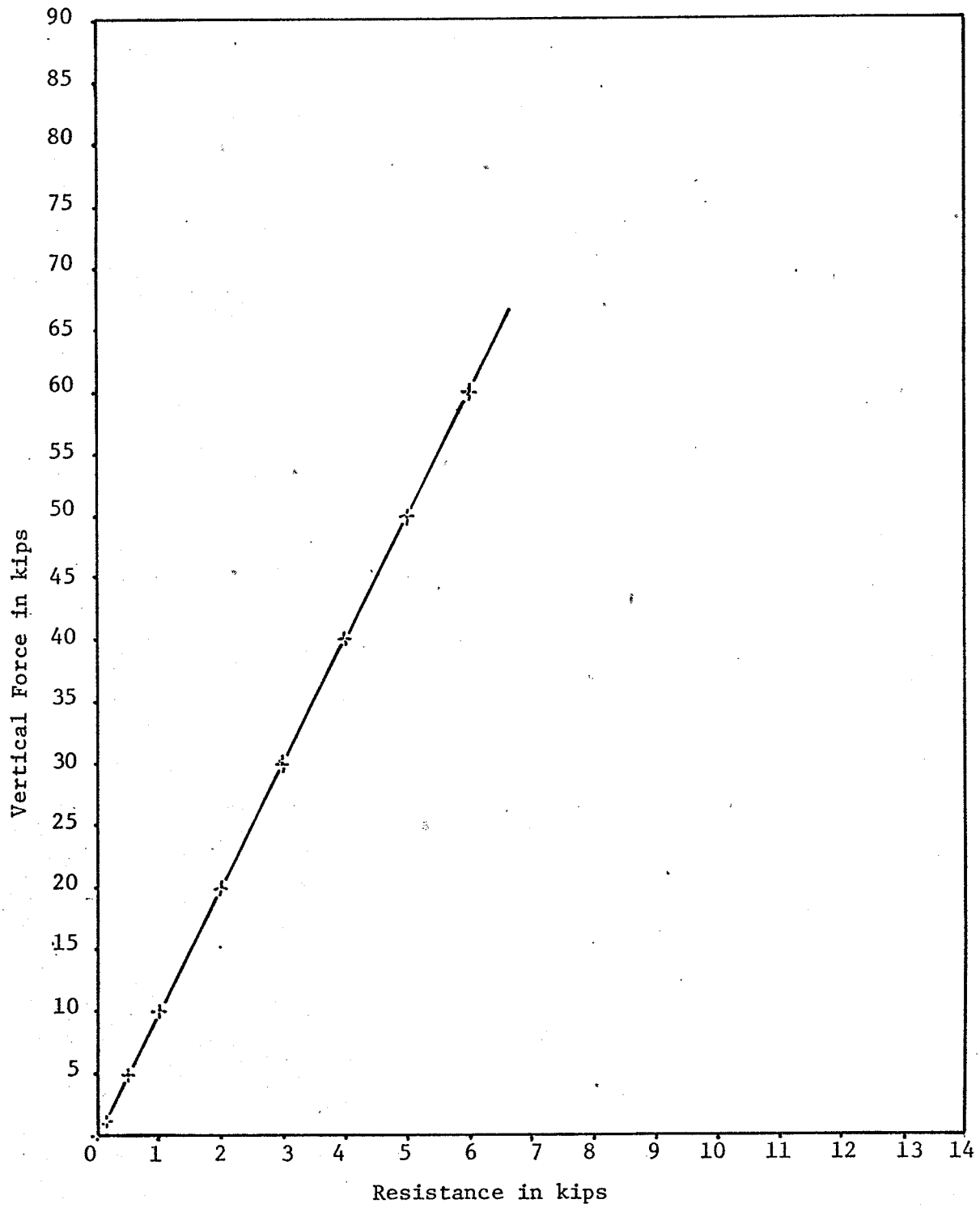


FIGURE C-2. ROLLING RESISTANCE IN VERTICAL LOAD

TABLE C-1. ROLLING RESISTANCE

Vertical Load in Lb.	Resistance in Lb.
1,000	96 ≈ 100
2,000	187 ≈ 200
4,000	374 ≈ 400
10,000	1050 ≈ 1000
15,000	1460 ≈ 1500
20,000	2060 ≈ 2000
25,000	2453 ≈ 2500
30,000	3000
35,000	3500
40,000	4000
45,000	4500
50,000	5000
55,000	5500
60,000	6000

Note: Only up to 25,000 lb. vertical load was tested. The rolling resistance for higher vertical loads were calculated using .10 of the vertical load.

APPENDIX D

RESULTS FROM STRAIN GAUGES

TABLE OF CONTENTS

APPENDIX D

		Page
Title Page		165
List of Tables		
D-1	Results of Strain Gauges (Strains in inches per in x 10^4) Wall A_1	167
D-2	Table D-1 Continued	168
D-3	Results of Strain Gauges - Wall B_3	169
D-4	Table D-3 Continued	170
D-5	Table D-3 Continued	171
D-6	Table D-3 Continued	172

TABLE D-1

RESULTS FROM STRAIN GAUGES

(STRAINS IN INCHES PER INCH x 10^4)

WALL A1

GAUGE	A	B	C	D	E	F
			HORIZONTAL LOAD 4,000 lb.			
εx	-4.1200	-0.0367	-0.0434	-0.0334	0.6614	0.6614
εy	0.5011	-0.0802	-0.0601	-0.0535	0.6347	0.6815
γxy	0.4643	-0.0501	-0.0468	-0.0167	0.6381	0.8285
			HORIZONTAL LOAD 6,000 lb.			
εx	-0.1436	-0.1069	-0.0735	-0.0969	1.0423	1.0256
εy	1.0523	-0.0167	-0.0200	-0.1269	0.9922	0.9955
γxy	0.9487	-0.0802	-0.0902	-0.1236	0.9454	0.9387
			HORIZONTAL LOAD 8,000 lb.			
εx	-0.0802	-0.0367	-0.0301	-0.0100	1.2594	7.7384
εy	1.1392	0.0134	0.0367	-0.0935	1.1191	1.1792
γxy	1.0490	-0.0067	0.0134	0.0535	1.2294	-4.9960
			HORIZONTAL LOAD 10,000 lb.			
εx	-0.1570	-0.1136	-0.0902	-0.1136	1.0423	1.0590
εy	1.1525	0.0568	0.0334	-0.2105	0.9421	0.9220
γxy	1.0122	-0.0568	-0.0835	-0.1303	0.9387	0.9187
			HORIZONTAL LOAD 12,000 lb.			
εx	-0.1637	-0.1236	-0.0969	-0.1303	0.9788	0.9922
εy	1.0790	0.0969	0.0468	-0.2439	0.8452	0.8185
γxy	0.9521	-0.0434	-0.0702	-0.1269	0.8752	0.8485
			HORIZONTAL LOAD 14,000 lb.			
εx	-7.1050	-0.1169	-0.1102	-0.1336	0.9955	1.0022
εy	1.1826	0.1470	0.0735	-0.2773	0.8285	0.7617
γxy	1.0389	-0.0200	-0.0635	-0.1436	0.8686	0.8251

TABLE D-2

RESULTS FROM STRAIN GAUGES

(STRAINS IN INCHES PER INCH x 10^4)

WALL A1

GAUGE	A	B	C	D	E	F
			HORIZONTAL LOAD 16,000 lb.			
εx	-0.1269	-0.1069	-0.1236	-0.1637	0.9755	0.9588
εy	1.2795	0.2338	0.0969	-0.3307	0.3580	0.6314
γxy	0.9955	0.0668	-0.0635	-0.1403	0.8619	0.7383
			HORIZONTAL LOAD 18,000 lb.			
εx	0.2739	0.2472	0.0100	-0.0367	1.2026	1.2861
εy	1.4365	0.2171	0.0334	-0.0902	1.1458	1.2227
γxy	0.8118	0.0267	-0.0501	-0.0568	1.0022	0.9988

TABLE D-3

RESULTS FROM STRAIN GAUGES

(STRAINS IN INCHES PER INCH x 10^4)

WALL B3

GAUGE	A	B	C	D	E	F
			HORIZONTAL LOAD 4,000 lb.			
εx	-0.1371	-0.1706	-0.2207	-0.0368	0.1672	-0.0569
εy	0.2608	0.2006	0.1104	0.1438	0.4314	0.5050
γxy	0.3612	0.3210	0.5351	0.2441	0.5016	0.5986
			HORIZONTAL LOAD 6,000 lb.			
εx	-0.0535	-0.1304	-0.1706	-0.0268	0.2274	-0.0702
εy	0.2809	0.1170	0.0468	0.1739	0.5451	0.6187
γxy	0.3545	0.2408	0.4682	0.2274	0.5953	0.7156
			HORIZONTAL LOAD 8,000 lb.			
εx	0.0000	-0.0970	-0.1405	-0.0134	0.2542	-0.0635
εy	0.2608	0.0936	0.0301	0.2006	0.5919	0.6521
γxy	0.3444	0.2408	0.4682	0.2408	0.6086	0.7324
			HORIZONTAL LOAD 10,000 lb.			
εx	0.0301	-0.0635	-0.1037	-0.0000	0.2776	-0.0468
εy	0.2441	0.0769	0.0067	0.2274	0.6488	0.7223
γxy	0.3043	0.2040	0.4381	0.2274	0.6321	0.7758
			HORIZONTAL LOAD 12,000 lb.			
εx	-6.3300	-0.0368	-0.0736	0.0167	0.3043	-0.0368
εy	0.2809	0.0669	-0.0067	0.2508	0.6922	0.7424
γxy	0.3010	0.1772	0.4214	0.2274	0.6387	0.7792
			HORIZONTAL LOAD 14,000 lb.			
εx	0.0669	-0.0201	-0.0502	0.0268	0.3244	-0.0167
εy	0.2809	0.0736	-0.0067	0.2742	0.7424	0.7926
γxy	0.3010	0.1772	0.4214	0.2341	0.6655	0.7992

TABLE D-4

RESULTS FROM STRAIN GAUGES
(STRAINS IN INCHES PER INCH x 10^4)
WALL B3

GAUGE	A	B	C	D	E	F
			HORIZONTAL LOAD 16,000 lb.			
εx	0.0936	0.0234	-0.0067	0.0502	0.3645	0.0033
εy	0.2742	0.0702	-0.0234	0.3177	0.8327	0.8695
γxy	0.2742	0.1605	0.4080	0.2341	0.7056	0.8327
			HORIZONTAL LOAD 18,000 lb.			
εx	-6.3300	0.0435	0.0167	0.0635	0.3745	0.0234
εy	0.2909	0.0736	-0.0301	0.3411	0.8762	0.9063
γxy	0.2776	0.1572	0.4080	0.2341	0.7056	0.8394
			HORIZONTAL LOAD 20,000 lb.			
εx	0.1237	0.0702	0.0401	-1.4279	0.3946	0.0435
εy	0.3010	0.0702	-0.0368	0.3645	0.9397	0.9631
γxy	0.2776	0.1538	0.4046	0.2374	0.7290	0.8594
			HORIZONTAL LOAD 22,000 lb.			
εx	-6.6660	0.0669	0.0368	0.0869	0.4113	0.0568
εy	0.3177	0.1003	-0.0134	0.4582	1.0768	1.0802
γxy	0.2876	0.2241	0.5284	0.3143	0.8293	0.9631
			HORIZONTAL LOAD 24,000 lb.			
εx	0.1538	0.0836	0.0468	0.0936	0.4247	0.0602
εy	0.3344	0.0936	-0.0234	0.4816	1.1136	1.1069
γxy	0.3010	0.2241	0.5317	0.3244	0.8427	0.9698
			HORIZONTAL LOAD 26,000 lb.			
εx	0.1906	0.1037	0.0702	0.1037	0.4448	0.0635
εy	0.3545	0.0803	-0.0334	0.5083	1.1805	1.1571
γxy	0.3110	0.2174	0.5284	0.3244	0.8762	1.0099

TABLE D-5

RESULTS FROM STRAIN GAUGES

(STRAINS IN INCHES PER INCH x 10^4)

WALL B3

GAUGE	A	B	C	D	E	F
			HORIZONTAL LOAD 28,000 lb.			
εx	0.3277	0.1338	0.0401	0.0970	0.2073	0.0869
εy	0.3344	0.1940	0.0435	0.5652	1.0300	0.9966
γxy	0.1839	0.2809	0.6454	0.4147	0.7391	0.8494
			HORIZONTAL LOAD 30,000 lb.			
εx	-1.1870	0.1471	0.0568	0.1170	0.2475	0.1137
εy	0.3545	0.1973	0.0401	0.5799	1.0902	1.0568
γxy	0.1973	0.3010	0.6554	5.8458	0.7692	0.8962
			HORIZONTAL LOAD 32,000 lb.			
εx	0.3946	0.1672	0.0769	0.1338	0.2675	0.1471
εy	0.3779	0.2006	0.0334	0.6287	1.1771	1.1404
γxy	0.2006	0.3143	0.6755	0.4448	0.8059	0.9430
			HORIZONTAL LOAD 34,000 lb.			
εx	0.4347	0.1806	0.0903	0.1505	0.3043	0.1906
εy	0.3913	0.1940	0.0268	0.6588	1.2641	1.2407
γxy	0.1940	0.3244	0.6889	0.4581	0.8494	0.9999
			HORIZONTAL LOAD 36,000 lb.			
εx	0.4682	0.1940	0.1070	0.1706	0.3344	0.2642
εy	0.4013	0.1973	0.0167	0.6989	1.3577	1.3510
γxy	0.1906	0.3311	0.7090	0.4815	0.8929	1.0634
			HORIZONTAL LOAD 38,000 lb.			
εx	0.5016	0.2040	0.1204	0.1940	0.4113	0.3645
εy	0.4180	0.1973	0.0201	0.7424	1.5082	1.5483
γxy	0.1906	0.3378	0.7357	0.5016	0.9865	1.1905

TABLE D-6

RESULTS FROM STRAIN GAUGES

(STRAINS IN INCHES PER INCH x 10^4)

WALL B3

GAUGE	A	B	C	D	E	F
			HORIZONTAL LOAD 40,000 lb.			
εx	0.5819	0.2508	0.1538	0.2441	0.5585	0.6354
εy	0.4815	0.2140	0.0167	0.8059	1.7624	1.8326
γxy	0.1973	0.3511	0.7825	0.5518	1.1370	1.4012
			HORIZONTAL LOAD 42,000 lb.			
εx	0.3980	0.1572	0.0803	0.1806	0.5016	0.4648
εy	0.3110	0.1672	0.0167	0.6722	1.5316	1.7356
γxy	0.1438	0.2140	0.6655	0.4916	1.0233	1.3744

APPENDIX E

DEFLECTION DATA

TABLE OF CONTENTS

APPENDIX E

		Page
Title Page		173
List of Tables		
E-1	Deflection Measurements in inches x 10^3 - Wall A_1	176
E-2	Deflection Measurements in inches x 10^3 - Wall A_2	178
E-3	Deflection Measurements in inches x 10^3 - Wall A_3	180
E-4	Deflection Measurements in inches x 10^3 - Wall A_4	182
E-5	Deflection Measurements in inches x 10^3 - Wall A_5	184
E-6	Deflection Measurements in inches x 10^4 - Wall B_{1A}	186
E-7	Deflection Measurements in inches x 10^5 - Wall B_{1B}	186
E-8	Deflection Measurements in inches x 10^3 - Wall B_3	189
E-9	Deflection Measurements in inches x 10^3 - Wall B_4	191
E-10	Deflection Measurements in inches x 10^3 - Wall B_5	193
E-11	Deflection Measurements in inches x 10^3 - Wall C_1	195
E-12	Deflection Measurements in inches x 10^3 - Wall C_2	197
List of Figures		
E-1	Location of Deflection Measuring Devices Wall A_1	177
E-2	Location of Deflection Measuring Devices Wall A_2	179
E-3	Location of Deflection Measuring Devices Wall A_3	181
E-4	Location of Deflection Measuring Devices Wall A_4	183
E-5	Location of Deflection Measuring Devices Wall A_5	185
E-6	Location of Deflection Measuring Devices Wall B_{1A}	187
E-7	Location of Deflection Measuring Devices Wall B_{1B}	188
E-8	Location of Deflection Measuring Devices Wall B_3	190
E-9	Location of Deflection Measuring Devices Wall B_4	192

Page

E-10 Location of Deflection Measuring Devices Wall B_5 194

E-11 Location of Deflection Measuring Devices Wall C_1 196

E-12 Location of Deflection Measuring Devices Wall C_2 198

TABLE E-1. DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL A1 TOTAL VERTICAL LOAD 2.33^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	000	
4,000	000	-039	041	060	072	050	
6,000	002	-046	054	069	088	060	
8,000	008	-052	055	084	110	071	
10,000	012	-053	063	095	123	078	
12,000	015	-056	075	113	162	090	
14,000	026	-062	083	128	167	098	
16,000							Failure

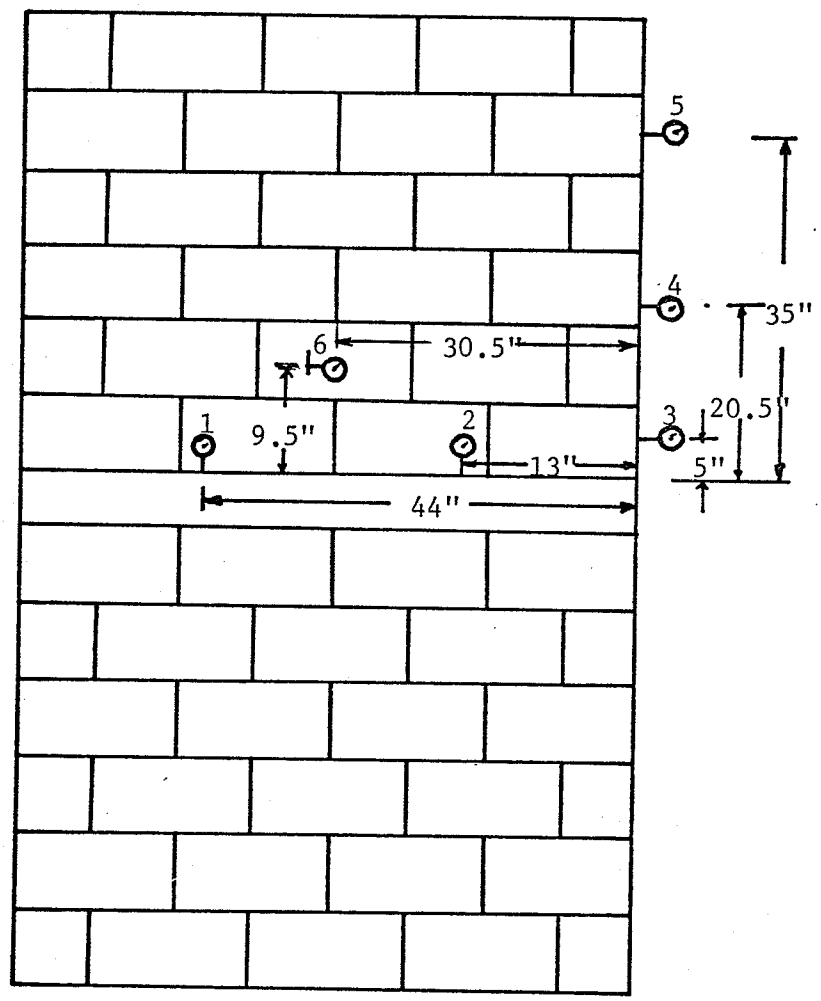


FIGURE E-1. LOCATION OF DEFLECTION MEASURING DEVICES - WALL A_1.

TABLE E-2. DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL A2 TOTAL VERTICAL LOAD 15.7^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	000	
4,000	-006	-025	019	020	030	024	
6,000	-006	-030	033	030	047	033	
8,000	-005	-035	045	036	047	040	
10,000	-005	-049	078	040	047	045	
12,000	-007	-040	081	041	050	051	
14,000	-004	-032	092	039	028	051	
16,000	-004	-038	106	048	035	066	
18,000	-004	-040	115	050	040	069	
20,000	+005	-042	122	055	039	070	
22,000	+007	-045	125	058	025	071	
24,000	+010	-048	129	062	018	073	
26,000							Failure

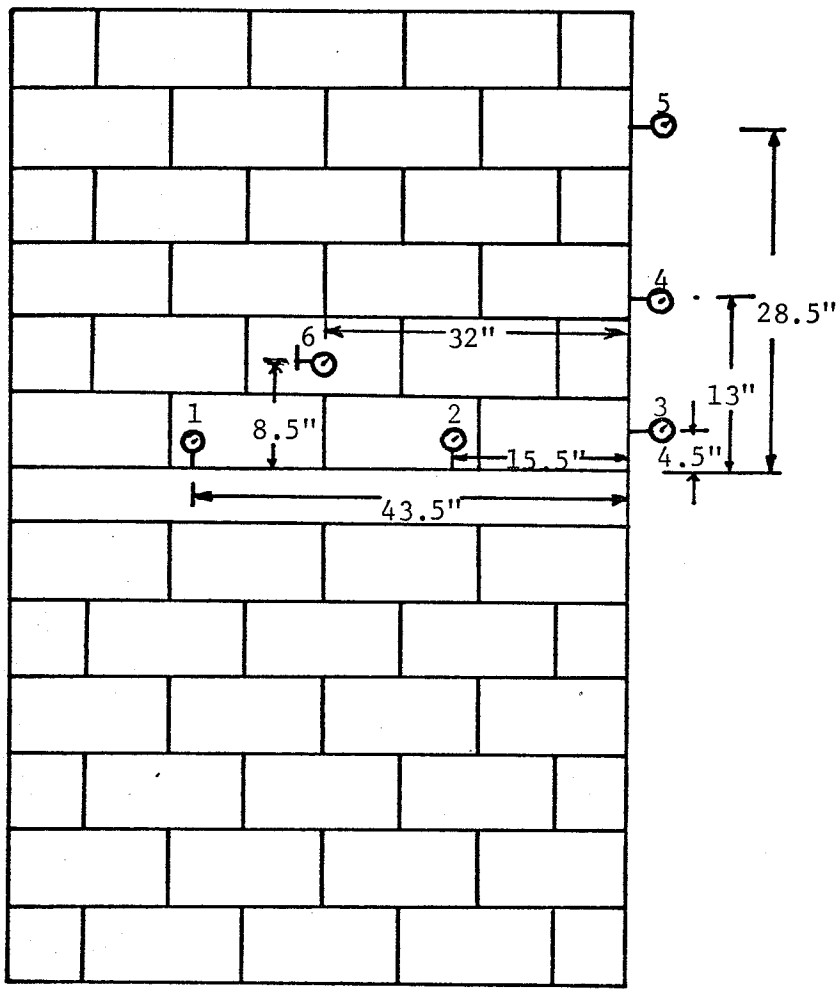


FIGURE E-2. LOCATION OF DEFLECTION MEASURING DEVICES - WALL A_2.

TABLE E-3. DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL A3 TOTAL VERTICAL LOAD 28.30^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	008	
4,000	-004	-002	010	015	020	012	
6,000	-006	-005	015	020	030	019	
8,000	-006	-011	010	025	036	026	
10,000	-011	-020	015	045	052	037	
12,000	-021	-027	015	043	066	047	
14,000	-047	-035	015	055	086	057	
16,000	-046	-042	016	067	105	067	
18,000	-048	-050	018	078	129	078	
20,000	-043	-059	019	092	140	092	
22,000	-041	-065	020	106	137	102	
24,000	-039	-107	022	107	144	107	
26,000	-021	-107	010	110	155	104	
28,000	-008	-100	010	112	158	112	
30,000	009	-100	019	126	179	114	
32,000	034	-094	021	150	179	147	
34,000	037	-099	088	187	199	187	
36,000			233	271	276	336	Failure
25,000			300	330	342	422	

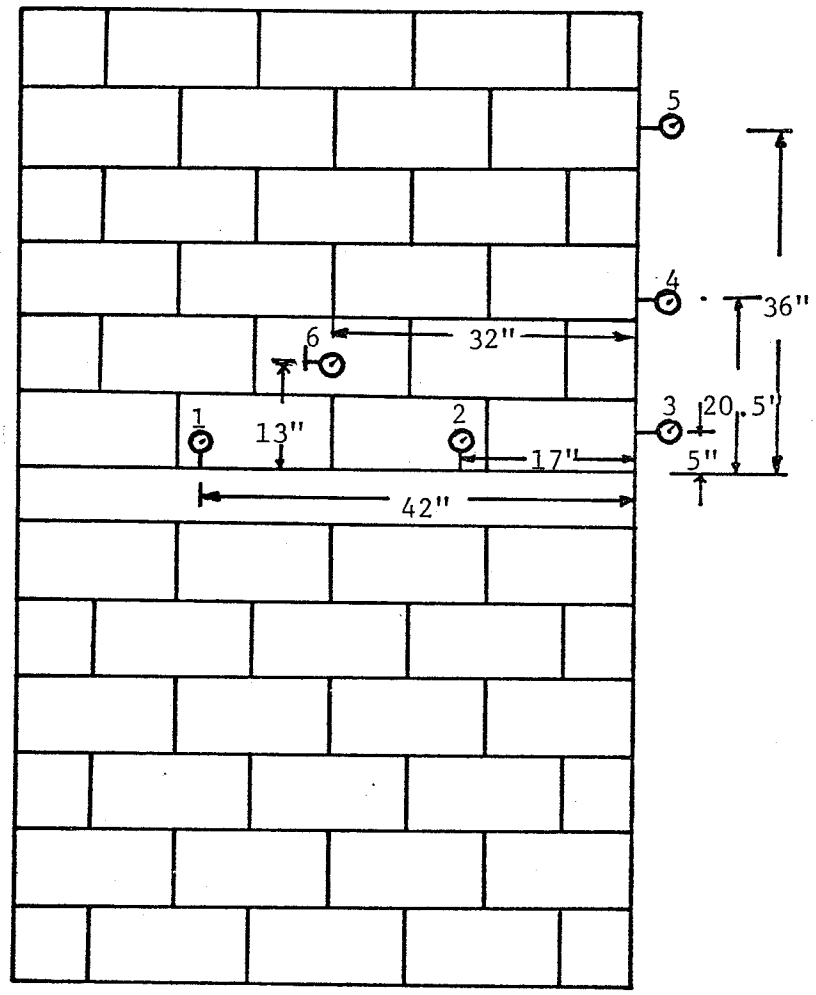


FIGURE E-3. LOCATION OF DEFLECTION MEASURING DEVICES - WALL A_3.

TABLE E-4. DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL A4 TOTAL VERTICAL LOAD 39.95^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	000	
4,000	000	-009	009	003	012	010	
6,000	-002	-010	009	004	016	014	
8,000	-010	-021	018	009	025	020	
10,000	-010	-013	026	032	048	028	
12,000	-010	-016	027	029	055	038	
14,000	-008	-022	033	036	068	048	
16,000	-007	-025	035	039	076	057	
18,000	-004	-029	045	052	092	065	
20,000	-002	-042	047	058	100	071	
22,000	000	-035	051	064	111	076	
24,000	002	-038	054	070	120	082	
26,000	005	-040	058	075	130	089	
28,000	008	-041	062	080	140	094	
30,000	011	-044	065	090	150	100	
32,000	014	-047	072	099	164	107	
34,000	016	-050	077	106	174	114	
36,000	020	-053	085	118	188	120	
38,000	022	-056	091	127	200	130	
40,000	026	-061	102	142	222	142	
42,000	028	-065	110	152	237	150	
44,000	035	-067	129	180	270	170	Failure
46,000	032	-067	155	208	202	195	
48,000	038	-071	171	230	330	214	
50,000	042	-077	190	250	360	231	

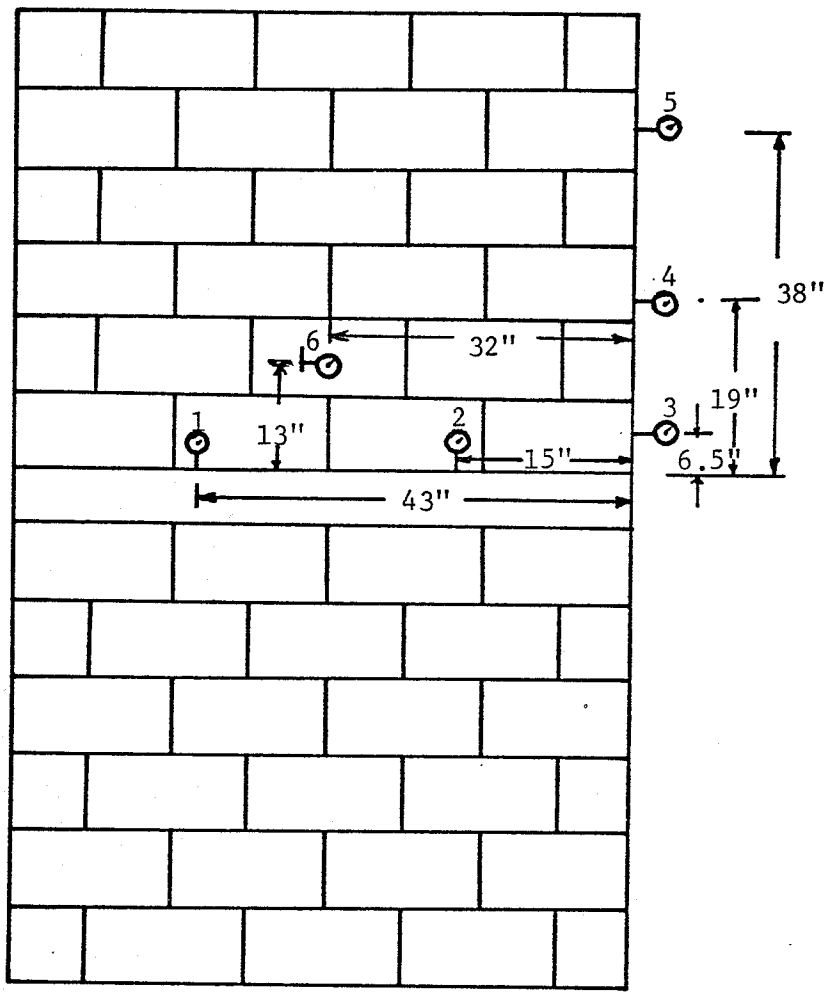


FIGURE E-4. LOCATION OF DEFLECTION MEASURING DEVICES - WALL A_4.

TABLE E-5. DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL A5 TOTAL VERTICAL LOAD 51.40^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	000	
4,000	000	000	000	000	000	000	
6,000	-005	-002	001	002	003	003	
8,000	-005	-005	007	009	011	005	
10,000	-005	-007	008	010	016	008	
12,000	-005	-010	010	011	018	010	
14,000	-006	-011	011	013	018	010	
16,000	-006	-015	014	016	020	010	
18,000	-008	-017	016	017	022	011	
20,000	-009	-021	020	018	030	011	
22,000	-010	-024	022	019	032	012	
24,000	-010	-027	032	030	040	013	
26,000	-010	-030	040	040	050	014	
28,000	-010	-035	048	048	058	015	
30,000	-010	-036	062	065	075	017	
32,000	-010	-040	062	070	080	018	
34,000	-010	-042	074	078	086	020	
36,000	-010	-035	080	085	095	021	
38,000	-020	-050	090	104	100	022	
40,000	-018	-052	110	125	126	023	
42,000	-015	-055	120	140	145	030	
44,000	-010	-060	138	185	192	055	
46,000	-010	-055	142	190	212	050	Failure
58,000 LOAD WAS REACHED AFTER FAILURE							

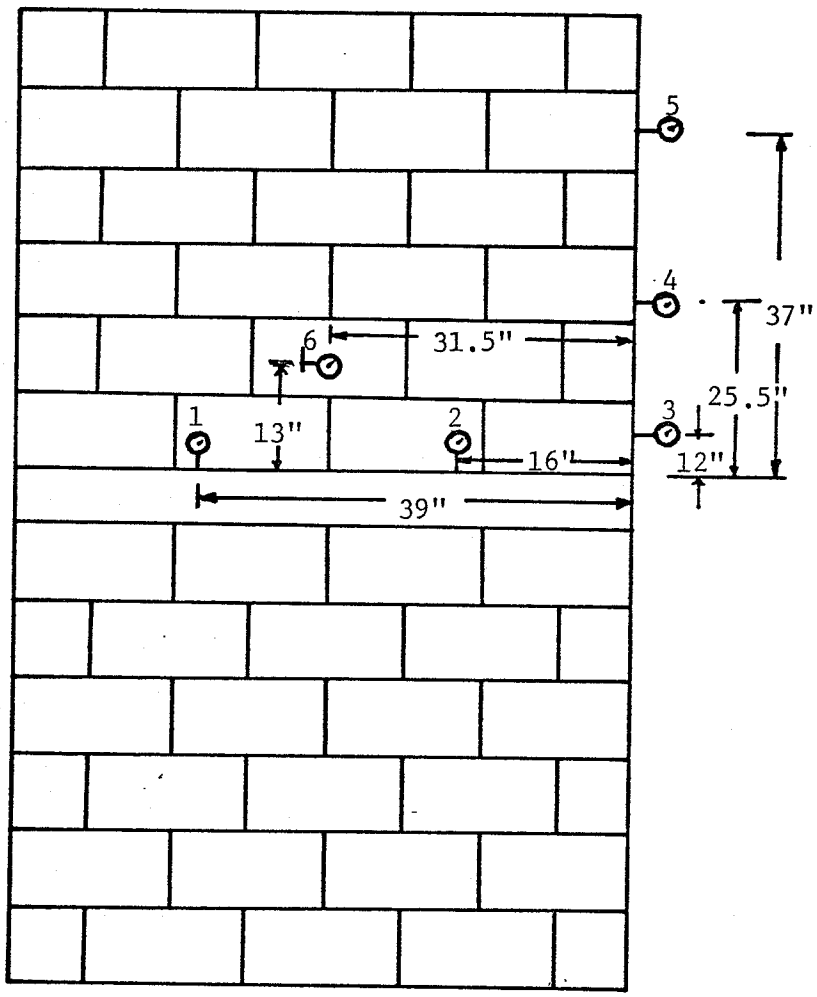


FIGURE E-5. LOCATION OF DEFLECTION MEASURING DEVICES - WALL A_5.

TABLE E-6.DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL B1A TOTAL VERTICAL LOAD 2.33^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	000	
4,000	003	-020	036	068	082	046	
6,000	004	-028	072	097	219	197	Failure

Table E-7.DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL B1B TOTAL VERTICAL LOAD 15.7^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	000	
4,000	-002	-030	005	081	109	010	
6,000	-010	-044	017	095	227	028	Failure

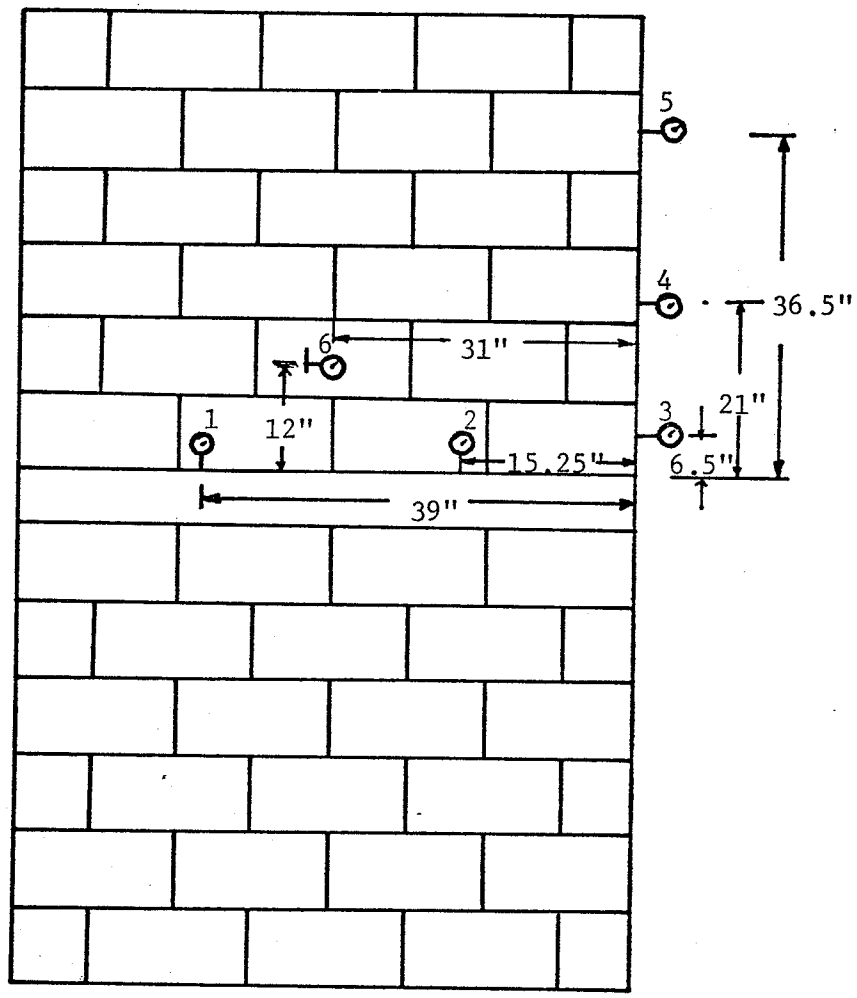


FIGURE E-6. LOCATION OF DEFLECTION MEASURING DEVICES - WALL B_{1A}

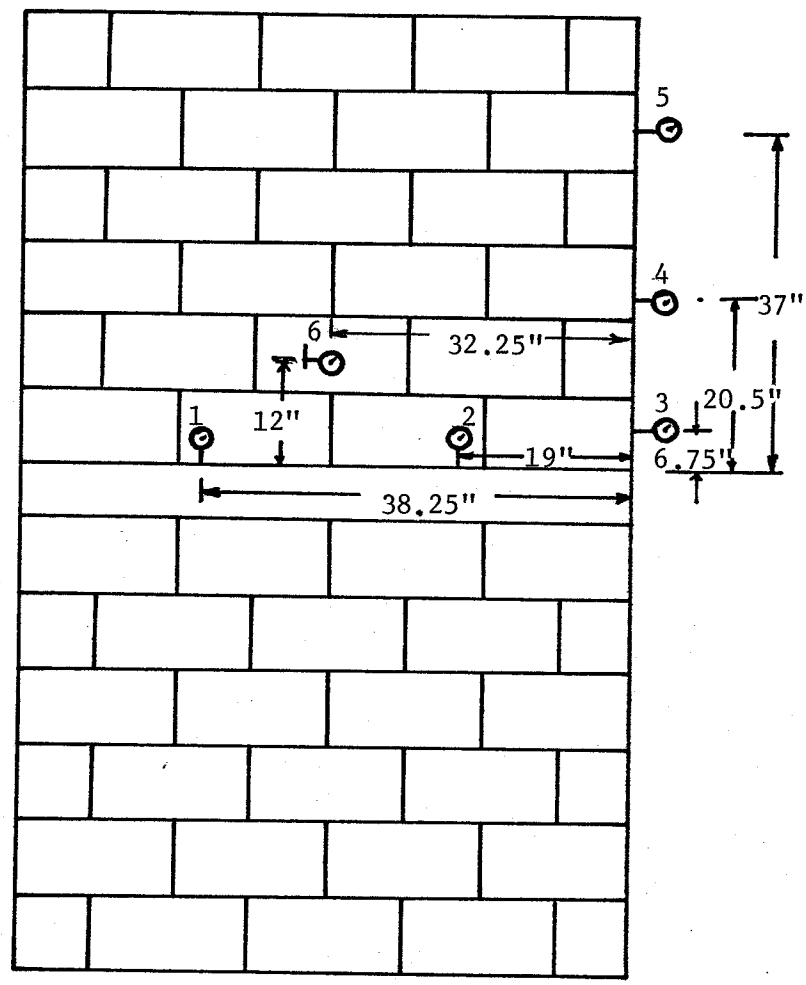


FIGURE E-7. LOCATION OF DEFLECTION MEASURING DEVICES - Wall B_{1B}.

TABLE. E-8 DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL B3 TOTAL VERTICAL LOAD 28.30^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	000	
4,000	000	-010	043	045	067	006	
6,000	000	-017	054	057	082	014	
8,000	002	-024	070	080	112	033	
10,000	004	-034	089	104	144	055	
12,000	006	-044	116	139	188	090	
14,000	008	-060	142	174	235	121	
16,000	009	-070	165	204	274	147	
18,000	017	-084	196	246	330	186	
20,000	007	-125	239	306	405	235	
22,000	006	-141	258	334	449	261	
24,000	008	-155	284	365	486	290	
26,000	006	-180	308	407	542	332	
28,000	008	-200	323	434	577	347	
30,000	012	-213	333	467	623	374	
32,000	013	-227	339	503	667	409	
34,000	018	-241	350	525	673	429	
36,000	021	-254	367	536	677	435	
38,000	025	-268	402	599	786	461	Shear Failure
22,000			785	641	1048	688	

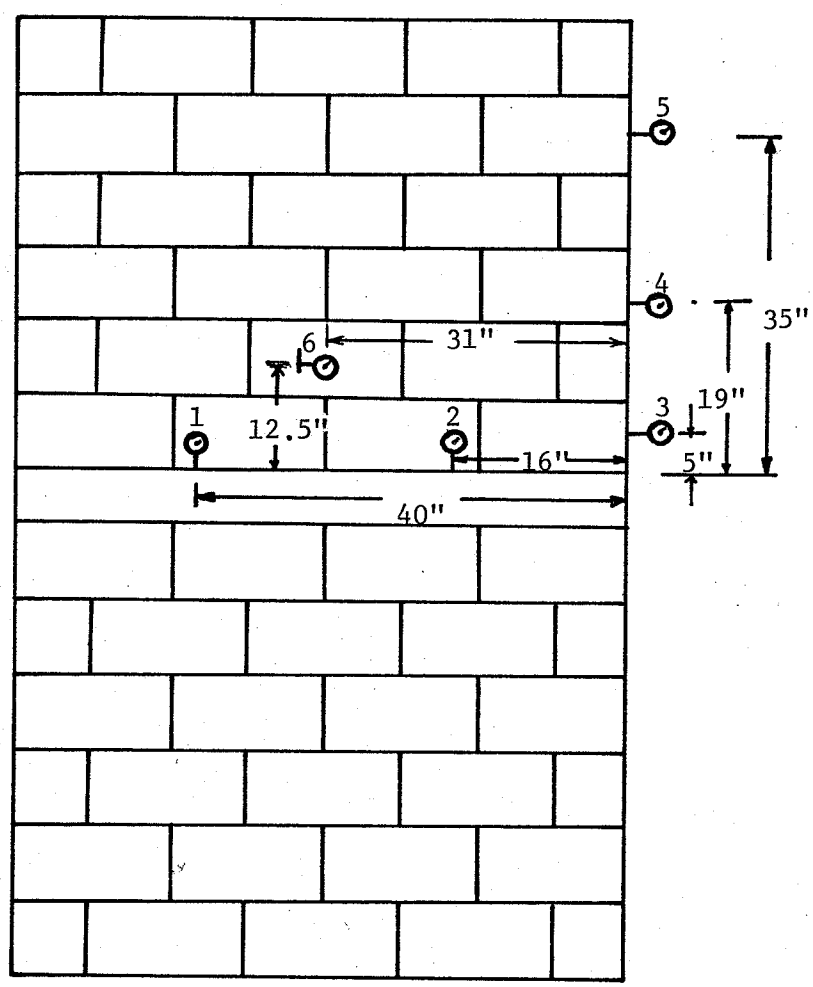


FIGURE E-8. LOCATION OF DEFLECTION MEASURING DEVICES - WALL B_3.

TABLE E-9. DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL B4 TOTAL VERTICAL LOAD 39.95^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	000	
4,000	-005	-005	015	020	028	013	
6,000	-001	-012	024	035	051	034	
8,000	-001	-019	040	058	080	055	
10,000	-001	-030	055	080	110	073	
12,000	-002	-038	072	104	144	095	
14,000	-003	-058	088	130	178	117	
16,000	-005	-083	125	182	253	156	
18,000	-013	-102	150	219	300	193	
20,000	-015	-147	165	243	336	213	
22,000	-015	-157	181	271	375	236	
24,000	-015	-172	194	296	411	257	
26,000	-016	-185	209	328	455	272	
28,000	-021	-193	220	338	472	292	
30,000	-020	-194	230	364	509	306	
32,000	-022	-273	234	368	521	318	
34,000	-017	-230	251	380	571	344	
36,000	-018	-236	258	384	592	354	
38,000	-018	-243	265	399	606	362	
40,000	-015	-253	279	422	647	385	
42,000	-013	-263	296	451	682	407	
44,000	-027	-233	715	822	900	841	Failure
22,000 SUSTAINED LOAD AFTER FAILURE							

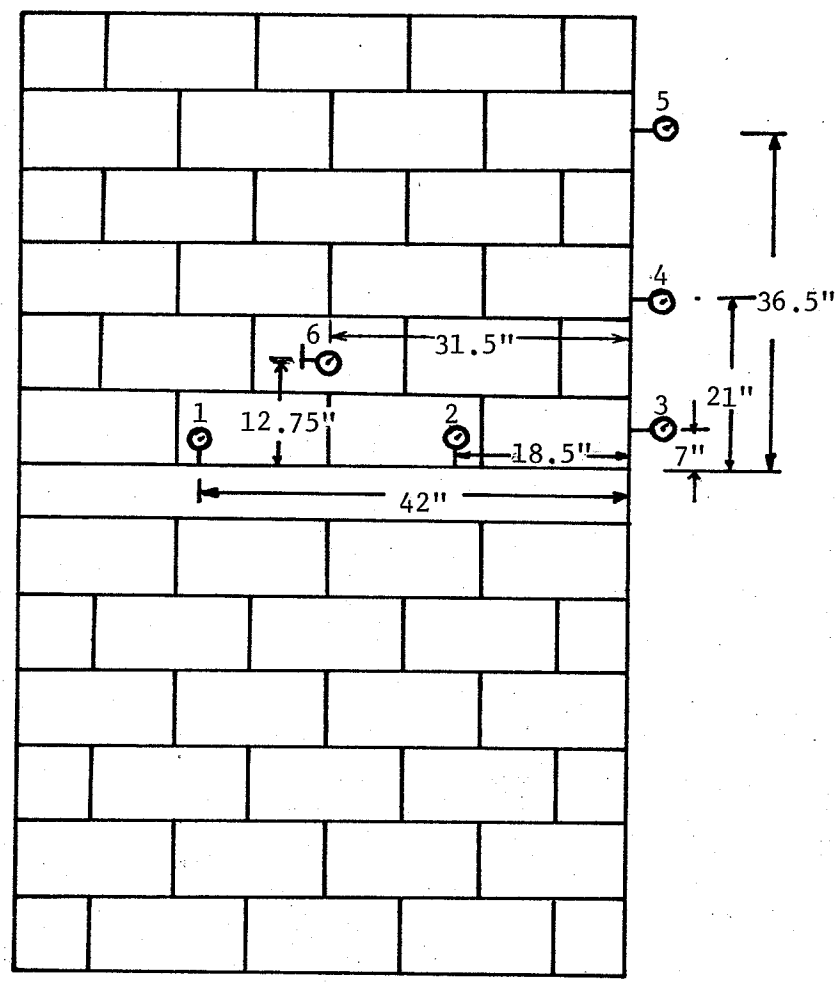


FIGURE E-9. LOCATION OF DEFLECTION MEASURING DEVICES - WALL B_4.

TABLE E-10. DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL B5 TOTAL VERTICAL LOAD 51.40^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	000	
4,000	-003	-012	023	031	039	027	
6,000	-005	-013	025	033	032	030	
8,000	-006	-017	031	042	042	037	
10,000	-006	-021	037	050	053	044	
12,000	-006	-024	042	058	063	050	
14,000	-005	-028	047	064	082	055	
16,000	-005	-030	052	070	090	059	
18,000	-005	-032	054	073	092	061	
20,000	-005	-035	058	077	099	064	
22,000	-005	-038	060	079	100	066	
24,000	-003	-040	060	080	105	067	
26,000	-002	-042	061	081	110	067	
28,000	-001	-043	063	081	113	067	
30,000	001	-045	064	082	116	066	
32,000	003	-047	065	085	119	066	
34,000	004	-048	069	091	120	065	
36,000	007	-049	071	096	124	064	
38,000	010	-050	072	102	126	064	
40,000	016	-053	076	109	209	051	Failure
36,000 REACHED WHEN LOADED AFTER FAILURE							

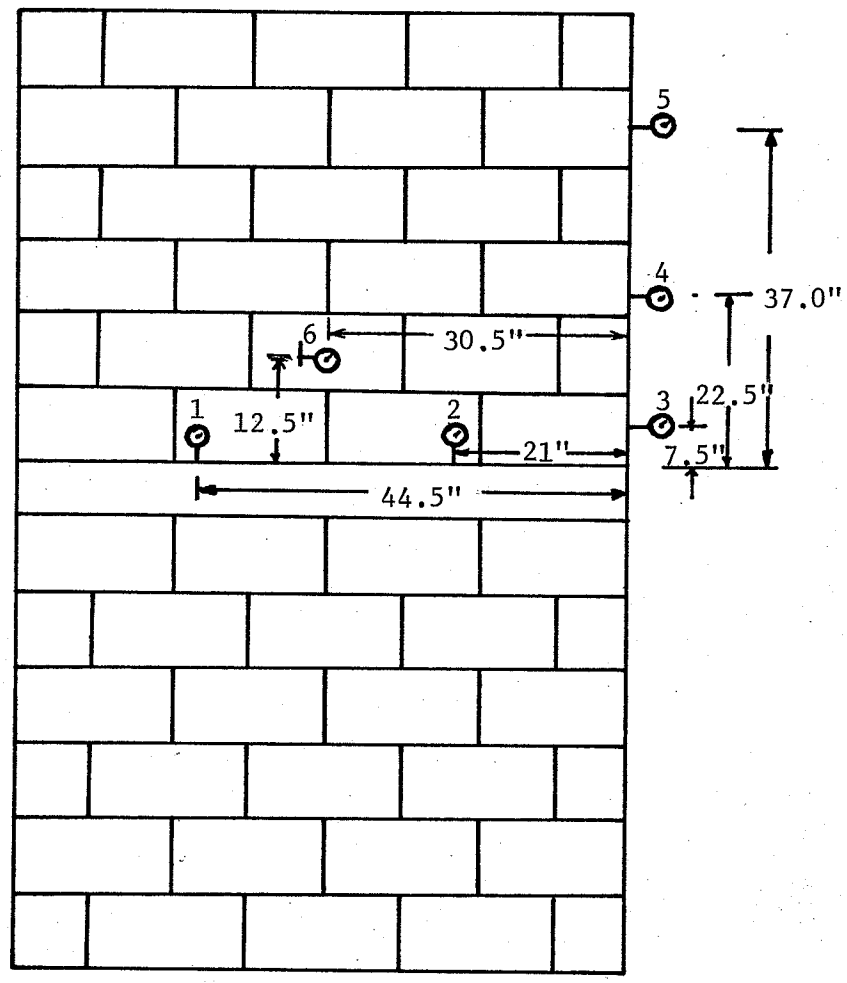


FIGURE E-10. LOCATION OF DEFLECTION MEASURING DEVICES - WALL B_5.

TABLE E-11. DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL C1 TOTAL VERTICAL LOAD 2.33^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	000	000	000	000	000	000	
4,000	006	-011	016	029	009	004	
6,000	014	-021	034	066	056	037	
8,000	024	-025	056	093	092	068	
10,000	033	-027	069	110	116	093	
12,000	047	-029	082	232	149	128	
14,000	059	-031	091	248	172	151	
16,000	069	-033	106	268	201	172	
18,000	080	-034	119	287	227	193	
20,000	092	-034	126	298	243	212	
22,000	103	-036	133	308	259	232	
24,000	112	-037	141	319	275	248	
26,000	121	-039	149	331	290	264	
28,000	128	-040	156	341	305	277	
30,000	137	-030	389	584	566	552	Failure by Shear
16,000 LOAD REMAINED AFTER FAILURE							

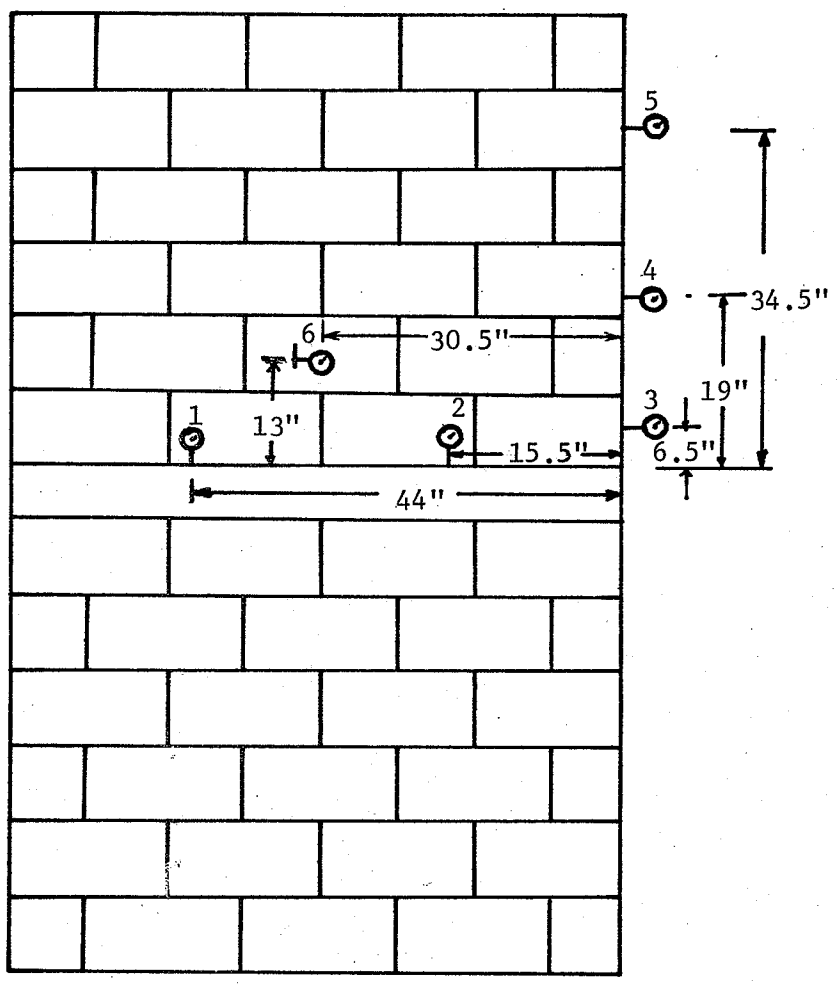


FIGURE E-11. LOCATION OF DEFLECTION MEASURING DEVICES - Wall C_1.

TABLE E-12. DEFLECTION MEASUREMENTS IN INCHES x 10^3

WALL C2 TOTAL VERTICAL LOAD 2.33^k

Total Hor. Load in Kips	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Comments
000	200	200	200	200	200	200	
4,000	204	190	221	233	240	235	
6,000	208	190	226	241	251	245	
8,000	213	185	242	264	282	265	
10,000	220	182	257	285	310	285	
12,000	228	180	270	305	339	302	
14,000	239	179	290	334	375	323	
16,000	250	180	304	355	402	340	
18,000	260	181	315	370	425	350	
20,000	270	185	320	380	440	360	
22,000	280	185	334	402	470	372	
24,000	290	189	345	415	490	385	
26,000	315	192	362	445	532	417	
28,000	340	200	390	482	585	450	
30,000	360	207	406	510	625	470	
32,000	390	216	424	535	660	492	
34,000	415	228	440	560	670	518	
36,000	375	200	608	840	1038	730	Failure

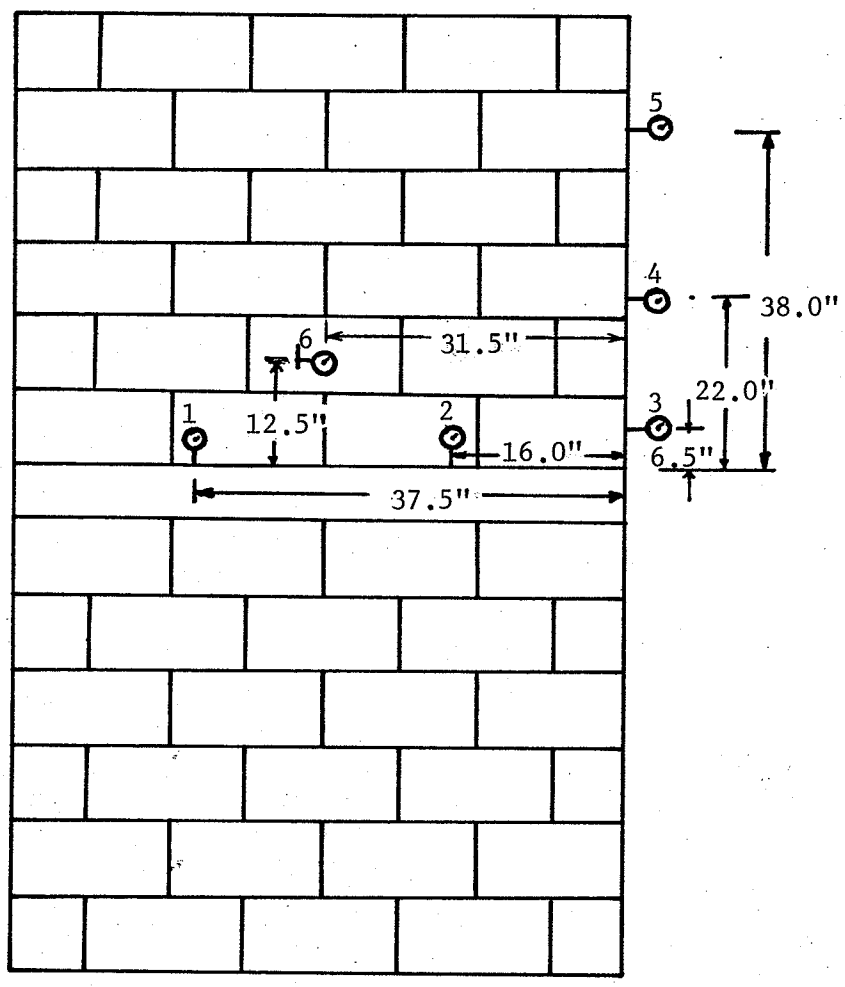


FIGURE E-12. LOCATION OF DEFLECTION MEASURING DEVICES - WALL C_2.

APPENDIX F

CIVIL ENGINEERING PROGRAM LIBRARY

) IDENTIFICATION

PSTRES - This program performs a linear finite element plane stress analysis. In-plane loadings, self weight loadings and temperature effects can be considered.

Programmed - Glenn A. Morris, March 1971.

) DESCRIPTION OF CONTINUUM AND LOADING

The contimuum must be subdivided into a series of quadrelateral elements interconnected by nodes which occur only at the corners of the elements. In describing the continuum, all nodes are numbered in an arbitrary sequence, as illustrated in Figure 1. The elements are numbered in a second arbitrary sequence.

All forces and modulus of elasticity must be expressed in the same units (kips or pounds).

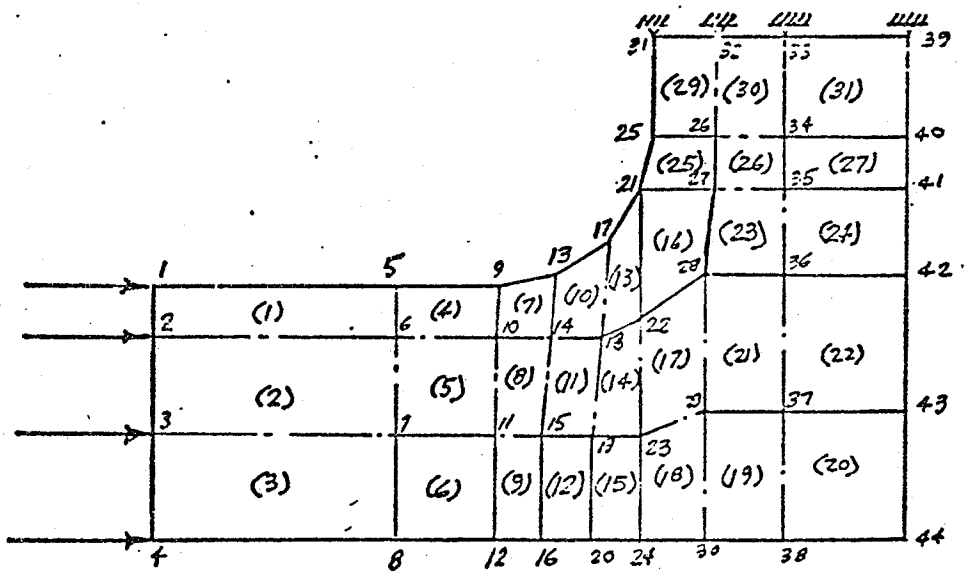


Fig 1

INPUT

The program input is described with reference to the Example below. Except for descriptive heading cards and the nodal information cards, each data card is divided into 10 column fields.

Data Cards:

(a) Job description - card to contain a job description which is printed as a heading over the output.

(b) General information

Field 1	number of elements
Field 2	number of nodes
Field 3	number of nodes loaded by external forces
Field 4	thickness of planar continuum
Field 5	thermal expansion coefficient α
Field 6 (Cols. 51 & 52)	units ('FT' or 'IN') used for element dimensions

If α is not provided,

α = .0000065 per degree F is assumed.

(c) Nodal information (one card for each node)

Field 1	node number
Field 2	Node X (horizontal) coordinate
Field 3	Node Y (vertical) coordinate
Field 4	Node type
	Blank = non-restrained node F, or FIXED = fixed node D, or DISPL = specified nodal displacements R - roller support; as illustrated in Fig 2.
Field 5	Slope Θ (degrees), as illustrated in Fig. 2, for any node constrained to move perpendicular to a given direction. Θ varies between 90° and -90°.
Fields 5 and 6	Specified nodal displacements, if any (horizontal (in), vertical (in))

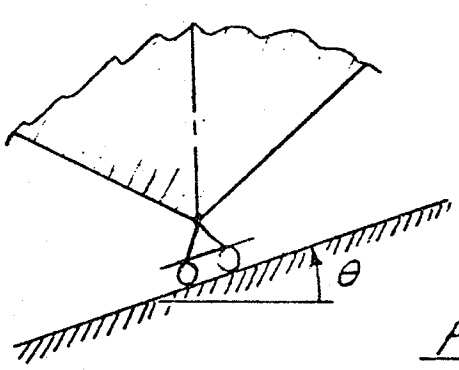


Fig 2

(d) Element information (one card for each element)

Field 1	Element number	5 column fields
Fields 2,3,4,5	Numbers of nodes at element corners Nodes must be listed counter clockwise	5 column fields
Field 6	Modulus of elasticity (ksi or psi)	
Field 7	Poisson's ratio	
Field 8	Unit weight of material (k/cu.ft., lb/cu in, etc.)	

Field 9	Element temperature If element temperatures are provided, the effects of these temperatures on the displacements and stresses are included. Otherwise, temperature effects are ignored.

If any of fields 6 to 9 are left blank, the corresponding values are assumed to be the same as for the preceding element.

(e) Loading information (one card for each nodal load)

Field 1	node number
Field 2	horizontal load
Field 3	vertical load

4) OUTPUT The output consists of:

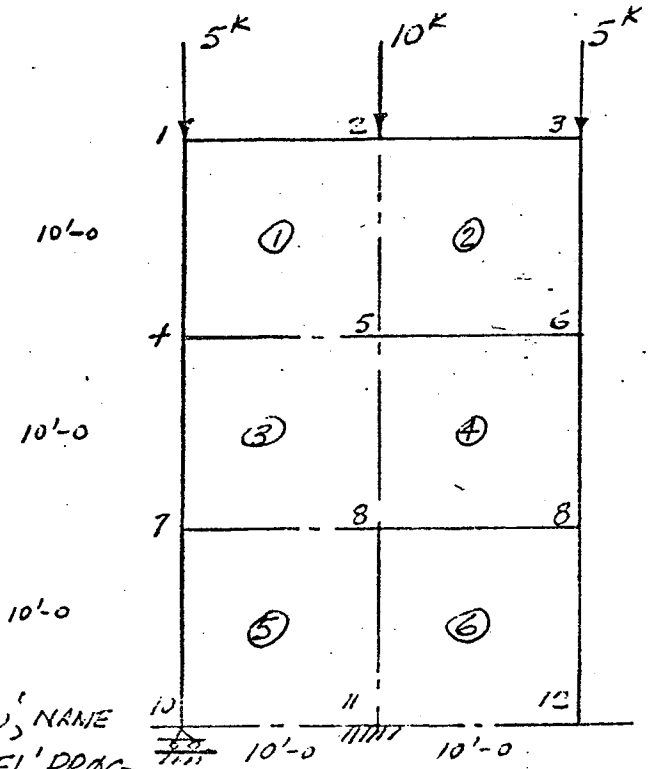
(a) A listing of all input quantities
(b) Nodal displacements
(c) Constant vertical, horizontal and shearing and principal stresses for each element (tension is positive).
(d) Stresses at each node, calculated as the average of the values for the elements adjacent to that node.

<u>Example</u> Analyze the 1 Ft thick plate shown.

$E = 2000.0$ ksi $\quad \nu = 0.25 \quad$ Unit weight $= .10$ k/cu ft.

Node 10 is free to displace horizontally.
Node 11 is fixed.
Node 12 has specified displacements:

horizontal	0.00002 in.
vertical	0.00001 in.



<u>Input</u>

```
//STRESS JOB '0    ,MATR,XX,R=180K,C=0,' NAME
// EXEC CIVENG,LIB='MORRIS.FINEL',PROG=STRESS
//GO.SYSIN DD *
      EXAMPLE PROBLEM    G.MORRIS
        6|        12|        3|          .0000066|FT
         1|  0.      |  30.    |          |
         2|  10.     |  30.    |          |
         3|  20.     |  30.    |          |
         4|  0.      |  20.    |          |
         5|  10.     |  20.    |          |
         6|  20.     |  20.    |          |
         7|          |  10.    |          |
         8|  10.     |  10.    |          |
         9|  20.     |  10.    |          |
         10|         |         |  R       |  0.
         11|  10.    |         |  E       |
         12|  20.    |         |  D       |  .00002   .00001
  1|  4|  5|  2|  1|  2000.   .25   .10
  2|  5|  6|  3|  2|
  3|  7|  8|  5|  4|
  4|  8|  9|  6|  5|
  5|  10|  11|  8|  7|
  6|  11|  12|  9|  8|
  *|  1|  -5.|
  1234 5|6  10|11  15|16  20|21  -5.|30 31  40  50  60  columns
  /*
```

EXAMPLE PROBLEM G.MORRIS

O DIMENSIONAL STRESS ANALYSIS

NPUT DATA

6 ELEMENTS 12 NODES ELEMENT THICKNESS = 1. (FT)

HERMAL EXPANSION COEFFICIENT = 0.0000066

ODAL COORDINATES (FT)

ODE	X COORD	Y COORD	BOUND. COND.	SLOPE	SPEC. DISPL.
1	0.0	30.0000			
2	10.0000	30.0000			
3	20.0000	30.0000			
4	0.0	20.0000			
5	10.0000	20.0000			
6	20.0000	20.0000			
7	0.0	10.0000			
8	10.0000	10.0000			
9	20.0000	10.0000			
10	0.0	0.0		0.0	
11	10.0000	0.0	FIXED		
12	20.0000	0.0		0.0002	0.0001

EMENT INCIDENCE TABLE

EMENT	NODE 1	NODE 2	NODE 3	NODE 4	MOD ELAST	POISS RATIO	UNIT W
1	4	5	2	1	2000.000	0.250	0.100
2	5	6	3	2	2000.000	0.250	0.100
3	7	8	5	4	2000.000	0.250	0.100
4	8	9	6	5	2000.000	0.250	0.100
5	10	11	8	7	2000.000	0.250	0.100
6	11	12	9	8	2000.000	0.250	0.100

N-ZERO NODAL LOADS

DE	PX(KIPS)	PY(KIPS)
1	0.0	-5.0000
2	0.0	-10.0000
3	0.0	-5.0000

RESULTS

NODAL DISPLACEMENTS (IN)

NODE	X	Y
1	-0.0004	-0.0031
2	-0.0002	-0.0031
3	-0.0000	-0.0030
4	-0.0004	-0.0025
5	-0.0002	-0.0025
6	0.0001	-0.0024
7	-0.0004	-0.0015
8	-0.0001	-0.0014
9	0.0002	-0.0013
10	-0.0004	-0.0000
11	-0.0000	-0.0000
12	0.0002	0.0001

ELEMENT STRESSES (KIPS/SQ FT)

ELEMENT	X STRESS	Y STRESS	XY STRESS	MIN STRESS	MAX STRESS	DIRECTIO
1	0.0820	-1.5100	-0.0043	-1.5100	0.0820	0.1544
2	0.0802	-1.5102	0.0043	-1.5102	0.0802	-0.1546
3	-0.0081	-2.5087	0.0083	-2.5087	-0.0081	-0.1901
4	-0.0275	-2.5115	-0.0083	-2.5116	-0.0275	0.1913
5	-0.0739	-3.4934	-0.0430	-3.4939	-0.0734	0.7210
6	-0.2990	-3.5268	0.0430	-3.5274	-0.2985	-0.7638

AVERAGE NODAL STRESSES

NODE	X STRESS	Y STRESS	XY STRESS	MIN STRESS	MAX STRESS	DIRECTI
1	0.0820	-1.0100	-0.0043	-1.0100	0.0820	0.2251
2	0.0811	-1.0101	0.0000	-1.0101	0.0811	-0.0000
3	0.0802	-1.0102	0.0043	-1.0102	0.0802	-0.2255
4	0.0370	-2.0093	0.0020	-2.0093	0.0370	-0.0561
5	0.0317	-2.0101	0.0000	-2.0101	0.0317	-0.0000
6	0.0264	-2.0109	-0.0020	-2.0109	0.0264	0.0563
7	-0.0410	-3.0010	-0.0174	-3.0011	-0.0409	0.3362
8	-0.1021	-3.0101	0.0000	-3.0101	-0.1021	-0.0000
9	-0.1633	-3.0192	0.0174	-3.0193	-0.1632	-0.3485
10	-0.0739	-3.9934	-0.0430	-3.9938	-0.0734	0.6291
11	-0.1865	-4.0101	0.0000	-4.0101	-0.1865	-0.0000
12	-0.2990	-4.0268	0.0430	-4.0273	-0.2985	-0.6614

NODAL COORDINATES (IN) WALLS A_1 TO A_5

NODE	X COORD	Y COORD
1	64.0000	101.0000
2	64.0000	93.0000
3	64.0000	85.0000
4	64.0000	77.0000
5	64.0000	69.0000
6	64.0000	61.0000
7	64.0000	53.0000
8	64.0000	48.0000
9	64.0000	40.0000
10	64.0000	32.0000
11	64.0000	24.0000
12	64.0000	16.0000
13	64.0000	8.0000
14	64.0000	0.0
15	56.0000	101.0000
16	56.0000	93.0000
17	56.0000	85.0000
18	56.0000	77.0000
19	56.0000	69.0000
20	56.0000	61.0000
21	56.0000	53.0000
22	56.0000	48.0000
23	56.0000	40.0000
24	56.0000	32.0000
25	56.0000	24.0000
26	56.0000	16.0000
27	56.0000	8.0000
28	56.0000	0.0
29	48.0000	101.0000
30	48.0000	93.0000
31	48.0000	85.0000
32	48.0000	77.0000
33	48.0000	69.0000
34	48.0000	61.0000
35	48.0000	53.0000
36	48.0000	48.0000
37	48.0000	40.0000
38	48.0000	32.0000
39	48.0000	24.0000
40	48.0000	16.0000
41	48.0000	8.0000
42	48.0000	0.0
43	40.0000	101.0000
44	40.0000	93.0000

NODAL COORDINATES (CONTINUED)

45	40.0000	85.0000
46	40.0000	77.0000
47	40.0000	69.0000
48	40.0000	61.0000
49	40.0000	53.0000
50	40.0000	48.0000
51	40.0000	40.0000
52	40.0000	32.0000
53	40.0000	24.0000
54	40.0000	16.0000
55	40.0000	8.0000
56	40.0000	0.0
57	32.0000	101.0000
58	32.0000	93.0000
59	32.0000	85.0000
60	32.0000	77.0000
61	32.0000	69.0000
62	32.0000	61.0000
63	32.0000	53.0000
64	32.0000	48.0000
65	32.0000	40.0000
66	32.0000	32.0000
67	32.0000	24.0000
68	32.0000	16.0000
69	32.0000	8.0000
70	32.0000	0.0
71	24.0000	101.0000
72	24.0000	93.0000
73	24.0000	85.0000
74	24.0000	77.0000
75	24.0000	69.0000
76	24.0000	61.0000
77	24.0000	53.0000
78	24.0000	48.0000
79	24.0000	40.0000
80	24.0000	32.0000
81	24.0000	24.0000
82	24.0000	16.0000
83	24.0000	8.0000
84	24.0000	0.0
85	16.0000	101.0000
86	16.0000	93.0000
87	16.0000	85.0000
88	16.0000	77.0000
89	16.0000	69.0000
90	16.0000	61.0000
91	16.0000	53.0000
92	16.0000	48.0000

NODAL COORDINATES (CONTINUED)

93	16.0000	40.0000
94	16.0000	32.0000
95	16.0000	24.0000
96	16.0000	16.0000
97	16.0000	8.0000
98	16.0000	0.0
99	8.0000	101.0000
100	8.0000	93.0000
101	8.0000	85.0000
102	8.0000	77.0000
103	8.0000	69.0000
104	8.0000	61.0000
105	8.0000	53.0000
106	8.0000	48.0000
107	8.0000	40.0000
108	8.0000	32.0000
109	8.0000	24.0000
110	8.0000	16.0000
111	8.0000	8.0000
112	8.0000	0.0
113	0.0	101.0000
114	0.0	93.0000
115	0.0	85.0000
116	0.0	77.0000
117	0.0	69.0000
118	0.0	61.0000
119	0.0	53.0000
120	0.0	48.0000
121	0.0	40.0000
122	0.0	32.0000
123	0.0	24.0000
124	0.0	16.0000
125	0.0	8.0000
126	0.0	0.0

TABLE F-1. ELEMENT INCIDENCE

ELEMENT	NODE 1	NODE 2	NODE 3	NODE 4	MOD ELAST	POISS RATIO	UNIT WT
1	1	15	16	2	3000.000	0.200	0.083
2	2	16	17	3	3000.000	0.200	0.083
3	3	17	18	4	3000.000	0.200	0.083
4	4	18	19	5	3000.000	0.200	0.083
5	5	19	20	6	3000.000	0.200	0.083
6	6	20	21	7	3000.000	0.200	0.083
7	7	21	22	8	3000.000	0.200	0.083
8	8	22	23	9	3000.000	0.200	0.083
9	9	23	24	10	3000.000	0.200	0.083
10	10	24	25	11	3000.000	0.200	0.083
11	11	25	26	12	3000.000	0.200	0.083
12	12	26	27	13	3000.000	0.200	0.083
13	13	27	28	14	3000.000	0.200	0.083
14	15	29	30	16	3000.000	0.200	0.083
15	16	30	31	17	3000.000	0.200	0.083
16	17	31	32	18	3000.000	0.200	0.083
17	18	32	33	19	3000.000	0.200	0.083
18	19	33	34	20	3000.000	0.200	0.083
19	20	34	35	21	3000.000	0.200	0.083
20	21	35	36	22	3000.000	0.200	0.083
21	22	36	37	23	3000.000	0.200	0.083
22	23	37	38	24	3000.000	0.200	0.083
23	24	38	39	25	3000.000	0.200	0.083
24	25	39	40	26	3000.000	0.200	0.083
25	26	40	41	27	3000.000	0.200	0.083
26	27	41	42	28	3000.000	0.200	0.083
27	29	43	44	30	3000.000	0.200	0.083
28	30	44	45	31	3000.000	0.200	0.083
29	31	45	46	32	3000.000	0.200	0.083
30	32	46	47	33	3000.000	0.200	0.083
31	33	47	48	34	3000.000	0.200	0.083
32	34	48	49	35	3000.000	0.200	0.083
33	35	49	50	36	3000.000	0.200	0.083

TABLE F-1. (CONTINUED)

ELEMENT	NODE 1	NODE 2	NODE 3	NODE 4	MOD.ELAST.	POISS.RATIO	UNIT WT.
34	36	50	51	37	3000.000	0.200	0.083
35	37	51	52	38	3000.000	0.200	0.083
36	38	52	53	39	3000.000	0.200	0.083
37	39	53	54	40	3000.000	0.200	0.083
38	40	54	55	41	3000.000	0.200	0.083
39	41	55	56	42	3000.000	0.200	0.083
40	43	57	58	44	3000.000	0.200	0.083
41	44	58	59	45	3000.000	0.200	0.083
42	45	59	60	46	3000.000	0.200	0.083
43	46	60	61	47	3000.000	0.200	0.083
44	47	61	62	48	3000.000	0.200	0.083
45	48	62	63	49	3000.000	0.200	0.083
46	49	63	64	50	3000.000	0.200	0.083
47	50	64	65	51	3000.000	0.200	0.083
48	51	65	66	52	3000.000	0.200	0.083
49	52	66	67	53	3000.000	0.200	0.083
50	53	67	68	54	3000.000	0.200	0.083
51	54	68	69	55	3000.000	0.200	0.083
52	55	69	70	56	3000.000	0.200	0.083
53	57	71	72	58	3000.000	0.200	0.083
54	58	72	73	59	3000.000	0.200	0.083
55	59	73	74	60	3000.000	0.200	0.083
56	60	74	75	61	3000.000	0.200	0.083
57	61	75	76	62	3000.000	0.200	0.083
58	62	76	77	63	3000.000	0.200	0.083
59	63	77	78	64	3000.000	0.200	0.083
60	64	78	79	65	3000.000	0.200	0.083
61	65	79	80	66	3000.000	0.200	0.083
62	66	80	81	67	3000.000	0.200	0.083
63	67	81	82	68	3000.000	0.200	0.083
64	68	82	83	69	3000.000	0.200	0.083
65	69	83	84	70	3000.000	0.200	0.083
66	71	85	86	72	3000.000	0.200	0.083

TABLE F-1. (CONTINUED)

ELEMENT	NODE 1	NODE 2	NODE 3	NODE 4	MOD.ELAST.	POISS.RATIO	UNIT WT.
67	72	86	87	73	3000.000	0.200	0.083
68	73	87	88	74	3000.000	0.200	0.083
69	74	88	89	75	3000.000	0.200	0.083
70	75	89	90	76	3000.000	0.200	0.083
71	76	90	91	77	3000.000	0.200	0.083
72	77	91	92	78	3000.000	0.200	0.083
73	78	92	93	79	3000.000	0.200	0.083
74	79	93	94	80	3000.000	0.200	0.083
75	80	94	95	81	3000.000	0.200	0.083
76	81	95	96	82	3000.000	0.200	0.083
77	82	96	97	83	3000.000	0.200	0.083
78	83	97	98	84	3000.000	0.200	0.083
79	85	99	100	86	3000.000	0.200	0.083
80	86	100	101	87	3000.000	0.200	0.083
81	87	101	102	88	3000.000	0.200	0.083
82	88	102	103	89	3000.000	0.200	0.083
83	89	103	104	90	3000.000	0.200	0.083
84	90	104	105	91	3000.000	0.200	0.083
85	91	105	106	92	3000.000	0.200	0.083
86	92	106	107	93	3000.000	0.200	0.083
87	93	107	108	94	3000.000	0.200	0.083
88	94	108	109	95	3000.000	0.200	0.083
89	95	109	110	96	3000.000	0.200	0.083
90	96	110	111	97	3000.000	0.200	0.083
91	97	111	112	98	3000.000	0.200	0.083
92	99	113	114	100	3000.000	0.200	0.083
93	100	114	115	101	3000.000	0.200	0.083
94	101	115	116	102	3000.000	0.200	0.083
95	102	116	117	103	3000.000	0.200	0.083
96	103	117	118	104	3000.000	0.200	0.083
97	104	118	119	105	3000.000	0.200	0.083
98	105	119	120	106	3000.000	0.200	0.083
99	106	120	121	107	3000.000	0.200	0.083
100	107	121	122	108	3000.000	0.200	0.083
101	108	122	123	109	3000.000	0.200	0.083
102	109	123	124	110	3000.000	0.200	0.083
103	110	124	125	111	3000.000	0.200	0.083
104	111	125	126	112	3000.000	0.200	0.083

TABLE F-2. AVERAGE NODAL STRESSES. WALL A_1

NODE	X STRESS	Y STRESS	XY STRESS	MIN STRESS	MAX STRESS	DIRECTION
1	0.0787	0.3466	0.0865	0.0532	0.3721	-73.5701
2	0.0298	-0.6491	0.2002	-0.7037	0.0844	-15.2684
3	-0.0447	-1.9672	0.4437	-2.0646	0.0528	-12.3879
4	-0.1261	-3.3820	0.7671	-3.5537	0.0456	-12.6147
5	-0.2962	-5.0749	1.3130	-5.4119	0.0408	-14.3951
6	-0.6279	-7.1804	2.3129	-7.9146	0.1063	-17.6102
7	-0.2255	-4.9214	3.2676	-6.5971	1.4502	-27.1501
8	0.3488	-1.0854	3.3555	-3.7996	3.0630	-38.9685
9	0.2944	-0.8023	2.8567	-3.1628	2.6550	-39.5671
10	0.2810	-0.6268	2.3108	-2.5278	2.1820	-39.4439
11	0.2800	-0.5827	1.7769	-1.9799	1.6771	-38.1772
12	0.2927	-0.6336	1.2247	-1.4798	1.1389	-34.6422
13	0.3017	-0.7754	0.6311	-1.0664	0.5928	-24.7611
14	0.3017	-1.2042	0.3220	-1.2701	0.3677	-11.5755
15	-0.0930	0.3525	0.1926	-0.1647	0.4242	-69.5737
16	-0.1337	-0.6053	0.3362	-0.7802	0.0412	-27.4775
17	-0.2089	-1.7901	0.6217	-2.0053	0.0063	-19.0898
18	-0.3255	-2.9333	0.9370	-3.2351	-0.0237	-17.8511
19	-0.5435	-4.1382	1.3503	-4.5889	-0.0928	-18.4583
20	-0.7654	-5.3800	1.8886	-6.0543	-0.0910	-19.6505
21	-0.3551	-4.0732	2.3200	-5.1871	0.7589	-25.6471
22	0.2537	-1.6917	2.4074	-3.3155	1.8775	-33.9994
23	0.4112	-1.3253	2.2064	-2.8282	1.9141	-34.2601
24	0.4702	-1.1065	1.8564	-2.3350	1.6987	-33.4954
25	0.4990	-1.0671	1.4440	-1.9267	1.3585	-30.7648
26	0.5300	-1.1701	0.9892	-1.6243	0.9842	-24.6631
27	0.5454	-1.4103	0.5026	-1.5319	0.6670	-13.6015
28	0.5444	-1.8999	0.2531	-1.9258	0.5703	-5.8500
29	-0.5115	0.3827	0.3308	-0.6205	0.4918	-71.7528
30	-0.4996	-0.4897	0.5000	-0.9946	0.0054	-45.2841
31	-0.5114	-1.4193	0.7974	-1.8830	-0.0477	-30.1736
32	-0.6195	-2.1207	1.0293	-2.6440	-0.0962	-26.9491
33	-0.8112	-2.6467	1.1840	-3.2269	-0.2309	-26.1097
34	-0.8420	-2.9144	1.1381	-3.4174	-0.3391	-23.8417
35	-0.4504	-2.7705	0.9823	-3.1305	-0.0904	-20.1290

TABLE F-2. AVERAGE NODAL STRESSES. WALL A1 (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
36	0.1239	-2.2036	1.0144	-2.5837	0.5040	-20.5392
37	0.5205	-1.9595	1.1050	-2.3804	0.9414	-20.8534
38	0.7216	-1.8057	1.0289	-2.1716	1.0875	-19.5768
39	0.8283	-1.8351	0.8336	-2.0745	1.0677	-16.0223
40	0.9017	-2.0416	0.5724	-2.1490	1.0090	-10.6260
41	0.9297	-2.4241	0.2860	-2.4484	0.9539	-4.8395
42	0.9274	-2.9937	0.1414	-2.9988	0.9325	-2.0630
43	-0.9857	0.4277	0.3337	-1.0605	0.5025	-77.3631
44	-0.8712	-0.3604	0.4709	-1.1515	-0.0801	-59.2371
45	-0.7266	-1.0826	0.6777	-1.6053	-0.2040	-37.6412
46	-0.7248	-1.5197	0.7644	-1.9839	-0.2607	-31.2649
47	-0.7741	-1.7863	0.7287	-2.1674	-0.3930	-27.6095
48	-0.6828	-1.9336	0.5381	-2.1332	-0.4832	-20.3554
49	-0.3499	-2.0584	0.3298	-2.1199	-0.2884	-10.5536
50	0.0928	-1.9478	0.2886	-1.9879	0.1328	-7.8967
51	0.4948	-2.0136	0.3505	-2.0617	0.5429	-7.8068
52	0.7851	-2.0587	0.3794	-2.1084	0.8348	-7.4710
53	0.9686	-2.2143	0.3409	-2.2504	1.0047	-6.0456
54	1.0861	-2.5142	0.2504	-2.5316	1.1035	-3.9588
55	1.1307	-2.9674	0.1312	-2.9716	1.1349	-1.8315
56	1.1311	-3.5662	0.0674	-3.5671	1.1321	-0.8215
57	-1.3559	0.4641	0.2346	-1.3856	0.4938	-82.7730
58	-1.1323	-0.2605	0.3010	-1.2262	-0.1667	-72.6871
59	-0.8155	-0.8389	0.3723	-1.1997	-0.4547	-44.0984
60	-0.6945	-1.1263	0.3366	-1.3103	-0.5105	-28.6627
61	-0.6347	-1.2908	0.2089	-1.3516	-0.5738	-16.2454
62	-0.4913	-1.4202	0.0038	-1.4202	-0.4913	-0.2338
63	-0.2218	-1.6118	-0.1700	-1.6323	-0.2013	6.8705
64	0.1086	-1.6166	-0.2259	-1.6457	0.1377	7.3386
65	0.4532	-1.8213	-0.1895	-1.8370	0.4688	4.7309
66	0.7521	-1.9983	-0.1140	-2.0030	0.7568	2.3702
67	0.9706	-2.2490	-0.0466	-2.2496	0.9712	0.8298
68	1.1209	-2.6137	-0.0029	-2.6137	1.1209	0.0441
69	1.1891	-3.1090	0.0131	-3.1091	1.1891	-0.1743
70	1.1988	-3.7238	0.0138	-3.7238	1.1988	-0.1606
71	-1.5123	0.4968	0.0622	-1.5143	0.4987	-88.2291
72	-1.2117	-0.1774	0.0325	-1.2127	-0.1764	-88.2028
73	-0.7757	-0.6507	-0.0517	-0.7943	-0.6321	-19.8111

TABLE F-2. AVERAGE NODAL STRESSES. WALL A_1 (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
74	-0.5803	-0.8434	-0.1798	-0.9346	-0.4891	26.9121
75	-0.4680	-0.9496	-0.3444	-1.1290	-0.2886	27.5207
76	-0.3218	-1.0562	-0.5262	-1.3307	-0.0474	27.5444
77	-0.1158	-1.2445	-0.6537	-1.5438	0.1835	24.5990
78	0.1230	-1.2594	-0.6925	-1.5466	0.4103	22.5269
79	0.3917	-1.4949	-0.6483	-1.6962	0.5930	17.2498
80	0.6518	-1.7138	-0.5305	-1.8273	0.7654	12.0789
81	0.8660	-1.9959	-0.3776	-2.0449	0.9150	7.3906
82	1.0330	-2.3785	-0.2179	-2.3924	1.0469	3.6392
83	1.1320	-2.8877	-0.0805	-2.8893	1.1336	1.1471
84	1.1598	-3.5102	-0.0215	-3.5103	1.1599	0.2641
85	-1.3723	0.5411	-0.1850	-1.3900	0.5589	-5.4726
86	-1.0607	-0.0816	-0.3226	-1.1574	0.0151	-16.6933
87	-0.6129	-0.4622	-0.5641	-1.1067	0.0316	-41.1964
88	-0.4137	-0.5859	-0.7586	-1.2633	0.2637	41.7637
89	-0.3026	-0.6474	-0.9334	-1.4241	0.4742	39.7685
90	-0.1871	-0.7203	-1.0844	-1.5705	0.6630	38.0942
91	-0.0469	-0.8761	-1.1702	-1.7030	0.7800	35.2460
92	0.1104	-0.8597	-1.1813	-1.6517	0.9024	33.8389
93	0.2953	-1.0660	-1.1115	-1.6887	0.9180	29.2595
94	0.4878	-1.2551	-0.9474	-1.6709	0.9036	23.6954
95	0.6640	-1.4994	-0.7150	-1.7143	0.8790	16.7324
96	0.8273	-1.8334	-0.4396	-1.9041	0.8981	9.1430
97	0.9686	-2.3047	-0.1650	-2.3130	0.9769	2.8783
98	1.0308	-2.9157	-0.0354	-2.9160	1.0311	0.5138
99	-0.8521	0.6196	-0.5490	-1.0343	0.8019	-18.3629
100	-0.6403	0.0558	-0.7806	-1.1469	0.5625	-32.9852
101	-0.3413	-0.2337	-1.1495	-1.4383	0.8633	-43.6594
102	-0.2148	-0.3063	-1.3843	-1.6456	1.1245	44.0532
103	-0.1473	-0.3331	-1.5581	-1.8010	1.3206	43.2939
104	-0.0842	-0.3691	-1.6847	-1.9173	1.4641	42.5831
105	-0.0129	-0.4803	-1.7410	-2.0032	1.5100	41.1779
106	0.0656	-0.4135	-1.7288	-1.9193	1.5714	41.0553
107	0.1603	-0.5562	-1.6340	-1.8708	1.4748	38.8162
108	0.2637	-0.6647	-1.4307	-1.7046	1.3036	36.0115
109	0.3670	-0.8074	-1.1328	-1.4961	1.0557	31.3000
110	0.4848	-1.0074	-0.7472	-1.3172	0.7946	22.5219

TABLE F-3 AVERAGE NODAL STRESSES. WALL A_2

NODE	X STRESS	Y STRESS	XY STRESS	MIN STRESS	MAX STRESS	DIRECTION
1	0.1391	0.4173	0.0977	0.1083	0.4481	-72.4596
2	0.0834	-0.5806	0.2072	-0.6400	0.1428	-15.9851
3	0.0007	-1.9025	0.4457	-2.0017	0.0999	-12.5498
4	-0.0830	-3.3211	0.7691	-3.4945	0.0904	-12.7045
5	-0.2544	-5.0205	1.3175	-5.3605	0.0855	-14.4633
6	-0.5882	-7.1374	2.3224	-7.8773	0.1518	-17.6726
7	-0.1957	-4.8962	3.2900	-6.5892	1.4973	-27.2299
8	0.3619	-1.0729	3.3844	-3.8151	3.1041	-39.0161
9	0.2973	-0.7935	2.8819	-3.1811	2.6849	-39.6417
10	0.2821	-0.6206	2.3348	-2.5473	2.2087	-39.5236
11	0.2818	-0.5785	1.7986	-1.9977	1.7010	-38.2748
12	0.2957	-0.6310	1.2417	-1.4929	1.1577	-34.7682
13	0.3058	-0.7738	0.6405	-1.0716	0.6037	-24.9378
14	0.3063	-1.2030	0.3270	-1.2708	0.3741	-11.7136
15	-0.0349	0.4151	0.1992	-0.1105	0.4906	-69.2366
16	-0.0309	-0.5433	0.3411	-0.7242	0.1000	-27.9323
17	-0.1636	-1.7301	0.6249	-1.9489	0.0551	-19.2922
18	-0.2837	-2.8770	0.9418	-3.1830	0.0222	-17.9966
19	-0.5044	-4.0382	1.3591	-4.5453	-0.0473	-18.5891
20	-0.7301	-5.3398	1.9037	-6.0243	-0.0456	-19.7717
21	-0.3292	-4.0457	2.3460	-5.1802	0.8053	-25.8086
22	0.2661	-1.6729	2.4383	-3.3274	1.9206	-34.1586
23	0.4150	-1.3105	2.2346	-2.8431	1.9476	-34.4442
24	0.4725	-1.0955	1.8828	-2.3510	1.7280	-33.6964
25	0.5028	-1.0594	1.4675	-1.9408	1.3841	-30.9876
26	0.5363	-1.1652	1.0076	-1.6331	1.0043	-24.9111
27	0.5540	-1.4075	0.5129	-1.5335	0.6800	-13.8035
28	0.5539	-1.8979	0.2586	-1.9249	0.5809	-5.9559
29	-0.4568	0.4356	0.3318	-0.5667	0.5454	-71.6828
30	-0.4490	-0.4363	0.5022	-0.9450	0.0596	-45.3630
31	-0.4682	-1.3666	0.8028	-1.8373	0.0025	-30.3844
32	-0.5819	-2.0715	1.0391	-2.6051	-0.0482	-27.1841

TABLE F-3. AVERAGE NODAL STRESSES. WALL A_2. (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX STRESS	DIRECTION
33	-0.7784	-2.6030	1.2030	-3.1982	-0.1832	-26.3770
34	-0.8150	-2.8776	1.1619	-3.3999	-0.2928	-24.2035
35	-0.4313	-2.7394	1.0135	-3.1213	-0.0494	-20.6457
36	0.1348	-2.1763	1.0489	-2.5814	0.5398	-21.1161
37	0.5260	-1.9361	1.1386	-2.3819	0.9718	-21.3831
38	0.7265	-1.7372	1.0604	-2.1747	1.1141	-20.0763
39	0.8366	-1.8217	0.8617	-2.0766	1.0915	-16.4771
40	0.9152	-2.0331	0.5945	-2.1485	1.0305	-10.9812
41	0.9481	-2.4197	0.2987	-2.4460	0.9744	-5.0287
42	0.9479	-2.9911	0.1484	-2.9966	0.9535	-2.1547
43	-0.9334	0.4785	0.3336	-1.0083	0.5534	-77.3521
44	-0.8240	-0.3099	0.4732	-1.1054	-0.0285	-59.2548
45	-0.6883	-1.0338	0.6853	-1.5678	-0.1543	-37.9257
46	-0.6934	-1.4744	0.7786	-1.9550	-0.2128	-31.6816
47	-0.7483	-1.7458	0.7502	-2.1478	-0.3462	-28.1917
48	-0.6629	-1.8978	0.5669	-2.1186	-0.4421	-21.2783
49	-0.3359	-2.0261	0.3643	-2.1013	-0.2607	-11.6607
50	0.1017	-1.9180	0.3260	-1.9693	0.1531	-8.9466
51	0.5010	-1.9868	0.3884	-2.0460	0.5603	-8.6700
52	0.7926	-2.0362	0.4162	-2.0962	0.8525	-8.1980
53	0.9817	-2.1972	0.3747	-2.2408	1.0252	-6.6331
54	1.1079	-2.5036	0.2782	-2.5249	1.1292	-4.3790
55	1.1614	-2.9632	0.1480	-2.9685	1.1667	-2.0525
56	1.1658	-3.5649	0.0772	-3.5662	1.1671	-0.9347
57	-1.3068	0.5142	0.2349	-1.3367	0.5440	-82.7669
58	-1.0897	-0.2113	0.3046	-1.1850	-0.1160	-72.6290
59	-0.7337	-0.7925	0.3830	-1.1711	-0.4051	-44.5706
60	-0.6702	-1.0340	0.3550	-1.2880	-0.4662	-29.3332
61	-0.6161	-1.2530	0.2347	-1.3301	-0.5389	-18.1951
62	-0.4776	-1.3863	0.0362	-1.3877	-0.4761	-2.2776
63	-0.2123	-1.5905	-0.1328	-1.5932	-0.1996	5.4929
64	0.1152	-1.5870	-0.1861	-1.6071	0.1353	6.1664
65	0.4590	-1.7939	-0.1483	-1.8036	0.4687	3.7507
66	0.7610	-1.9741	-0.0726	-1.9760	0.7629	1.5201
67	0.9377	-2.2297	-0.0065	-2.2297	0.9377	0.1158
68	1.1514	-2.6018	0.0326	-2.6021	1.1517	-0.4933
69	1.2351	-3.1066	0.0367	-3.1069	1.2354	-0.4849
70	1.2526	-3.7265	0.0291	-3.7266	1.2527	-0.3351
71	-1.4682	0.5461	0.0636	-1.4702	0.5481	-88.1937
72	-1.1754	-0.1299	0.0384	-1.1768	-0.1285	-87.8393

TABLE F-3. AVERAGE NODAL STRESSES. WALL A_2. (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
73	-0.7518	-0.6076	-0.0369	-0.7607	-0.5987	-13.5455
74	-0.5636	-0.8054	-0.1568	-0.8825	-0.4865	26.1854
75	-0.4560	-0.9162	-0.3145	-1.0758	-0.2965	26.9027
76	-0.3134	-1.0263	-0.4907	-1.2764	-0.0634	27.0026
77	-0.1102	-1.2166	-0.6145	-1.4903	0.1634	24.0030
78	0.1271	-1.2327	-0.6510	-1.4941	0.3885	21.8731
79	0.3961	-1.4693	-0.6051	-1.6484	0.5752	16.4857
80	0.6604	-1.6898	-0.4858	-1.7862	0.7568	11.2295
81	0.8850	-1.9751	-0.3315	-2.0130	0.9229	6.5249
82	1.0713	-2.3648	-0.1726	-2.3735	1.0799	2.8679
83	1.1977	-2.8877	-0.0452	-2.8882	1.1982	0.6337
84	1.2408	-3.5190	0.0052	-3.5190	1.2408	-0.0628
85	-1.3363	0.5876	-0.1807	-1.3531	0.6044	-5.3209
86	-1.0332	-0.0386	-0.3123	-1.1231	0.0513	-16.0629
87	-0.5977	-0.4259	-0.5433	-1.0618	0.0382	-40.5089
88	-0.4042	-0.5556	-0.7302	-1.2140	0.2542	42.0397
89	-0.2961	-0.6216	-0.8993	-1.3728	0.4551	39.8721
90	-0.1829	-0.6975	-1.0464	-1.5178	0.6374	38.0920
91	-0.0442	-0.8547	-1.1296	-1.6496	0.7506	35.1321
92	0.1122	-0.8389	-1.1391	-1.5978	0.8711	33.6696
93	0.2977	-1.0453	-1.0678	-1.6352	0.8876	28.9180
94	0.4941	-1.2341	-0.9016	-1.6188	0.9788	23.1081
95	0.6810	-1.4786	-0.6652	-1.6671	0.8694	15.8178
96	0.8691	-1.8164	-0.3838	-1.8701	0.9228	7.9758
97	1.0591	-2.3046	-0.1092	-2.3091	1.0627	1.8590
98	1.1529	-2.9292	0.0169	-2.9293	1.1530	-0.2370
99	-0.8309	0.6517	-0.5353	-1.0036	0.8249	-17.9215
100	-0.6250	0.0842	-0.7612	-1.1101	0.5693	-32.5396
101	-0.3345	-0.2117	-1.1210	-1.3957	0.8496	-43.4319
102	-0.2109	-0.2889	-1.3504	-1.6008	1.1010	44.1724
103	-0.1447	-0.3187	-1.5205	-1.7547	1.2912	43.3627
104	-0.0826	-0.3566	-1.6447	-1.8700	1.4308	42.6192

TABLE F-3. (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
105	-0.0121	-0.4686	-1.6996	-1.9552	1.4745	41.1757
106	0.0661	-0.4020	-1.6868	-1.8708	1.5350	41.0503
107	0.1612	-0.5443	-1.5913	-1.8215	1.4384	38.7510
108	0.2668	-0.6517	-1.3867	-1.6532	1.2683	35.6382
109	0.3774	-0.7927	-1.0844	-1.4398	1.0244	30.8265
110	0.5187	-0.9918	-0.6868	-1.2574	0.7843	21.1405
111	0.7988	-1.2963	-0.1893	-1.3133	0.8157	5.1229
112	0.9953	-1.8168	0.0853	-1.8194	0.9978	-1.7360
113	-0.4734	0.6908	-0.7473	-0.8386	1.0560	-26.0423
114	-0.3500	0.1554	-1.0133	-1.1416	0.9471	-37.9977
115	-0.1780	-0.0938	-1.4260	-1.5625	1.2907	-44.1540
116	-0.1078	-0.1483	-1.6703	-1.7985	1.5424	44.6529
117	-0.0714	-0.1622	-1.8398	-1.9571	1.7235	44.2935
118	-0.0389	-0.1809	-1.9554	-2.0665	1.8467	43.9606
119	-0.0033	-0.2686	-2.0000	-2.1404	1.8685	43.1033
120	0.0358	-0.1735	-1.9787	-2.0503	1.9126	43.4058
121	0.0840	-0.2779	-1.8736	-1.9793	1.7854	42.2413
122	0.1391	-0.3349	-1.6540	-1.7688	1.5730	40.9224
123	0.2013	-0.4104	-1.3256	-1.4649	1.2559	38.5045
124	0.3015	-0.5173	-0.8755	-1.0744	0.8586	32.4691
125	0.6330	-0.6738	-0.2401	-0.7165	0.6758	10.0899
126	0.8984	-1.1008	0.1368	-1.1101	0.9078	-3.9975

TABLE F-4 AVERAGE NODAL STRESSES WALL A_3

NODE	X STRESS	Y STRESS	XY STRESS	MIN STRESS	MAX STRESS	DIRECTION
1	0.2054	0.4792	0.1086	0.1675	0.5170	-70.7928
2	0.1420	-0.5213	0.2141	-0.5844	0.2051	-16.4228
3	0.0498	-1.8476	0.4482	-1.9481	0.1503	-12.6436
4	-0.0365	-3.2714	0.7725	-3.4464	0.1385	-12.7639
5	-0.2097	-4.9797	1.3248	-5.3229	0.1336	-14.5255
6	-0.5463	-7.1115	2.3374	-7.8587	0.2008	-17.7267
7	-0.1636	-4.8821	3.3210	-6.5966	1.5509	-27.3049
8	0.3771	-1.0629	3.4222	-3.8400	3.1542	-39.0594
9	0.3013	-0.7861	2.9141	-3.2068	2.7219	-39.7157
10	0.2839	-0.6154	2.3644	-2.5725	2.2410	-39.6163
11	0.2843	-0.5750	1.8247	-2.0200	1.7292	-38.3754
12	0.2994	-0.6289	1.2616	-1.5090	1.1796	-34.9010
13	0.3107	-0.7727	0.6515	-1.0783	0.6163	-25.1285
14	0.3116	-1.2024	0.3328	-1.2723	0.3816	-11.8668
15	0.0287	0.4700	0.2062	-0.0526	0.5513	-68.4676
16	-0.0235	-0.4894	0.3464	-0.6739	0.1610	-28.0414
17	-0.1151	-1.6789	0.6291	-1.9006	0.1066	-19.4101
18	-0.2392	-2.8307	0.9486	-3.1408	0.0709	-18.1033
19	-0.4631	-4.0497	1.3710	-4.5138	0.0009	-18.6993
20	-0.6934	-5.3129	1.9237	-6.0091	0.0028	-19.8946
21	-0.3016	-4.0275	2.3785	-5.1857	0.8567	-25.9648
22	0.2804	-1.6578	2.4762	-3.3478	1.9705	-34.3132
23	0.4204	-1.2981	2.2688	-2.8649	1.9872	-34.6292
24	0.4761	-1.0861	1.9143	-2.3725	1.7625	-33.9017
25	0.5079	-1.0529	1.4952	-1.9591	1.4141	-31.2194
26	0.5440	-1.1613	1.0288	-1.6449	1.0276	-25.1745
27	0.5641	-1.4057	0.5247	-1.5367	0.6951	-14.0231
28	0.5650	-1.8969	0.2649	-1.9251	0.5932	-6.0727
29	-0.3976	0.4821	0.3340	-0.5100	0.5945	-71.3961
30	-0.3947	-0.3895	0.5059	-0.8981	0.1138	-45.1477

TABLE F-4 (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
31	-0.4224	-1.3210	0.8101	-1.7981	0.0547	-30.4928
32	-0.5424	-2.0300	1.0517	-2.5744	0.0019	-27.3648
33	-0.7443	-2.5675	1.2196	-3.1785	-0.1333	-26.6117
34	-0.7873	-2.8486	1.1895	-3.3918	-0.2440	-24.5456
35	-0.4112	-2.7149	1.0489	-3.1209	-0.0052	-21.1599
36	0.1472	-2.1537	1.0879	-2.5866	0.5801	-21.6988
37	0.5333	-1.9161	1.1767	-2.3898	1.0070	-21.9278
38	0.7335	-1.7711	1.0961	-2.1831	1.1454	-20.5976
39	0.8471	-1.8101	0.8934	-2.0826	1.1196	-16.9592
40	0.9312	-2.0263	0.6194	-2.1507	1.0556	-11.3632
41	0.9695	-2.4171	0.3129	-2.4457	0.9981	-5.2342
42	0.9716	-2.9902	0.1562	-2.9963	0.9777	-2.2547
43	-0.8781	0.5233	0.3349	-0.9540	0.5992	-77.2258
44	-0.7741	-0.2655	0.4772	-1.0605	0.0210	-59.0260
45	-0.6480	-0.9911	0.6953	-1.5357	-0.1034	-38.0708
46	-0.6606	-1.4353	0.7956	-1.9329	-0.1630	-32.0195
47	-0.7217	-1.7111	0.7747	-2.1356	-0.2973	-28.7193
48	-0.6423	-1.8674	0.5989	-2.1115	-0.3983	-22.1762
49	-0.3211	-1.9986	0.4020	-2.0899	-0.2298	-12.8046
50	0.1120	-1.8920	0.3668	-1.9570	0.1770	-10.0523
51	0.5090	-1.9631	0.4298	-2.0358	0.5816	-9.5874
52	0.8023	-2.0162	0.4565	-2.0883	0.8744	-8.9749
53	0.9975	-2.1822	0.4120	-2.2347	1.0500	-7.2634
54	1.1331	-2.4950	0.3088	-2.5211	1.1592	-4.8304
55	1.1962	-2.9612	0.1665	-2.9679	1.2029	-2.2897
56	1.2051	-3.5661	0.0880	-3.5677	1.2067	-1.0563
57	-1.2562	0.5584	0.2364	-1.2865	0.5887	-82.6971
58	-1.0456	-0.1679	0.3098	-1.1439	-0.0696	-72.3896
59	-0.7506	-0.7517	0.3959	-1.1470	-0.3552	-44.9604
60	-0.6449	-1.0469	0.3758	-1.2721	-0.4197	-30.9320
61	-0.5967	-1.2198	0.2631	-1.3160	-0.5005	-20.0881
62	-0.4633	-1.3564	0.0711	-1.3620	-0.4576	-4.5256
63	-0.2021	-1.5526	-0.0931	-1.5590	-0.1957	3.9260
64	0.1229	-1.5604	-0.1436	-1.5726	0.1351	4.8403

TABLE F-4. (CONTINUED)

NODE	X STRESS	Y STRESS	SY STRESS	MIN.STRESS	MAX. STRESS	DIRECTION
65	0.4664	-1.7689	-0.1042	-1.7738	0.4712	2.6631
66	0.7720	-1.9519	-0.0280	-1.9522	0.7723	0.5896
67	1.0079	-2.2122	0.0369	-2.2126	1.0083	-0.6569
68	1.1861	-2.5919	0.0712	-2.5932	1.1874	-1.0793
69	1.2868	-3.1067	0.0625	-3.1076	1.2877	-0.8145
70	1.3127	-3.7320	0.0458	-3.7325	1.3131	-0.5200
71	-1.4239	0.5897	0.0660	-1.4261	0.5918	-88.1254
72	-1.1386	-0.0879	0.0456	-1.1406	-0.0859	-87.5179
73	-0.7269	-0.5695	-0.0203	-0.7294	-0.5669	-7.2379
74	-0.5460	-0.7717	-0.1319	-0.8324	-0.4853	24.7243
75	-0.4433	-0.8865	-0.2826	-1.0240	-0.3058	25.9499
76	-0.3044	-0.9995	-0.4533	-1.2232	-0.0807	26.2638
77	-0.1039	-1.1913	-0.5733	-1.4377	0.1425	23.2605
78	0.1320	-1.2081	-0.6073	-1.4424	0.3663	21.0940
79	0.4018	-1.4454	-0.5594	-1.6016	0.5580	15.5995
80	0.6709	-1.6671	-0.4381	-1.7465	0.7503	10.2720
81	0.9070	-1.9555	-0.2820	-1.9830	0.9345	5.5739
82	1.1143	-2.3526	-0.1237	-2.3570	1.1187	2.0413
83	1.2707	-2.8901	-0.0069	-2.8901	1.2707	0.0954
84	1.3304	-3.5309	0.0343	-3.5311	1.3307	-0.4038
85	-1.3010	0.6286	-0.1759	-1.3169	0.6445	-5.1667
86	-1.0058	-0.0006	-0.3012	-1.0892	0.0827	-15.4694
87	-0.5818	-0.3938	-0.5216	-1.0177	0.0422	-39.8930
88	-0.3938	-0.5286	-0.7006	-1.1651	0.2426	42.2523
89	-0.2891	-0.5984	-0.8641	-1.3215	0.4341	39.9253
90	-0.1781	-0.6768	-1.0069	-1.4648	0.6099	38.0463
91	-0.0411	-0.8351	-1.0875	-1.5957	0.7196	34.9725
92	0.1147	-0.8195	-1.0952	-1.5431	0.8383	33.4513
93	0.3011	-1.0255	-1.0221	-1.5807	0.8562	28.5094
94	0.5018	-1.2137	-0.8532	-1.5658	0.8539	22.4248
95	0.7003	-1.4582	-0.6121	-1.6197	0.8618	14.7805
96	0.9155	-1.7997	-0.3237	-1.8378	0.9536	6.7052
97	1.1588	-2.3062	-0.0488	-2.3069	1.1595	0.8067
98	1.2871	-2.9456	0.0737	-2.9469	1.2884	-0.9975
99	-0.8096	0.6800	-0.5223	-0.9745	0.8449	-17.5191
100	-0.6100	0.1094	-0.7422	-1.0751	0.5745	-32.0731

TABLE F-4. (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
101	-0.3272	-0.1922	-1.0926	-1.3543	0.8350	-43.2314
102	-0.2065	-0.2734	-1.3161	-1.5565	1.0766	44.2720
103	-0.1418	-0.3058	-1.4823	-1.7084	1.2607	43.4172
104	-0.0807	-0.3451	-1.6038	-1.8221	1.3963	42.6437
105	-0.0110	-0.4577	-1.6571	-1.9064	1.4377	41.1617
106	0.0669	-0.3910	-1.6434	-1.8213	1.4972	41.0340
107	0.1624	-0.5327	-1.5470	-1.7708	1.4004	38.6688
108	0.2705	-0.6387	-1.3405	-1.5996	1.2314	35.6329
109	0.3891	-0.7778	-1.0330	-1.3807	0.9920	30.2702
110	0.5562	-0.9758	-0.6218	-1.1964	0.7768	19.5334
111	0.9299	-1.2870	-0.0922	-1.2908	0.9337	2.3776
112	1.2003	-1.8142	0.2049	-1.8281	1.2142	-3.8698
113	-0.4617	0.7086	-0.7279	-0.8105	1.0574	-25.6030
114	-0.3420	0.1707	-0.9889	-1.1073	0.9359	-37.7340
115	-0.1747	-0.0827	-1.3940	-1.5235	1.2660	-44.0545
116	-0.1059	-0.1398	-1.6339	-1.7569	1.5111	44.7030
117	-0.0702	-0.1553	-1.8003	-1.9135	1.6881	44.3231
118	-0.0382	-0.1748	-1.9138	-2.0216	1.8086	43.9776
119	-0.0029	-0.2628	-1.9575	-2.0946	1.8289	43.1011
120	0.0361	-0.1677	-1.9357	-2.0041	1.8725	43.4936
121	0.0845	-0.2716	-1.8303	-1.9325	1.7454	42.2224
122	0.1409	-0.3274	-1.6096	-1.7198	1.5333	40.8620
123	0.2081	-0.4008	-1.2768	-1.4089	1.2162	38.2939
124	0.3301	-0.5047	-0.8111	-0.9996	0.8249	31.3844
125	0.7807	-0.6553	-0.1205	-0.6653	0.7908	4.7624
126	1.1474	-1.0782	0.2998	-1.1179	1.1871	-7.5384

TABLE F-5. AVERAGE NODAL STRESSES WALL A_4

NODE	X STRESS	Y STRESS	XY STRESS	MIN STRESS	MAX STRESS	DIRECTION
1	0.3073	0.5394	0.1223	0.2547	0.5919	-66.7477
2	0.2314	-0.4650	0.2227	-0.5302	0.2965	-16.3022
3	0.1238	-1.7992	0.4523	-1.9003	0.2249	-12.5950
4	0.0334	-3.2333	0.7802	-3.4101	0.2102	-12.7659
5	-0.1432	-4.9591	1.3415	-5.3076	0.2053	-14.5614
6	-0.4858	-7.1197	2.3707	-7.8798	0.2743	-17.7770
7	-0.1153	-4.8910	3.3829	-6.6435	1.6376	-27.3919
8	0.4028	-1.0554	3.4540	-3.8956	3.2429	-39.1065
9	0.3091	-0.7800	2.9746	-3.2589	2.7860	-39.8117
10	0.2881	-0.6107	2.4172	-2.6199	2.2974	-39.7240
11	0.2892	-0.5719	1.8697	-2.0600	1.7773	-38.5163
12	0.3061	-0.6273	1.2954	-1.5375	1.2162	-35.0333
13	0.3190	-0.7725	0.6699	-1.0908	0.6373	-25.4149
14	0.3206	-1.2027	0.3425	-1.2762	0.3940	-12.1066
15	0.1267	0.5238	0.2163	0.0316	0.6189	-66.2784
16	0.0637	-0.4377	0.3547	-0.6213	0.2474	-27.3746
17	-0.0426	-1.6329	0.6373	-1.8568	0.1812	-19.3549
18	-0.1734	-2.7940	0.9622	-3.1093	0.1419	-18.1465
19	-0.4031	-4.0271	1.3947	-4.5017	0.0715	-18.7926
20	-0.6410	-5.3100	1.9621	-6.0251	0.0740	-20.0229
21	-0.2606	-4.0266	2.4377	-5.2238	0.9366	-26.1578
22	0.3044	-1.6470	2.5437	-3.3956	2.0531	-34.5070
23	0.4310	-1.2875	2.3293	-2.9109	2.0545	-34.8757
24	0.4838	-1.0775	1.9686	-2.4146	1.8210	-34.1847
25	0.5176	-1.0471	1.5419	-1.9937	1.4643	-31.5433
26	0.5574	-1.1585	1.0641	-1.6674	1.0664	-25.5603
27	0.5810	-1.4056	0.5441	-1.5448	0.7202	-14.3560
28	0.5834	-1.8979	0.2752	-1.9281	0.6135	-6.2539
29	-0.3077	0.5285	0.3396	-0.4283	0.6488	-70.4552
30	-0.3135	-0.3437	0.5143	-0.8432	0.1859	-44.1574

TABLE F-5. (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
31	-0.3555	-1.2785	0.8249	-1.7622	0.1282	-30.3869
32	-0.4856	-1.9943	1.0750	-2.5533	0.0733	-27.4705
33	-0.6964	-2.5403	1.2538	-3.1746	-0.0621	-26.8367
34	-0.7487	-2.8295	1.2354	-3.4042	-0.1739	-24.9491
35	-0.3818	-2.6986	1.1055	-3.1414	0.0611	-21.8315
36	0.1678	-2.1356	1.1502	-2.6116	0.6438	-22.4814
37	0.5477	-1.8979	1.2384	-2.4155	1.0653	-22.6824
38	0.7475	-1.7557	1.1540	-2.2065	1.1983	-21.3593
39	0.8664	-1.7994	0.9445	-2.1001	1.1671	-17.6612
40	0.9586	-2.0214	0.6592	-2.1607	1.0978	-11.9321
41	1.0047	-2.4178	0.3354	-2.4503	1.0373	-5.5452
42	1.0103	-2.9933	0.1686	-3.0004	1.0174	-2.4072
43	-0.7971	0.5677	0.3297	-0.8770	0.6476	-76.7676
44	-0.7014	-0.2216	0.4867	-1.0041	0.0311	-58.1216
45	-0.5901	-0.9498	0.7144	-1.5067	-0.0332	-37.0336
46	-0.6139	-1.3988	0.8250	-1.9200	-0.0928	-32.2795
47	-0.6845	-1.6799	0.8147	-2.1369	-0.2275	-29.2902
48	-0.6137	-1.8405	0.6488	-2.1200	-0.3343	-23.3018
49	-0.2994	-1.9737	0.4595	-2.0915	-0.1816	-14.3806
50	0.1289	-1.8675	0.4288	-1.9557	0.2171	-11.6232
51	0.5240	-1.9396	0.4934	-2.0347	0.6192	-10.9150
52	0.8206	-1.9957	0.5138	-2.0882	0.9131	-10.1131
53	1.0253	-2.1675	0.4696	-2.2351	1.0930	-8.1953
54	1.1750	-2.4386	0.3562	-2.5229	1.2093	-5.5013
55	1.2528	-2.9641	0.1951	-2.9731	1.2618	-2.6429
56	1.2683	-3.5735	0.1047	-3.5758	1.2706	-1.2379
57	-1.1854	0.6024	0.2414	-1.2174	0.6344	-82.4454
58	-0.9834	-0.1248	0.3206	-1.0899	-0.0183	-71.6235
59	-0.7033	-0.7112	0.4184	-1.1256	-0.2888	-44.7351
60	-0.6088	-1.0104	0.4095	-1.2657	-0.3535	-31.9396
61	-0.5692	-1.1372	0.3069	-1.3137	-0.4427	-22.4020
62	-0.4428	-1.3267	0.1236	-1.3436	-0.4259	-7.8113
63	-0.1866	-1.5243	-0.0345	-1.5252	-0.1858	1.4745
64	0.1358	-1.5326	-0.0808	-1.5365	0.1397	2.7661

TABLE F-4. (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
65	0.4891	-1.7420	-0.0386	-1.7427	0.4898	0.9951
66	0.7921	-1.9274	0.0388	-1.9280	0.7926	-0.8177
67	1.0421	-2.1934	0.1025	-2.1966	1.0453	-1.8125
68	1.2425	-2.5835	0.1297	-2.5879	1.2469	-1.9337
69	1.3691	-3.1139	0.1016	-3.1153	1.3714	-1.2978
70	1.4078	-3.7464	0.0712	-3.7474	1.4088	-0.7910
71	-1.3649	0.6330	0.0715	-1.3674	0.6356	-87.9525
72	-1.0384	-0.0460	0.0583	-1.0917	-0.0427	-86.8097
73	-0.6914	-0.5312	0.0057	-0.6916	-0.5310	-87.9545
74	-0.5204	-0.7377	-0.0944	-0.7730	-0.4852	20.4911
75	-0.4247	-0.8560	-0.2357	-0.9598	-0.3209	23.7741
76	-0.2911	-0.9713	-0.3991	-1.1555	-0.1068	24.7924
77	-0.0940	-1.1639	-0.5139	-1.3708	0.1129	21.9266
78	0.1408	-1.1807	-0.5443	-1.3760	0.3360	19.7388
79	0.4125	-1.4180	-0.4928	-1.5422	0.5367	14.1505
80	0.6894	-1.6402	-0.3679	-1.6969	0.7461	8.7640
81	0.9433	-1.9321	-0.2085	-1.9472	0.9584	4.1248
82	1.1829	-2.3397	-0.0503	-2.3404	1.1837	0.8136
83	1.3850	-2.8992	0.0509	-2.8998	1.3856	-0.6909
84	1.4703	-3.5543	0.0783	-3.5555	1.4715	-0.3931
85	-1.2561	0.6693	-0.1682	-1.2707	0.6839	-4.9547
86	-0.9697	0.0373	-0.2851	-1.0448	0.1124	-14.7602
87	-0.5589	-0.3614	-0.4907	-0.9607	0.0404	-39.3117
88	-0.3782	-0.5010	-0.6589	-1.1014	0.2221	42.3330
89	-0.2782	-0.5742	-0.8143	-1.2539	0.4015	39.8496
90	-0.1706	-0.6544	-0.9514	-1.3942	0.5692	37.3676
91	-0.0357	-0.8130	-1.0280	-1.5234	0.6746	34.0448
92	0.1194	-0.7979	-1.0329	-1.4638	0.7912	33.0395
93	0.3076	-1.0017	-0.9567	-1.5063	0.8121	27.8080
94	0.5153	-1.1380	-0.7829	-1.4932	0.8205	21.2969
95	0.7319	-1.4317	-0.5337	-1.5562	0.8564	13.1299
96	0.9886	-1.7786	-0.2337	-1.7982	1.0082	4.7935
97	1.3132	-2.3127	0.0426	-2.3132	1.3137	-0.6735
98	1.4941	-2.9745	0.1601	-2.9802	1.4999	-2.0492
99	-0.7842	0.7082	-0.5066	-0.9399	0.8639	-17.0860

TABLE F-5. (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
100	-0.5906	0.1344	-0.7184	-1.0328	0.5766	-31.6111
101	-0.3165	-0.1725	-1.0554	-1.3023	0.8134	-43.0432
102	-0.1995	-0.2575	-1.2701	-1.4989	1.0419	44.3467
103	-0.1371	-0.2922	-1.4301	-1.6468	1.2176	43.4483
104	-0.0776	-0.3326	-1.5473	-1.7577	1.3475	42.6442
105	-0.0088	-0.4453	-1.5980	-1.8399	1.3858	41.1122
106	0.0687	-0.3780	-1.5827	-1.7530	1.4437	40.9034
107	0.1651	-0.5184	-1.4844	-1.6999	1.3466	38.5179
108	0.2771	-0.6218	-1.2741	-1.5234	1.1787	35.2839
109	0.4082	-0.7576	-0.9576	-1.2958	0.9464	29.3365
110	0.6146	-0.9537	-0.5246	-1.1130	0.7739	16.8926
111	1.1313	-1.2749	0.0551	-1.2761	1.1325	-1.3120
112	1.5147	-1.8123	0.3869	-1.8567	1.5591	-6.5474
113	-0.4477	0.7262	-0.7059	-0.7788	1.0573	-25.1291
114	-0.3319	0.1859	-0.9598	-1.0671	0.9211	-37.4527
115	-0.1697	-0.0715	-1.3533	-1.4748	1.2336	-43.9609
116	-0.1028	-0.1311	-1.5858	-1.7029	1.4689	44.7440
117	-0.0680	-0.1480	-1.7471	-1.8555	1.5395	44.3448
118	-0.0368	-0.1682	-1.8571	-1.9607	1.7558	43.9869
119	-0.0021	-0.2562	-1.8987	-2.0321	1.7738	43.0259
120	0.0367	-0.1606	-1.8758	-1.9404	1.8165	43.4942
121	0.0855	-0.2635	-1.7693	-1.8669	1.6880	42.1833
122	0.1440	-0.3174	-1.5461	-1.6499	1.4766	40.7571
123	0.2191	-0.3875	-1.2055	-1.3273	1.1589	37.9392
124	0.3744	-0.4366	-0.7150	-0.8908	0.7786	29.4735
125	1.0070	-0.6281	0.0612	-0.6304	1.0092	-2.1417
126	1.5283	-1.0448	0.5481	-1.1567	1.6402	-11.5384

TABLE F-6. AVERAGE NODAL STRESSES. WALL A_5

NODE	X STRESS	Y STRESS	XY STRESS	MIN STRESS	MAX STRESS	DIRECTION
1	0.2741	0.5985	0.1247	0.2317	0.6409	-71.2272
2	0.2038	-0.4044	0.2242	-0.4781	0.2775	-18.1971
3	0.1030	-1.7340	0.4502	-1.8384	0.2074	-13.0552
4	0.0146	-3.1505	0.7719	-3.3373	0.1923	-12.9639
5	-0.1595	-4.8709	1.3239	-5.2174	0.1870	-14.6674
6	-0.4969	-7.0093	2.3370	-7.7612	0.2549	-17.8333
7	-0.1283	-4.8196	3.3308	-6.5478	1.5999	-27.4231
8	0.3896	-1.0398	3.4399	-3.8385	3.1882	-35.1313
9	0.3026	-0.7704	2.9311	-3.2137	2.7459	-39.9139
10	0.2835	-0.6046	2.3836	-2.5351	2.2640	-39.7231
11	0.2851	-0.5676	1.8440	-2.0339	1.7513	-38.4906
12	0.3019	-0.6239	1.2776	-1.5199	1.1978	-35.0423
13	0.3145	-0.7593	0.6607	-1.0819	0.4271	-25.3197
14	0.3160	-1.1996	0.3378	-1.2714	0.3879	-12.0130
15	0.0949	0.5752	0.2145	0.0130	0.6571	-69.1177
16	0.0378	-0.3841	0.3519	-0.5834	0.2371	-29.5280
17	-0.0609	-1.5742	0.6310	-1.8028	0.1676	-19.9135
18	-0.1887	-2.7282	0.9503	-3.0444	0.1276	-19.4061
19	-0.4148	-3.9523	1.3751	-4.4239	0.0569	-18.9315
20	-0.6486	-5.2244	1.9325	-5.9313	0.0583	-20.0927
21	-0.2704	-3.9661	2.3978	-5.1454	0.9089	-26.1899
22	0.2925	-1.6225	2.5013	-3.3434	2.0133	-34.5267
23	0.4220	-1.2717	2.2923	-2.8686	2.0189	-34.3619
24	0.4761	-1.0671	1.9378	-2.3813	1.7903	-34.1444
25	0.5101	-1.0395	1.5175	-1.9685	1.4392	-31.4756
26	0.5494	-1.1521	1.0468	-1.6503	1.0476	-25.4500
27	0.5722	-1.3995	0.5350	-1.5353	0.7081	-14.2439
28	0.5743	-1.9918	0.2705	-1.9212	0.6036	-6.1870
29	-0.3339	0.5705	0.3326	-0.4430	0.6796	-71.8523
30	-0.3346	-0.2996	0.5056	-0.8230	0.1388	-45.9857

TABLE F-6 (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
31	-0.3692	-1.2304	0.8124	-1.7193	0.1197	-31.0583
32	-0.4952	-1.9428	1.0585	-2.5013	0.0633	-27.8174
33	-0.7020	-2.4873	1.2332	-3.1170	-0.0723	-27.0504
34	-0.7520	-2.7785	1.2127	-3.3455	-0.1850	-25.0593
35	-0.3875	-2.6559	1.0818	-3.0891	0.0457	-21.8229
36	0.1581	-2.1042	1.1244	-2.5680	0.6219	-22.4142
37	0.5362	-1.8755	1.2116	-2.3790	1.0397	-22.5676
38	0.7354	-1.7397	1.1288	-2.1772	1.1728	-21.1839
39	0.8531	-1.7869	0.9229	-2.0775	1.1437	-17.4792
40	0.9435	-2.0103	0.6429	-2.1441	1.0773	-11.7615
41	0.9877	-2.4067	0.3265	-2.4378	1.0188	-5.4440
42	0.9924	-2.9820	0.1638	-2.9888	0.9991	-2.3557
43	-0.8139	0.6081	0.3318	-0.8875	0.6817	-77.4528
44	-0.7157	-0.1309	0.4761	-0.9944	0.0978	-59.6600
45	-0.6001	-0.9085	0.6999	-1.4710	-0.0375	-38.7876
46	-0.6206	-1.3576	0.8082	-1.8773	-0.1009	-32.7444
47	-0.6883	-1.6405	0.7965	-2.0924	-0.2365	-29.5669
48	-0.6164	-1.8047	0.6307	-2.0771	-0.3441	-23.3560
49	-0.3041	-1.9424	0.4419	-2.0540	-0.1925	-14.1732
50	0.1210	-1.8410	0.4102	-1.9233	0.2032	-11.3450
51	0.5131	-1.9184	0.4732	-2.0073	0.6019	-10.6331
52	0.8070	-1.9792	0.4981	-2.0655	0.8934	-9.8369
53	1.0087	-2.1536	0.4501	-2.2165	1.0715	-7.9449
54	1.1543	-2.4753	0.3402	-2.5069	1.1859	-5.3085
55	1.2279	-2.9498	0.1855	-2.9580	1.2361	-2.5377
56	1.2415	-3.5584	0.0991	-3.5604	1.2435	-1.1829
57	-1.1926	0.6422	0.2340	-1.2219	0.6715	-82.8464
58	-0.9907	-0.0358	0.3107	-1.0871	0.0107	-72.7612
59	-0.7103	-0.6740	0.4051	-1.0977	-0.2866	-46.2822
60	-0.6142	-0.9759	0.3950	-1.2295	-0.3606	-32.6990
61	-0.5729	-1.1562	0.2922	-1.2774	-0.4517	-22.5252
62	-0.4459	-1.2996	0.1095	-1.3134	-0.4321	-7.1926
63	-0.1910	-1.5007	-0.0482	-1.5025	-0.1892	2.1044
64	0.1292	-1.5122	-0.0953	-1.5177	0.1347	3.3117

TABLE F-6. (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
65	0.4705	-1.7250	-0.0547	-1.7264	0.4718	1.4273
66	0.7788	-1.9137	0.0211	-1.9138	0.7790	-0.4480
67	1.0239	-2.1813	0.0840	-2.1835	1.0261	-1.5307
68	1.2174	-2.5707	0.1126	-2.5741	1.2208	-1.7010
69	1.3362	-3.0973	0.0899	-3.0991	1.3380	-1.1618
70	1.3710	-3.7287	0.0635	-3.7295	1.3718	-0.7137
71	-1.3637	0.6721	0.0655	-1.3658	0.6742	-88.1535
72	-1.0901	-0.0086	0.0506	-1.0924	-0.0062	-87.3246
73	-0.6964	-0.4977	-0.0040	-0.6965	-0.4977	-1.1587
74	-0.5253	-0.7088	-0.1048	-0.7563	-0.4778	24.3971
75	-0.4285	-0.8315	-0.2462	-0.9482	-0.3118	25.3489
76	-0.2943	-0.9508	-0.4094	-1.1473	-0.0979	25.6382
77	-0.0978	-1.1467	-0.5243	-1.3638	0.1196	22.6969
78	0.1355	-1.1661	-0.5555	-1.3709	0.3403	20.2425
79	0.4048	-1.4060	-0.5059	-1.5378	0.5365	14.5974
80	0.6779	-1.6308	-0.3836	-1.6929	0.7400	9.1904
81	0.9258	-1.9241	-0.2268	-1.9421	0.9437	4.5218
82	1.1552	-2.3303	-0.0702	-2.3317	1.1566	1.1533
83	1.3432	-2.8342	0.0344	-2.8845	1.3434	-0.4665
84	1.4206	-3.5353	0.0654	-3.5361	1.4214	-0.7559
85	-1.2495	0.7061	-0.1714	-1.2644	0.7210	-4.9725
86	-0.9678	0.0711	-0.2886	-1.0426	0.1459	-14.5280
87	-0.5626	-0.3336	-0.4950	-0.9562	0.0599	-38.4874
88	-0.3825	-0.4787	-0.6641	-1.0964	0.2352	42.9287
89	-0.2816	-0.5564	-0.8203	-1.2507	0.4127	40.2463
90	-0.1734	-0.6402	-0.9579	-1.3928	0.5791	38.1539
91	-0.0386	-0.8016	-1.0353	-1.5234	0.6832	34.3857
92	0.1158	-0.7877	-1.0413	-1.4710	0.7991	33.2746
93	0.3023	-0.9947	-0.9670	-1.5106	0.8182	28.0765
94	0.5070	-1.1334	-0.7955	-1.4955	0.8232	21.6503
95	0.7176	-1.4289	-0.5518	-1.5625	0.8512	13.6041
96	0.9613	-1.7754	-0.2574	-1.7994	0.9353	5.3271
97	1.2608	-2.3017	0.0164	-2.3018	1.2608	-0.2638
98	1.4254	-2.9568	0.1344	-2.9610	1.4295	-1.7551
99	-0.7778	0.7337	-0.5019	-0.9293	0.8852	-16.7944

TABLE F-6. (CONTINUED)

NODE	X STRESS	Y STRESS	XY STRESS	MIN.STRESS	MAX.STRESS	DIRECTION
100	-0.5884	0.1568	-0.7141	-1.0212	0.5897	-31.2240
101	-0.3189	-0.1556	-1.0529	-1.2933	0.3187	-42.7829
102	-0.2023	-0.2448	-1.2701	-1.4938	1.0467	44.5210
103	-0.1393	-0.2825	-1.4322	-1.6449	1.2231	43.5684
104	-0.0793	-0.3253	-1.5509	-1.7581	1.3535	42.7328
105	-0.0104	-0.4395	-1.6029	-1.8421	1.3922	41.1879
106	0.0669	-0.3736	-1.5887	-1.7573	1.4505	41.0540
107	0.1625	-0.5154	-1.4924	-1.7068	1.3540	38.6021
108	0.2730	-0.6205	-1.2852	-1.5344	1.1868	35.4156
109	0.3999	-0.7581	-0.9737	-1.3119	0.9538	29.6311
110	0.5941	-0.9554	-0.5498	-1.1307	0.7693	17.6815
111	1.0665	-1.2739	0.0120	-1.2739	1.0666	-0.2942
112	1.4151	-1.8082	0.3319	-1.8421	1.4489	-5.8180
113	-0.4433	0.7422	-0.6953	-0.7642	1.0632	-24.7773
114	-0.3303	0.1994	-0.9504	-1.0521	0.9212	-37.2141
115	-0.1713	-0.0620	-1.3472	-1.4649	1.2317	-43.8385
116	-0.1044	-0.1242	-1.5835	-1.6978	1.4692	44.8208
117	-0.0693	-0.1429	-1.7475	-1.8540	1.6418	44.3967
118	-0.0377	-0.1644	-1.8595	-1.9617	1.7596	44.0246
119	-0.0029	-0.2533	-1.9026	-2.0348	1.7786	43.1173
120	0.0358	-0.1585	-1.8809	-1.9448	1.8221	43.5214
121	0.0843	-0.2622	-1.7763	-1.8736	1.6957	42.2144
122	0.1420	-0.3171	-1.5559	-1.6603	1.4851	40.8037
123	0.2145	-0.3887	-1.2209	-1.3438	1.1596	38.0573
124	0.3595	-0.4896	-0.7397	-0.9179	0.7878	30.0727
125	0.9358	-0.6341	0.0076	-0.6341	0.9358	-0.2782
126	1.4093	-1.0527	0.4726	-1.1403	1.4969	-10.5022