

AD-755 096

DESIGN AND TESTING OF A BLAST-RESISTANT
REINFORCED CONCRETE SLAB SYSTEM

Marvin E. Criswell

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Prepared for:

Army Engineer Waterways Experiment Station

November 1972

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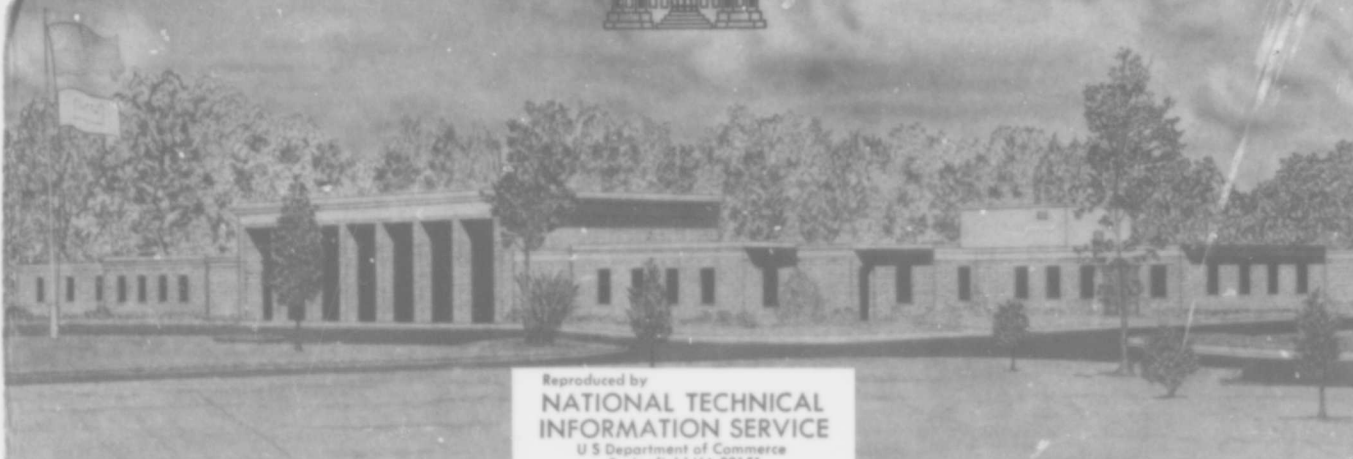


TECHNICAL REPORT N-72-10

DESIGN AND TESTING OF A BLAST-RESISTANT REINFORCED CONCRETE SLAB SYSTEM

by

M. E. Criswell



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Unclassified

Security Classification

DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY (Corporate author) U. S. Army Engineer Waterways Experiment Station Vicksburg, Mississippi		2a. REPORT SECURITY CLASSIFICATION Unclassified	
		2b. GROUP	
3. REPORT TITLE SUMMARY OF DESIGN AND TESTING OF A BLAST-RESISTANT REINFORCED CONCRETE SLAB SYSTEM			
4. DESCRIPTIVE NOTES (Type of report and inclusive dates) Final report			
5. AUTHOR(S) (First name, middle initial, last name) Marvin E. Criswell			
6. REPORT DATE November 1972		7a. TOTAL NO. OF PAGES 316	7b. NO. OF REFS 57
8a. CONTRACT OR GRANT NO. Work Order No. DAHC20-68-W-0192		8b. ORIGINATOR'S REPORT NUMBER(S) Technical Report N-72-10	
8c. PROJECT NO. c. Work Unit 1127E		8d. OTHER REPORT NO(S) (Any other numbers that may be assigned this report)	
10. DISTRIBUTION STATEMENT Approved for public release; distribution unlimited.			
11. SUPPLEMENTARY NOTES		12. SPONSORING MILITARY ACTIVITY Defense Civil Preparedness Agency Washington, D. C.	
13. ABSTRACT The objectives of this investigation were to design and model test a blast-resistant reinforced concrete slab system serving as the roof of a basement shelter area. The slab system was designed to offer sufficient radiation and blast protection to insure a survival probability for its occupants of 85 to 95 percent for a 15-psi airblast overpressure loading. Static and dynamic tests were conducted on two 1/4-scale models of a prototype shelter. The prototype shelter, as designed, had a reinforced concrete flat slab roof consisting of three 18-foot spans in each direction supported by four interior columns and by a continuous wall around the perimeter. The model included the perimeter walls and interior columns, thereby including the effects of soil loading on the walls and different panel configurations which would influence the load-carrying capacity of the prototype structure. The slab system was designed using the empirical method of the 1963 American Concrete Institute Code with modifications to account for the dynamic loading effects. Flexural yield-line analysis which assumed a 15 percent strength increase because of the effects of in-plane forces was used to determine the design strength of the slab system. The models were statically and dynamically tested in the 22-foot-10-inch-diameter Large Blast Load Generator at the U. S. Army Engineer Waterways Experiment Station. The models were placed in the chamber, and a fine uniform sand was used as backfill around the models. Approximately 90 channels of instrumentation were recorded on each test. These included column loads, overpressures, soil pressures against the walls, slab deflections, concrete and steel strains, and (for the dynamic tests only) accelerations. The static test model supported a maximum load of 26.6 psi including dead loads. As the flexural capacity of the slab was approached, shear distress developed around the column capitals, causing a sudden increase in slab deflections and a corresponding reduction in load-carrying capacity. The dynamic model was tested twice. In the first test, the model was subjected to a load of 19.7 psi. The model incurred very little damage from this loading and responded largely within the elastic range. However, the slab system suffered extensive damage during a second loading of 30.1 psi. The slab failure was very similar to that of the statically tested model. In-plane soil forces increased the strengths of the slabs tested by approximately 30 percent over that computed by yield-line analysis. Some other factors which increased the strengths of the slabs over the calculated design value were conservative assumptions in the flexural design procedure regarding the effective slab depth and yield-line placement and the use of round column capitals. The increased slab strengths obtained point out the importance of conservative column design since factors which increase the strength of the slab will increase the loads applied to the columns but not the column strength.			

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this document may be better
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Sponsored by **Defense Civil Preparedness Agency**

Work Order No. DAHC20-68-W-0192, Work Unit 1127E

Conducted by **U. S. Army Engineer Waterways Experiment Station**
Weapons Effects Laboratory
Vicksburg, Mississippi

ARMY-MRC VICKSBURG, MISS.

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SUMMARY

This investigation concerned the design and model testing of a blast-loaded reinforced concrete slab system serving as the roof of a basement shelter area. The slab system was designed to provide radiation and blast protection sufficient to insure a survival probability of at least 85 to 95 percent for shelter inhabitants for a 15-psi airblast overpressure loading.

The slab system roof for the prototype shelter consisted of three 18-foot spans in each direction supported by four interior columns and by a continuous wall around the perimeter. The shelter was assumed to be backfilled to the level of the top of the roof slab. This prototype configuration allowed modeling of a multiple-panel slab system at a reasonable scale, and with the effects of the wall and soil behavior included without an unduly complex and nonsymmetrical structure.

The flat slab system, including column capitals and drop panels but with no beams spanning between columns, was found to be cost-competitive with one- and two-way slab systems considered in a preliminary design study and was selected partially because of the lack of experimental information on flat slabs loaded to failure and the need for the multiple-panel configuration to accurately model the system.

A dynamic analysis was used to establish an equivalent static loading on the slab. Flexural yield-line methods using conservative assumptions were employed to determine the strength of the slab system. In-plane slab forces resulting from the soil pressure loading on the walls were assumed to increase the slab strength by 15 percent. Flexural moments were distributed according to the empirical method of the 1963 ACI Building Code for Reinforced Concrete. Grade 40 reinforcement and normal-weight concrete with a design compressive strength of 4,000 psi were assumed. The design included an 11-inch-thick slab with 16-inch-thick drop panels, 66-inch-diameter column capitals, and slab tensile reinforcement ratios varying from 0.0039 to 0.0100. Tensile reinforcement was extended to provide compressive reinforcement at all locations equal to approximately one-half the tensile reinforcement. No shear reinforcement was included.

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Two 1/4-scale models (165 inches square) of the prototype slab system, walls, and columns were constructed using small-aggregate concrete and 3/16-inch-diameter rods for slab reinforcement. A separate base slab served as a foundation for the model. Forms for the model consisted primarily of dimension lumber and plywood and were assembled atop the base slab. The model walls were reinforced with welded wire fabric. The slab, columns, and walls of the models were cast monolithically. One model was prepared for static testing and the other for dynamic testing. Reinforcement ratios for the models were increased 20 percent to compensate for the 36.9-ksi yield strength of the reinforcement used being only 5/6 the expected 44-ksi average yield strength for Grade 40 reinforcement.

The U. S. Army Engineer Waterways Experiment Station's Large Blast Load Generator (LBLG) was used for both static and dynamic testing. The models were placed in the 22-foot-10-inch-diameter test chamber of the LBLG with a fine uniform sand used as the backfill material. The approximately 90 channels of instrumentation used in each test included load cells under each column, overpressure transducers, two pressure transducers inside the structure (for dynamic tests only), eight soil-structure pressure transducers on one-half of one wall, nine accelerometers (for dynamic tests only), deflection gages at 13 points on the slab, including at the center of each panel, numerous strain gages on the tensile reinforcement in one direction in one-quarter of the slab, and other strain gages placed on the compression reinforcement of the slab, on the column and wall reinforcement, and on the compressive surface of the concrete.

The statically tested model was loaded with slowly applied water pressure. The model supported a maximum load of 26.6 psi, including dead load. Although the flexural strength of the slab was approached, shear distress developed around the perimeters of the column capitals when the center deflections of the outer panels were near 1-1/4 inch and limited the slab capacity. Well-anchored bottom bars extended into the column capital areas and were sufficient in preventing the slab from falling from the columns. However, the negative moment capacity of the slab near the columns decreased enough that the resistance of the slab

floor system will be the cause of much debris, which may cause serious injury to shelter occupants unless reinforcement is placed in the drop panels to tie them into the rest of the slab.

4. The basement walls serve two important functions for the slab system, i.e. they provide (1) restraint needed for ultimate moment at the edge of the slab to be fully utilized, and (2) a means of gathering the soil forces, thus providing in-plane compression on the slab.

5. Adequacy of the columns must be carefully studied. Unless the effects that can increase the slab's strength and thereby the column loads are considered in the design of the columns, the failure mode of the entire shelter area could easily be controlled by column failure.

dropped approximately 40 percent. The slab resistance with further load was 15 to 17 psi until an anchorage failure in a bottom bar splice zone near two column capitals terminated the test. Deflections at the centers of panels at that time exceeded 7 inches.

The first loading of the dynamically tested model was at an overpressure of 19.7 psi and a rise time of 8-1/2 msec. The model sustained very little damage and performed largely within its elastic range. Cracking and some yielding of the reinforcement occurred at the negative moment section of the slab over the inner face of the wall. Minor cracking along the centers of some panels occurred on the underside of the slab.

The slab system was extensively damaged by a second dynamic loading of 30.1-psi overpressure with a rise time of 4-1/2 msec. Shear distress in the column areas developed about 30 msec into the test. The bottom bar splice zones in the corner and edge panels near the columns soon failed, allowing pressure to enter into the slab. The mechanism of slab failure was similar to that of the statically loaded structure. The interior panel of the slab remained attached to the columns and pulled the columns in toward the center of the model. The slab deflected 13 to 15 inches over the area inside the center of the outer row of panels. The columns displayed distress, which indicated that they were loaded to very near their capacity.

Some of the significant findings from the model test program are as follows:

1. A flat slab floor system such as that tested in this program can be designed to withstand blast loadings of relatively long duration (on the order of 1 second or more) using design methods closely tied to standard methods for designing slowly loaded structures.
2. The in-plane forces in the slab caused by horizontal soil pressures acting on the perimeter wall of the structure accounted for about 30 percent of the 48 percent increase in the strength of the statically tested model above the computed yield-line strength disregarding in-plane forces.
3. At large deflections, the drop panels used in the flat slab

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PREFACE

The study reported herein was conducted by the U. S. Army Engineer Waterways Experiment Station (WES) under the sponsorship of the Defense Civil Preparedness Agency (formerly the Office of Civil Defense) and conducted under Work Order No. DAHC20-68-W-0192, Work Unit 1127E.

The investigation was conducted during the period June 1968 to December 1971 under the general supervision of Mr. G. L. Arbuthnot, Jr., Chief of the Weapons Effects Laboratory, WES, and under the direct supervision of Mr. W. J. Flathau, Chief of the Protective Structures Branch, WES. This report was prepared by Dr. M. E. Criswell, Assistant Professor of Civil Engineering, Colorado State University, formerly a project manager in the Protective Structures Branch, WES. Acknowledgment is made to Mr. W. L. Huff, Project Engineer, Protective Structures Branch, WES, for his efforts in assembling this report and continuing the test program after Dr. Criswell's departure from WES.

Directors of WES during the investigation and the preparation of this report were COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE. Technical Directors were Messrs. J. B. Tiffany and F. R. Brown.

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NOTATION

A_s	Area of reinforcement steel
b	Width of the web of a beam; also perimeter of a section located $d/2$ away from the column outline for a slab
c	Effective support size, column capital diameter
d	Effective depth of a reinforced concrete section, the distance from the compressive face to the centroid of the reinforcement
d'	Effective slab depth less collar recess, if any
D	Column diameter
D_p^+	Duration of overpressure positive phase
e	Eccentricity of the applied load on a compression member measured from the plastic centroid of the member
E	Young's modulus of elasticity
f'_c	Compressive cylinder strength of concrete
f_{ce}	Average effective concrete prestress
f_y	Yield strength of the steel reinforcement
F_e	Effective prestressing force per cable
g	Acceleration of gravity, 32.2 ft/sec^2
L	Column spacing, span length of a flat slab measured from center to center of supports
M_b	Moment capacity at simultaneous crushing of concrete and yielding of tension steel (balanced condition)
M_o	Numerical sum of assumed positive and average negative moments at the critical design sections of a flat slab
M_u	Ultimate resisting moment of a reinforced concrete section
p	Tensile reinforcement ratio of a flexural member
p'	Compressive reinforcement ratio
p_m	Applied pressure
P	In-plane compression
P_b	Axial load capacity at simultaneous crushing of concrete and yielding of tension steel (balanced condition)
P_i	Overpressure of the i^{th} triangle
P_{so}	Static overpressure
PF	Radiation protection factor
q_s	Static design resistance

q_{sv}	Equivalent static load to be resisted by shear
q_y	Dynamic design resistance
Q	Yield-line slab strength
r	Column diameter (from Reference 26)
R	Length reduction factor
S	Spacing of prestressing cables
t	Slab thickness; also time
t_D	Duration time of positive pressure loading
t_i	Duration of i^{th} overpressure triangle
t_r	Load rise time
T	Natural period of vibration
U	Total design load
v	Unit shear stress
v_u	Ultimate unit shear stress
V	Total shear transferred from slab to column
V_u	Maximum shear supported by columns
W	Total design load on a panel
X_y	System deflection at yield
ϵ_{cu}	Crushing strain of concrete, 0.003 in/in
μ	Ductility ratio
$\emptyset$	Capacity reduction factor; also diameter

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used herein may be converted to metric units as follows:

Multiply	By	To Obtain
mils	0.0254	millimeters
inches	2.54	centimeters
square inches	6.4516	square centimeters
feet	0.3048	meters
cubic feet	0.0283168	cubic meters
cubic yards	0.764555	cubic meters
gallons (U. S. liquid)	3.785412	cubic decimeters
pounds	0.45359237	kilograms
kip	453.59237	kilograms
tons (2,000 pounds)	907.185	kilograms
megatons	0.9071847	gigagrams
pounds per inch	0.1751268	kilonewtons per meter
pounds per square inch	0.6894757	newtons per square centimeter
pounds per square foot	47.88026	newtons per square meter
pounds per cubic foot	16.0185	kilograms per cubic meter
kip per square inch	6.894757	meganewtons per square meter
inch-kip	0.0001129848	meter-kilonewtons
inch-pounds per inch	0.4535924	meter-newtons per meter
inches per second	2.54	centimeters per second
feet per second	0.3048	meters per second
Fahrenheit degrees	5/9	Celsius or Kelvin degrees ^a

^a To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Few existing shelter areas, especially those that are at appreciable heights above ground level, would be able to withstand or offer significant protection from the effects of an airblast peak overpressure approaching 15 psi¹ (2,160 psf). One possible method of providing shelters that would remain functional after exposure to an overpressure of this level is that of "slanting" the design of basement areas in new construction for shelter use. Slanting involves designing shelter areas to provide protection against combined nuclear weapons effects at a given overpressure level as well as designing a structure to satisfy the requirements for day-to-day use.

One problem involved in the slanting process is to design and construct an economical basement ceiling system with the necessary structural strength. Reinforced concrete slab systems are an attractive type of basement ceiling system because of the structural strength achievable at a relatively low cost and because concrete slabs generally have the mass required to adequately attenuate the radiation and heat from the environment outside the shelter.

1.2 OBJECTIVES

The objectives of the program discussed herein were (1) to design a reinforced concrete slab system using a common slab-type system as a ceiling over a basement shelter area that will provide the protection necessary to insure a survival probability of 85 to 95 percent of the shelter inhabitants when the shelter area is subjected to a 15-psi airblast overpressure level from a multimegaton nuclear weapon, and (2) to investigate the performance of the slab system by model testing the

¹ A table of factors for converting British units of measurement to metric units is presented on page 18.

design under both static and dynamic loadings.

1.3 SCOPE

Before the slab system to be modeled was selected, preliminary designs were prepared for several basic reinforced concrete slab systems having identical design loads and column spacings. The criteria for selecting the slab system for further study were (1) economy of material; (2) probable acceptance of the system by architects, design engineers, contractors, and building owners; (3) compatibility of the slab (structural) system with other building subsystems; (4) probable ductility and large-deflection behavior of the slab system; (5) the need for research to understand the behavior of the slab, its resistance at large deflections, and the mode of failure; and (6) the need for using a complex multipanel model rather than a simpler model for a portion of the slab system to answer the research questions. A more detailed design was then prepared using the selected slab system for a reasonably simple structure.

The prototype structure selected consisted of a basement area 54 feet square having a reinforced concrete flat slab roof with three 18-foot spans in each direction. Reinforced concrete walls cast monolithically with the slab supported the perimeter of the slab, and four columns served as interior supports. The structure included the slab panels supported on either one or two adjacent sides by walls and one interior panel. It was large enough to reasonably represent an actual structure yet simple enough to be modeled at a reasonable scale.

The models constructed for the laboratory investigation were 1/4-scale representations of the prototype structure. The basement walls and the soil surrounding the structure were included in order to model the conditions that would exist around the perimeter of the prototype slab as accurately as possible. The restraint (or fixity) offered to the slab by the wall and the in-plane forces resulting from a perimeter wall, especially a wall loaded with soil forces, were expected to be very important in determining the behavior of the slab system.

Two 1/4-scale (13-1/2-foot-square) multipanel models were

constructed. One was slowly loaded to failure using water pressure; the other was subjected to blast loadings at two pressure levels. Both models were tested in the Large Blast Load Generator (LBLG) at the U. S. Army Engineer Waterways Experiment Station (WES).

CHAPTER 2

SELECTION OF SLAB SYSTEM AND DESIGN OF PROTOTYPE SYSTEM AND SCALE MODELS

The selection of the slab system, the design of the prototype system, and the design of the 1/4-scale models are described in this chapter. First the design requirements for the basement shelter area are reviewed, and then four preliminary slab designs are summarized and the rationale behind the selection of the flat slab system used in the test models are presented. A detailed design of the prototype slab system, including a dynamic loading analysis and a yield-line analysis of the flexural behavior of the slab, follows. Next, the loads and changes needed to reproduce the prototype design using the 1963 American Concrete Institute (ACI) building code requirements (Reference 1) and the application of the recently adopted 1971 ACI building code (Reference 2) are examined. Finally, the scaling down of the prototype design to the 1/4-scale model and details of this model are described.

2.1 DESIGN REQUIREMENTS

The slab system was designed to satisfy the following basic requirements:

1. The slab system was to form the ceiling of a "fully slanted" basement shelter area. The process of full slanting entails the inclusion of a minimum number of design changes involving modest additional costs to the conventional design of a structure so that the area can offer a desired level of protection against all nuclear weapons effects and still fully maintain its intended day-to-day functions (Reference 3).

2. The shelter area was to provide a survival probability of from 85 to 95 percent against all nuclear weapons effects from a large-yield weapon at the 15-psi airblast overpressure exposure level. To provide adequate fallout radiation protection, the shelter was to provide a fallout radiation protection factor (PF) of at least 100. The basement area was assumed to be entirely below grade.

The following decisions and assumptions were made prior to design of the slab:

1. In keeping with the concept of full slanting, the slab configuration and design were to be kept as similar as practical to conventional design practice as used in commercial construction. It was thought that this approach would, by requiring a minimum of new design and construction procedures, minimize any designer and/or builder resistance that might result with an exotic slab design and would also serve to minimize the cost of slanting the structural system for shelter use.

2. The basement walls were to be designed as structural elements. Also, the slab was to be cast monolithically with and reinforced continually with the walls, resulting in sizable edge fixity around the slab perimeter.

3. Interior supports in the shelter area were assumed to be columns rather than walls. Column supports were chosen because a column-support system is more critical than a wall-support system in governing slab performance and also because the column-support system commonly exists due to the large room sizes desired in many basement areas.

4. Materials were to include Grade 40 reinforcement (40,000-psi minimum yield strength) satisfying the requirements of ASTM Standard A 615-68 (Reference 4) and normal-weight concrete with a design compressive strength of 4,000 psi.

5. The structure was to consist of three 18-foot spans in each direction with continuous walls around the slab perimeter. This configuration includes corner, exterior, and interior (all four sides continuous) slab panels; avoids the complications of rectangular and odd-shaped panels; and is simple enough to allow modeling of the prototype, yet complex enough to reasonably represent an actual basement shelter area.

2.2 SELECTION OF THE SLAB SYSTEM

2.2.1 Systems Considered. The first step in the selection of the slab system to be designed in detail and model tested was the preparation of preliminary designs for the 3-span, 54-foot-square prototype shelter using the following four reinforced concrete slab systems, all

of which are used in conventional construction: (1) flat slab with drop panels and column capitals, (2) flat slab with drop panels and shear-heads, (3) two-way slab, and (4) one-way slab. Waffle slabs and joist slabs were not considered because they are basically one of the above types with some concrete removed from tension areas (concrete which would be needed in shelter areas for radiation protection). The flat plate, a widely used variation of the flat slab with a slab thickness equal to the needed drop panel depth and a column size equal to the needed capital size, was not chosen as one of the design options. The magnitude of shear to be transferred at the slab-column connection of the flat plate would require either a much greater slab thickness than would be needed for flexural resistance or radiation shielding and/or would require extremely large columns. Prestressed systems also were not considered.

The preliminary designs were analyzed to determine the approximate slab thicknesses and reinforcement ratios needed and to establish if any one slab system was clearly superior or inferior to the other three for use in slanted shelters.

2.2.2 Preliminary Design Guidelines. In addition to the requirements and design decisions stated in Section 2.1, the preliminary designs were based on the following:

1. The 15-psi overpressure dynamic loading (plus the slab dead-weight) was approximated as being equivalent to a static loading of 14 psi. Because of the finite rise time present (due primarily to the crossing time, i.e. the time needed for the pressure front to pass over the structure) and because of the ductility that a properly designed and detailed concrete slab should possess, the material strength increases resulting from the rapid loading rates were expected to exceed the ratio of required resistance for a dynamic loading to the maximum dynamic loading, q_y/p_m . Calculations in Section 2.3.2 show that this assumption is true.

2. An upward load of one-third the downward load, or 4.67 psi, was used to represent the effects of rebound, the negative overpressure

phase, and any pressure ingress into the shelter. This load also provided some compression reinforcement at all sections.

3. A live load of 100 psf was assumed as the design value for the conventional use of the slab.

4. The provisions of the 1963 ACI building code for ultimate strength design (Reference 1), including the capacity reduction or ϕ ($\emptyset$) factors, were used except in cases in which previously conducted tests had shown that modifications or other procedures were more valid.

5. The effects of compressive in-plane forces due to soil forces acting on the wall and to the geometry of the slab-wall system itself and postyield strength effects from strain hardening and tensile membrane action were not included in the analysis.

6. The basement walls were assumed to be 12 inches thick and reinforced to support the moments from the slab and those from an inwardly directed soil force of from 0.3 to 0.9 times the 14-psi downward load, with the bases of the walls assumed to be either pinned or fixed.

7. Shear reinforcement in the slab was to be avoided unless absolutely necessary on the basis that a less expensive alternative to a complex system of stirrups in the slab would be to use a thicker slab. Also, the effectiveness of most schemes in reinforcing concrete slabs for shear is considered problematical (Reference 5). Slabs reinforced for shear throughout are not used in conventional construction and would likely be quite costly. Bent-up flexural reinforcement alone could not provide shear reinforcement in all the slab locations needed. Stirrups were used where needed in the beams of the one- and two-way systems.

8. The columns were designed to sustain 150 percent of the slab reaction. Although in-plane forces could increase the slab capacity and thereby the column loads, they would not increase the column capacity. A column failure would be a very undesirable and easily avoided mode of failure.

2.2.3 Results of Analyses of Preliminary Designs. The slab configurations determined to be necessary, along with the range of reinforcement ratios and the quantities of reinforcement steel and concrete needed for the four slab systems, are shown in Figures 2.1 through 2.4.

The walls were included in the designs because their design and steel quantities would be dependent upon the slab system employed. The foundation and basement floor were excluded from this study. Although no rigorous (and necessarily lengthy) study was included to minimize costs of slabs and especially beams by optimizing dimensions and reinforcement ratios, an attempt was made to provide an economical and reasonable design for each alternative considered.

Notable changes from the 1963 ACI code (Reference 1) and limiting factors in the design of each alternative were as follows for the four slab systems:

1. Flat slab with capitals. Two major design changes were included. First, the slab was designed to have sufficient moment capacity to satisfy the requirements of static equilibrium (Reference 6) rather than to provide as low as 80 percent of statics, as is allowed for the ultimate strength design method in the 1963 ACI building code (as low as 72 percent of statics was specified by the 1963 and earlier ACI building codes for the working stress design method (Reference 7)). The equation in Article 2101(e)1 of ACI 318-63 for total moment for a flat slab panel, M_o , was replaced with:

$$M_o = 0.125WL \left(1 - \frac{2c}{3L}\right)^2 \quad (2.1)$$

Where: W = total design load on a panel

L = span length of a flat slab panel measured from center to center of supports

c = effective support size

Second, the allowable shearing stress around the columns was decreased 25 percent from that allowed by ACI 318-63 because of experimental evidence indicating that the ACI code value may be excessive in slab-column connections in which column size to slab thickness ratios are greater than 3 and in which extensive flexural yielding in the slab adjacent to the column occurs (Reference 8), a combination expected with this alternative.

The 12-inch slab thickness was necessary to provide adequate shear

strength around the slab perimeter without using slab stirrups. Drop panel depth and column capital size were controlled by shear requirements at the columns. Flexural reinforcement ratios were quite low throughout the slab.

2. Flat slab with shearheads. The use of shearheads to increase the shear resistance of flat slabs and plates was investigated in 1968 (Reference 9). The suggested design methods in Reference 9, now included in the 1971 ACI code (Reference 2), were used.

This flat slab was also designed to have a moment capacity satisfying statics rather than the lesser amount specified by ACI 318-63.

Basically, the same items controlled this design as controlled the design of the flat slab with column capitals. The smaller interior support size (24-inch diameter versus 66-inch diameter) led to an approximately 20 percent increase in the slab moments and a corresponding increase in the flexural reinforcement requirements.

3. Two-way slab. The procedure in ACI 318-63 for determining design moments for two-way slabs considers the possibility of pattern loads and specifies maximum moments. As a result, design moments for the slab system from ACI 318-63 (Method 1 from Appendix A, "Design of Two-Way Slabs," was used) provide 120 percent of the resistance needed to satisfy statics. These moments were reduced to 100 percent of statics in the preliminary design of the two-way slab system in order to provide a design with a computed flexural resistance equal to that of the flat slab alternatives.

The 10-inch slab thickness chosen was the minimum thickness that would satisfy shearing stress requirements around the slab perimeter without shear reinforcement. Flexural reinforcement ratios in the slab were all quite low (below 1 percent).

Maximum shearing stresses in the beams were over four times those allowed by ACI 318-63 without stirrups and 81 percent of the maximum stress of $10\phi\sqrt{f'_c}$ allowed for beams with stirrups, where ϕ is a capacity reduction factor and f'_c is the compressive cylinder strength of concrete. The maximum beam reinforcement ratio of 2.79 is 75 percent of that allowed without compression steel.

4. One-way slab. The moments for the one-way slab and beams were reduced from the 122 percent of statics provided by the methods of ACI 318-63 to 100 percent of statics.

As was the case with the two-way slab, shear determined the slab thickness and was the primary influence in determining the beam size.

2.2.4 Cost Comparison. Approximate relative costs of the four systems were computed from the amounts of materials included in the slab and perimeter walls for each alternative considered. For this comparison, shown in tabular form below, the cost of 1 ton of reinforcement steel was assumed to be 10 times that of 1 yd³ of concrete (from Reference 10, the cost of approximately 4,000-psi ready-mix concrete in 20 major U. S. cities in March 1971 ranged from \$14.25/yd³ to \$22.50/yd³, with an average of \$17.60/yd³; from Reference 11, the base price of Grade 40 reinforcement, FOB warehouse, at this same time ranged from \$148/ton to \$220/ton, averaging \$180/ton). In this comparison, the influences of differing costs for formwork, labor, equipment, overhead, etc., for the four alternatives were neglected.

System	Concrete Needed	Steel Needed	Cost Index ^a	Relative Cost ^b
	yd ³	tons		
Flat slab with capitals	199	19.5	394	100.0
Flat slab with shearheads	197	23.4	431	109.3
Two-way slab	194	20.8	402	102.0
One-way slab	226	18.1	407	103.2

^a (Concrete) + 10 (steel)

^b Lowest = 100

Thus, except for the flat slab using shearheads, the amounts of materials in the slab systems were, for all practical purposes, nearly equal. It should be noted that without the correction of the moments given by ACI 318-63 to 100 percent of statics, the designs for the flat slabs would have clearly required the smallest amounts of materials.

2.2.5 Selection of the Flat Slab with Capitals. The final

selection from the three alternatives using nearly equal amounts of materials (the flat slab with capitals, the two-way slab, and the one-way slab) was determined after considering the following questions, some necessarily requiring answers based on engineering judgment:

1. Which system would minimize the cost of the rest of the shelter area and be most acceptable to the architect and building occupants?
2. For which system would the expected postyield behavior (large deflection) be most desirable?
3. For which system would the potential information to be gained best justify the construction and testing of elaborate models?

The lack of beams results in the flat slab with capitals generally being accepted as superior to the one-way and especially to the two-way slab systems (but not to the flat plate) with respect to minimizing the cost of mechanical systems. Beams in a slab system interfere with the running of ductwork and utilities; often these are placed below the beams, and the slab system effectively becomes as deep as the beams. This may also increase the floor to top-of-slab distance (or depth of basement) needed to satisfy either the requirements for the shelter or those for everyday use of the area. The very widespread use of flat slab systems in heavily loaded structures such as garages, warehouses, stores, and manufacturing areas attests to the acceptance of this system by architects and users of structures.

The postyield behavior of two-way slabs has received considerable attention (see Appendix C), and the two-way slab system is quite favorable for shelter use. If the beams in such a system are strong enough not to fail, they can anchor tensile forces from the slab panels (i.e. act as beam struts) allowing sizable deflections and tensile membrane strength to be developed in the slab. The mobilization of as favorable a postyield behavior in the one-way and flat slab systems is much more problematical due to the lack of distinct beam struts along all column lines. Any postyield behavior of the slab would certainly involve an interaction with the wall.

The flat slab system was selected as the system which would best justify constructing and testing a complex structural model, both

from the amount of potentially useful information on slab strength and behavior it would offer and from the need for such a model to reasonably investigate the slab system. Results of tests of restrained and continuous beams offer considerable guidelines in predicting the behavior and strength of one-way slabs. Two-way slab systems can be much more realistically studied by single panel structures surrounded by beams, such as those discussed in Reference 12, than can a flat slab system. Indeed, a single panel representation of a flat slab system would be most difficult to achieve, and the results of tests of such a representation would be questionable. Also, compared with the available experimental knowledge concerning the large-deflection behavior of two-way slab panels and systems, very little is known about the potential large-deflection behavior of flat slabs, especially those with sizable restraint from surrounding walls and reinforced such that punching shear distress around the columns would not immediately lead to the effective total failure of the slab by the separation of the column from the slab.

Based on the information presented in the discussion above, the flat slab system with capitals was chosen for further design and model testing, with the two-way slab being a close second choice.

2.3 DESIGN PROCEDURE AND CALCULATIONS FOR THE PROTOTYPE SLAB DESIGN

The design of the prototype flat slab system entailed the following steps:

1. Static resistances. The static design resistances of the slab elements in shear and flexure were selected based on both the specifications of the 1963 ACI building code (Reference 1) and on published experimental results.

2. Dynamic analysis. A dynamic analysis was made of a structure having the dimensions anticipated for the final design. Rebound and other factors causing upward loadings were examined. The results of the dynamic analysis together with the effects of rapid straining on material strengths were then used to calculate the static design resistance required to withstand the 15-psi design overpressure.

3. Flexural design. The first step in the flexural design was to

distribute design moments for the slab according to the 1963 ACI building code (Reference 1) for an arbitrary ultimate static load of 10 psi. A yield-line flexural analysis using the methods presented in Reference 13 was then made for this structure to determine the yield-line flexural strength of the nominal 10-psi-capacity slab design and to determine the proportion of the reactions going to the walls and columns (this information was needed for the shear design). All static design moments were then proportionately increased to provide the needed static strength determined in Step 2.

4. Shear design. The adequacy of the slab in shear at the critical locations was examined, and necessary changes in the slab dimensions were made.

5. Detailing of the slab. With the slab thicknesses and design moments established, the reinforcement was then proportioned and the reinforcement layout prepared with attention given to bar cutoff points and splice lengths.

6. Column design. The columns were then designed to support 175 percent of the column load computed from the distribution of reactions determined by the yield-line analysis. A factor of this order of magnitude is necessary because the column strength cannot be increased by the in-plane actions, although this can appreciably increase slab strength and thereby the column loads.

The design procedure described above entailed determining a needed static resistance by dividing the required resistance determined from the dynamic analysis, q_y , by the ratio of dynamic to static material strengths and then designing on the basis of this required static resistance. This is a deviation from the more common procedure of including the strain rate effects on the material strengths directly by increasing material strengths and designing for the required resistance, q_y . However, the two procedures are equivalent, i.e., reducing the design loads accomplishes the same thing as increasing the allowable material stresses. The equivalent static strength procedure was used primarily to facilitate determining what modifications would be necessary in a standard building code such as ACI 318-63 (Reference 1) or the

1971 ACI building code (Reference 2) in order that a designer could reproduce the slab design (see Section 2.4).

In the design procedure that follows, two references will be used frequently. References to the 1963 ACI building code (Reference 1) will be given as ACI followed by section cited; e.g. ACI 1601(a). References to the Air Force Design Manual (Reference 14) will be given as AFDM followed by the page number(s); e.g. AFDM, p. 9-15.

Except in Section 2.4, the capacity reduction factor, or factors (ϕ), for the ultimate strength design procedure in the ACI code will be taken as unity rather than the values given in ACI 1504. The ϕ factors are coefficients related to the probability that small adverse variations in material strengths, dimensions, workmanship, and other factors may combine to result in an undercapacity or strength below the computed ultimate strength. The computed ultimate strength is that given by the equations with the ϕ factor omitted. Indeed, if the expected (or average) strength is desired, a coefficient greater than 1 should be used because the average reinforcement strength is more than 10 percent greater than the design values (AFDM, p. 6-5), and the concrete strength averages at least this much above the design values (see ACI 504(a)--footnote).

2.3.1 Static Resistance. In-plane forces and restraints of movement of the slab in its own plane can lead to the flexural strength of an underreinforced slab being significantly above that calculated by the ordinary yield-line analysis. Increases in flexural capacity of a slab of 15 percent or more above that predicted by yield-line analysis are commonly reported (see Appendix C), and increases considerably greater than 100 percent have been observed in tests of single panels.

The flexural strength of the flat slab system being designed will be greater than that computed using yield-line methods at small deflections due to the in-plane compressive forces resulting from the soil forces acting on the peripheral walls and due to "compressive membrane" effects resulting from the geometrical restraint of the slab. At large deflections, a sizable load-carrying capacity resulting in tensile membrane action (when the entire slab acts as a membrane in tension draped

over the supports) can be mobilized if the slab does not fail previously in other modes and if the walls are able to provide sufficient anchorage for the outer slab spans. The development of adequate restraint by the wall is certainly questionable.

Uncertainties concerning the magnitude of possible strength increases in the flat slab system, uncertainties primarily resulting from the lack of beams which would be capable of acting as struts between panel points, and the desire to provide some design margin lead to a design assumption of a rather conservative increase of 15 percent above the yield-line strength in flexure due to these in-plane effects.

In-plane force effects were not neglected entirely because a very conservative assumption concerning the flexural strength would result in considerably higher slab strengths and shear forces than those expected in the shear design, thus producing a system with a strength and behavior likely to be controlled more by shear than is desired.

Results of past experiments conducted for the Office of Civil Defense (OCD) at WES (Reference 8) and other published results discussed in Reference 8 have indicated that so-called punching shear failure in the column vicinity of the slab can occur significantly below the unit shear strength of $4\sqrt{f'_c}$ specified in ACI 1707(c), especially when the ratio of column support size to effective slab depth, c/d , is large and when the flexural reinforcement ratio is low enough in the column vicinity that flexural distress is reached in this area before shear distress. Because fairly large column capitals and low reinforcement ratios were expected in the slab design, a reduced value for the unit static shear strength, v_u , was selected, namely $v_u \leq 3\sqrt{f'_c}$.

Any compressive in-plane forces developed in the slab before large deflections are reached will improve the shear resistance; likewise, shear strength will be greatly reduced if and when tensile membrane action is reached. The occurrence of shear distress at the time of tensile membrane action appears to be almost unavoidable. Therefore, the reinforcement should be detailed so that the slab and columns are effectively tied together in the column vicinity.

Round column capitals were selected for use in the slab system not

only because spiral reinforced columns were desired, but also because the shear performance was expected to be better for a round support than for a square support of equal perimeter (Reference 8). Recently reported tests (Reference 15) have shown that the unit shear strength for round supports is indeed higher than that for rectangular supports.

No evidence was found to justify any modification of the simplified expression for ultimate unit shearing stress of $2\sqrt{f'_c}$ presented in ACI 1701(c). Indeed, there is limited experimental evidence (Reference 16) that the shear strength increases with the ratio of beam or slab width to its effective depth.

2.3.2 Effects of Blast Loading. The slab thickness needed to produce a radiation PF of at least 100, the minimum allowable value, using normal-weight concrete (145 pcf) is 9.5 inches (Reference 3, Chapter 5) which is less than the slab thickness needed for structural requirements.

A radiation protection analysis conducted at WES using the procedures included in the manual for the OCD Fallout Analysis Course (Reference 17) and the final slab design produced a calculated PF of 155. The degrading effects of a shelter entranceway were not included in this analysis.

The first step in the dynamic analysis was the determination of the natural period of vibration, T , of the slab system. The methods presented in Reference 18 were used with the following assumed slab properties:

Column spacing, $L = 18$ feet

Column capital diameter, $c = 66$ inches

Average slab thickness, $t = 12$ inches

Average slab reinforcement ratio, $p = 0.0060$

The calculated prototype slab natural period, T , was 36 msec.

The resistance of a dynamically loaded and underreinforced flexural member is nearly directly proportional to the reinforcement strength. The natural period discussed above (36 msec) implies that the average strain rate in the reinforcement (within its elastic range) in the slab under design when subjected to a blast load near the slab's capacity is on the order of 0.1 in/in/sec. Various investigations have shown a

strength increase of approximately 15 percent at this strain rate (Table 7.1 of Reference 8). Consequently, the required (dynamic) resistance, q_y , computed from the dynamic analysis that follows was assumed to be 115 percent of the needed static design member resistance, q_s .

The blast overpressure acting on the slab was modeled by the three-triangle representation of a 1-Mt surface blast given in AFDM, p. 3-47, and shown in Figure 2.5. The response chart from AFDM, p. 9-15, was used to find the necessary ratio of required dynamic resistance to maximum applied load, q_y/p_m . Assuming an equivalent single-degree-of-freedom system with $T = 36$ msec, an elastoplastic resistance, and a ductility ratio of 7, the needed dynamic resistance was computed using the following expression:

$$q_y = \frac{P_1}{(p_m/q_y)_1} + \frac{P_2}{(p_m/q_y)_2} + \frac{P_3}{(p_m/q_y)_3} \quad (2.2)$$

Where: P_1 = overpressure of triangle 1 (see Figure 2.5).
 $(p_m/q_y)_1$ = load factor for 1th overpressure triangle (from AFDM, p. 9-15) considering the duration, t_1 , of the loading.

The required dynamic resistance, q_y , needed for the slab system subjected to a maximum dynamic loading, p_m , of 15 psi is therefore:

$$q_y = \left(\frac{1.06}{1.07} + \frac{10.50}{0.955} + \frac{3.44}{0.938} \right) \text{ psi} = 15.65 \text{ psi} = 1.043p_m \quad (2.3)$$

This analysis neglects the finite time needed for the shock front to travel across the slab, which would result in a finite rise time of the force acting on the system and, consequently, a lower needed resistance. This effect did not exist in the dynamic laboratory test because the loading device to be used produces a vertically traveling wave which ideally strikes all of the slab surface simultaneously.

The design ductility ratio, μ , of 7 corresponds to a maximum deflection of approximately 1/20 the column spacing and is considerably less than the often-used expression for ductility ratio given in AFDM, p. 7-8, $\mu = \frac{10}{p - p'} \leq 30$ for beams and one-way slabs, where p and p'

are tensile and compressive reinforcement percentages, respectively.

The required static design flexural strength becomes:

$$q_s = 15 \text{ psi} (1.043) \left(\frac{1}{1.15} \right) + 1 \text{ psi} = 14.60 \text{ psi} \quad (2.4)$$

— static overpressure, p_{so}
 — ratio of required dynamic resistance to maximum pressure
 — effects of strain-rate-caused strength increase
 — dead load of slab system

The natural period, T , of the area within the lines of contraflexure (lines of zero principal moment) around the column was determined to be nearly 6 msec using medium-thick plate equations for the characteristics of a centrally supported slab.

The column load will have a nonzero rise time due to the time needed for the slab to respond. The elastoplastic system representing the slab system driven to seven times the yield deflection by a sudden applied load will reach yield deflection at approximately $0.25 T$, where T is the period of the slab system. Thus, a minimum rise time of 9 msec would be expected for the load acting on the column. In the prototype, additional rise time would exist due to the crossing time of the wave front. The 15-psi blast front moves at nearly 1,500 ft/sec and would cross two spans (36 feet) of the structure in 25 msec. Thus, a rise time, t_r , of nearly 30 msec, or six times the natural frequency of the slab area inside the contraflexure boundary, would be expected with a horizontally traveling blast wave.

Using the conservative assumption of a ductility ratio of 1 for the slab in the column area and a rise time of twice or more the natural period of this area (a ratio of $t_r/T = 2$ was used rather than 6 to more closely represent the laboratory conditions of 0 crossing time and a short but finite rise time of several milliseconds), a dynamic load factor, DLF, of 1.13 or less is obtained (Figure 2.6 and p. 48 of Reference 19).

The shear strength of slab-column connections loaded with a load rise time of 10 to 30 msec has been found to be approximately 25 percent above that of similar connections loaded very slowly, an increase largely explained by the strain rate effect on the concrete (Reference 8).

The equivalent static load to be resisted by shear thus becomes:

$$15 \text{ psi} (1.13) \left(\approx \frac{1}{1.25} \right) + 1.0 \text{ psi} = 14.55 \text{ psi} = q_{sv} \quad (2.5)$$

Based on the above analysis, the slab was designed for a static downward load of 14.6 psi.

The following three phenomena can act to cause upward forces acting on the slab:

1. Rebound of the slab. Based on a t_{50}/T value of 22.5 (810 msec/36 msec) and a ductility ratio, μ , of 7, the coefficient of elastic rebound is nearly 15 percent, or 2-1/4 psi (AFDM, p. B-32).
2. Negative overpressure phase. A maximum suction of approximately $0.15 P_{so}$ acts at about twice the positive duration time, t_D , for a 1-Mt burst at the 15-psi overpressure level (Reference 20). Thus, a negative pressure of approximately 2-1/4 psi at 3 seconds elapsed time was considered in the design loadings.
3. Pressure ingress into shelter area. Positive pressure which enters the shelter area, if not dissipated at least as rapidly as that above the slab, will be equivalent to an upward load when the overpressure level drops.

To provide resistance to these forces, a minimum design upward load of 5 psi, or one-third the downward loading, was considered adequate. This requires placement of steel in both faces of the slab at all

locations. The minimum reinforcement provisions of ACI 911 were expected to often control this loading.

The inclusion of some steel in both faces was also expected to provide better behavior of the slab at large deflections, especially in the column areas. It was thought that the added steel would serve as compression reinforcement for the downward loads and, more importantly, would allow the reinforcement mats to act more as a continuous suspension net. The suspension net behavior is discussed more thoroughly in Section 2.3.5.

2.3.3 Determination of Design Flexural Moments. The moments and flexural reinforcement were apportioned throughout the slab using the distributions specified in Chapter 21 of the 1963 ACI code. This method divides the slab into column and middle strips, each one-half of the column spacing in width, and specifies coefficients for the positive and negative moments at the midspan and transverse column lines, respectively, in each strip.

The first step in the flexural design was to prepare a moment distribution for the slab system using the ultimate strength provisions of the 1963 ACI code for an arbitrary loading of 10 psi. The yield-line capacity of the slab was then investigated.

Figure 2.7a summarizes the moment coefficients used. Assuming a 12-inch-thick exterior wall and a 66-inch-diameter column capital, the value for M_o , the total panel moment, using ACI 2101(e)1 is:

$$M_o \text{ (exterior span)} = 7,798 \text{ in-kips}$$

$$M_o \text{ (interior span)} = 6,390 \text{ in-kips}$$

The moments at each design section for the 10-psi load are shown in Figure 2.7b. The unbalanced moments at the first interior support resulting from the two M_o values were distributed in proportion to the uncracked column and slab stiffnesses. The resulting calculated moment on the column was 433 in-kips along each column line direction.

The yield-line patterns shown in Figure 2.8 were considered in the investigation of the flexural strength of the 10-psi design. Yield lines were assumed to be located at the centroid of one-half of the area of the 66-inch support (21.0 inches from the column centerline).

A slab collapse pattern including all but the center panel (Figure 2.8a) gave the lowest yield-line strength, 9.50 psi, with a distance of 93.2 inches from the negative yield line at the column location described above to the positive yield line near the center of the outer panel. The slab collapse mechanism including three panels along the structure centerline (Figure 2.8b) gave a predicted strength of 10.05 psi. A panel mechanism including only the center panel (Figure 2.8c) produced a much larger value of 27.50-psi applied load necessary to form a mechanism. The equations for the analyses above were generated using the principles of virtual work. At some locations, the compression reinforcement, assumed to be one-half of the tensile steel ratio, functioned as tensile reinforcement. This had little effect on the computed strengths. All sections were considered to be singly reinforced.

As mentioned in Section 2.3.1, the conventional Johanson yield-line analysis neglects any benefits of in-plane forces, and, for this design, the flexural capacity will be assigned a value of 115 percent of the yield-line strength. On this basis, the flexural strength of the 10-psi design is 10.93 psi (9.50 psi times 1.15). These design moments, shown in Figure 2.7b, were then multiplied by the ratio of necessary flexural static strength to that of the trial 10-psi ACI Chapter 21 design, or by $14.60/10.93 = 1.335$, to obtain the design static moments for the slab system.

2.3.4 Design of the Slab System for Shear. Using the controlling yield-line pattern (Figure 2.8a) and the principles of statics, the proportions of the vertical load supported both by the four interior columns and by the wall were determined. This method predicted that the column would support 50.2 percent and the walls 49.8 percent of the load placed inside the wall centerline. This method allots 13 percent more load to the columns than does assuming that each column supports load from an area L (L = column spacing) on a side.

The adequacy of the slab system to transfer punching shear to the column was checked assuming a 66-inch-diameter column capital, a 16-inch-thick drop panel, and an average effective depth of 14.50 inches (16 inches - $3/4$ -inch cover - $3/4$ -inch bar diameter).

The force V to be transferred is:

$$V = \left[(324 \text{ in})^2 (0.502) - (66 + 14.50)^2 \frac{\pi}{4} \right] 14.60 \text{ psi} = 694 \text{ kips} \quad (2.6)$$

The resulting unit shearing stress is:

$$v = V/bd = V/\pi(c + d)d \quad (\text{ACI 1705b})$$

$$\begin{aligned} \frac{694 \text{ kips}}{\pi(80.5 \text{ in})(14.5 \text{ in})} &= 189.2 \text{ psi} \\ &= 2.99 \sqrt{f'_c} \text{ for } f'_c = 4,000 \text{ psi} \end{aligned} \quad (2.7)$$

Thus, the unit shearing stress is nearly the same as the selected value of $3\sqrt{f'_c}$ (see Section 2.3.1), and the assumed slab and capital dimensions in the column vicinity are adequate.

The shear conditions around a 108-inch-square drop panel perimeter were found to be less critical than around the column capital.

The smaller effective depth, d , corresponding to the reinforcement layer closer to the middepth of the slab was used to compute the shearing stress at the critical section d inside the wall, the location specified by ACI 1701(c) for shear of beams and slab sections. The value of d for an assumed 11-inch-thick slab is $d = 11 \text{ inches} - 3/4 \text{ inch (cover)} - 1-1/8 \text{ inch (1-1/2 bar diameters)} = 9-1/8 \text{ inches}$.

The shear force at this critical section is 614 kips along a 617.75-inch length (54 feet - 12-inch wall - $2d$). The calculated unit shearing stress near the wall is:

$$\begin{aligned} v = \frac{V}{bd} &= \frac{614 \text{ kips}}{(617.8 \text{ in})(9.13 \text{ in})} = 108.9 \text{ psi} \\ &= 1.72 \sqrt{f'_c} \text{ for } f'_c = 4,000 \text{ psi} \end{aligned} \quad (2.8)$$

This is 14 percent below the $2\sqrt{f'_c}$ design value. Although the shear force along the wall would not be uniformly distributed, but would be larger along the column lines, the design was deemed adequate.

The method of transferring the load to the column after the occurrence of punching shear failure needed to be considered because a punching shear failure in the column capital areas was expected at large slab deflections and at or before the beginning of tensile membrane action in the slab.

Adequately anchored reinforcing bars should be able to act as a suspension net draped over the columns and supporting the transverse (vertical) loads after punching failure occurs. However, the reinforcement near the top surface of the slab will be of questionable assistance in supporting these loads since the concrete above it could easily be torn off, leaving the bars unbonded over a considerable length (indeed, over the entire bar length for most conventionally reinforced slabs, see Figure 2.9a). The length of the negative moment (top) bars carried into adjacent positive moment areas to provide compression reinforcement in this design should be of some help, but not as much as the bottom bars extended over the column capital to provide compression reinforcement in these areas. These bottom bars would not be easily torn from the slab, but would act as shown in Figure 2.9b. Bent up bars would also be efficient in holding the slab together and transferring load to the column.

Sufficient bottom bars crossing the column area are provided in the slab design (see Section 2.3.5 for reinforcement details) with a compressive reinforcement ratio equal to one-half the positive steel to support a 21-psi load with the bars assumed to cross the shear crack at an angle of 45 degrees to the horizontal.

2.3.5 Detailing of the Reinforcement. After the slab depth had been determined from the shear strength investigation, the flexural reinforcement could be proportioned.

One bar size and only straight bars were used as flexural reinforcement, partially to minimize the problems of modeling the slab.

The flexural reinforcement was proportioned using Equation 16-1 of the ACI code with $\phi = 1.00$ and for Grade 40 reinforcement.

The design moments (from Section 2.3.3) and the design tensile reinforcement percentages are shown in Figure 2.10. For design, the effective depth of the layer closer to the slab middepth was used. The minimum reinforcement provisions of the ACI code (Section 911a) controlled reinforcement percentage in several areas including all center span positive moment sections.

All bottom steel in each direction including that within the drop panel area was located at the same level.

The use of No. 6 bars resulted in reasonable bar spacings. The prototype minimum bar spacing is 3-1/2 inches over the column; the maximum spacing is 18 inches. The provisions of ACI 804c controlled some spacings of the compression steel.

The reinforcement layouts for one-fourth of the prototype in one direction are shown in Figures 2.11 and 2.12, for the top and bottom mats, respectively. Compression steel was obtained by extending bars from the positive moment areas.

The design minimum bar lengths used (shown in Figure 2.13) were based largely on the provisions of ACI Figure 2104(g)a. Some design bar lengths were extended beyond the minimums given by ACI due to (1) the unusual column capital size, which moved the line of contraflexure further from the column centerline; (2) the large drop panel size, which caused the bottom steel cutoff points (when measured from the drop panel edge) to be further from the column line than desired; (3) the much better than normal slab edge fixity offered by the wall, which should move the contraflexure line near the wall slightly inward, and (4) the need to extend some bottom bars near the column to obtain distance desired for lap splices in bars participating in the postshear distress load transfer to the column. Some lengths would have been extended further had not the ACI recommendations included effects of pattern or partial loads which are not present in blast loading but which act to increase bar lengths over those needed for uniform loadings only.

The bottom bars spliced over the outside half of the column capital and expected to help hold the slab together after shear distress are lapped 64 or 100 bar diameters (48 or 72 inches), appreciably more than

the 24 bar diameters (18 inches) required by ACI 805b. These high values are used because (1) many of the bars are not placed in contact with the other bar in the lap, (2) a shear crack forming at the middle of the lap and yielding all bars could effectively half the splice length, and (3) deformed bars could not be obtained in the size needed for the model.

Design of the wall reinforcement was not detailed for the prototype slab system design.

2.3.6 Column Design. The calculated static column load (from Section 2.3.4) is 769 kips, and the column moment based on distribution of unequal slab moments was found to be 630 in-kips in each span direction. The equivalent moment of 891 in-kips along the diagonal yields an eccentricity, e , of 1.42 inches.

Because the capacity of the columns cannot benefit from in-plane compressive and tensile membrane effects of the slab (although these effects may increase the slab strength and therefore the load on the column significantly), the columns were designed to carry a load in excess of that computed above. Also to be considered in selecting a column design load are the serious consequences (collapse of the floor system) accompanying a total column failure and the relatively small incremental costs of increasing the column capacity. Based on these considerations, a column load of 1-3/4 times the computed load from the design static slab loading, or 1,350 kips, was selected.

The ACI code provisions for minimum column moment of inertia (Equation 21.1) and for column bending moment (Section 2104(b)2) consider pattern loading, or unequal placement of the live loads. These provisions should not be applied for the case of a nearly uniform blast pressure, but rather for a live load equal to the design live load for the everyday use of the slab, a value assumed to be 100 psf. (This loading and the provisions above did not govern the column design.)

Assuming a column diameter of 22 inches, the column reinforcement area needed to carry 1,350 kips at an eccentricity, e , of 1.16 inches (and using $\phi = 1.00$) was found to be 10.4 in^2 (Reference 21), an area requirement satisfied by the use of eight No. 11 bars. The column base

was assumed pinned for computing length effects.

A No. 3 spiral spaced at 2-1/4 inches is necessary in the columns to satisfy the ACI code requirements.

2.4 DESIGN OF THE SLAB SYSTEM BY CONTEMPORARY ACI BUILDING CODES

One possible method of designing slanted shelter areas which would minimize the additional training needed by the designer, particularly in the area of structural dynamics and nuclear weapon blast effects, would be to specify equivalent static design loads and guidelines on reinforcement placement, especially at connections and other important so-called "detail areas" to insure that the slab will possess the desired strength and ductility. This approach is practical because the ratio of peak airblast overpressure to needed static resistance for similar slab types (the flat slab in the case of this project) also designed for a blast loading of nearly 15 psi would be quite insensitive to the span length for a considerable range of spans, perhaps from 8 to 30 feet. The necessary changes from the ACI design provisions would also be fairly constant for any given slab type.

With this possible design method in mind, the slab system design discussed in the previous section was examined to determine the loads and changes needed to allow direct design of the slab system from the 1963 or 1971 ACI building code requirements (References 1 and 2). Numerical values were adjusted so that ϕ (capacity reduction factor) in the ultimate strength design procedure would be retained and would have its usual value.

2.4.1 Design by the 1963 ACI Building Code. The slab system design discussed in Section 2.3 can be reproduced using the ultimate strength design procedure for a flat slab contained in the 1963 ACI code with the following loads and modifications:

1. The slab is designed for a total design load, U , of 1,580 psf + 1.0 times dead load.
2. An average unit shearing stress along the wall (beam shear), based on the load contributed from areas within one-half the span length of the wall, of $1.93 \phi \sqrt{f'_c}$ is allowed.

3. An average unit shearing stress of $2.54 \phi \sqrt{f'_c}$ is allowed for the column areas assuming that each column supports an area one span length on a side.

4. Design moments from upward loads of one-half the moments resulting from the downward loadings are specified at all locations, or, alternately stated, a compression reinforcement ratio equal to one-half the tensile reinforcement ratio is used throughout.

5. Several bar lengths are extended from those given in Figure 2104(g)a of the 1963 ACI code to the lengths shown in Figure 2.13 of this report.

6. Equations 21-4 and 21-5 of the 1963 ACI code are not used to establish a minimum slab thickness.

7. The columns are designed for an axial load of 148 percent of the computed load from an area one span length on a side ($\phi = 0.70$ or 0.75). The use of round, spirally reinforced columns is encouraged.

8. The requirements of ACI Equation 21-1 and Section 2104(b)2 relating to minimum column moment of inertia and column moments resulting from pattern loadings are applied using the daily-use live load rather than the blast loading as the live load value.

The design also assumes that the walls are monolithic with the slab and sufficiently reinforced to allow the slab to reach yielding around its perimeter.

For designing other similar slab systems to carry the same over-pressure by this modified ACI code procedure, the loads and stresses in items 1, 2, 3, and 7 above could be rounded to a load of 1,600 psf plus dead load, a unit shearing stress near the slab perimeter of $2.0 \phi \sqrt{f'_c}$, a unit shearing stress around the column of $2.6 \phi \sqrt{f'_c}$, and a column design load of 1.5 times the load on an area one span length on a side.

2.4.2 Design by the 1971 ACI Building Code. The 1971 ACI building code requirements for reinforced concrete (Reference 2) contain unified provisions for the flexural design of all but one-way slab systems. The two-way and flat slab-flat plate structural systems are both designed by a procedure similar to that used for the flat slab system in the 1963 ACI building code. The use of a considerably larger factor of safety

for the two-way system has been dropped.

The effect of the 1971 code on the slab design was studied by first determining the flexural moment distribution resulting from the 1971 code with the previously chosen slab dimensions and for a total design load, U , of 1,730 psf (1,580 psf + dead load). This is the design load needed to reproduce the slab design using the 1963 ACI code, see Section 2.4.1. The yield-line strength of the resulting slab design could then be computed and compared with that of the slab used. The ϕ (capacity reduction) factors specified by the 1971 ACI code were used.

The expected moment capacities of the various design sections resulting from the use of the 1971 code are shown in Figure 2.14. The moments resulting from the direct design method presented in Chapter 13 of the 1971 code have been multiplied by 1.111 (equal to $1/\phi$ where $\phi = 0.90$) to account for the expected moment capacities being higher than computed moments because of the understrength factor which would be used in determining the flexural reinforcement areas.

The differences in the moment distributions resulting from the use of the 1963 and 1971 ACI codes for the same design loads and slab geometry can be studied by comparing the moments given in Figure 2.10a (1963 code) with those given in Figure 2.14 (1971 code). The 1971 code provisions increase the total design moment, M_o , slightly, but appreciably decrease the reinforcement required in the edge strip parallel to a very stiff beam or wall. The moments in the middle strip adjacent to this edge strip are also decreased somewhat. The negative moments supported by the structural wall are more evenly distributed along the wall than they are when the 1963 code provisions are used.

Flexural strength provided by the moments shown in Figure 2.14, based on a conventional yield-line analysis, is 12.44 psi. This is 98.3 percent of the computed yield-line strength of 12.67 psi for the design modeled in this study. The exterior slab yield-line pattern (Figure 2.8a) governed the yield-line analysis. The 1971 code design would compare more favorably for slab systems with more panels since the lower moments provided in areas near and parallel to the walls would have less influence. Although the 1971 code results in different

distributions of moment, the expected slab strength is nearly identical with that provided by the 1963 code.

Applicable provisions for beam and slab shear design have remained unchanged from the 1963 code. The slab design meets the minimum thickness requirements of Section 9.5.3 of the 1971 code.

Reinforcement lengths have been changed slightly in the 1971 code. However, the lengths are still less than those given in Figure 2.13. These later lengths are still preferable for use with the large column capital and drop panel sizes of the slab design used.

The impossibility of the blast loading to act as a pattern loading should be recognized when examining the restrictions for the 1971 code direct design procedure and the requirements for column and wall moments and the effects of pattern loadings presented in Section 13.3 of the 1971 code.

2.5 MODELING OF THE PROTOTYPE

2.5.1 The Model Scale. A model scale of one-fourth was chosen for the experimental study. This scale resulted in structures which were reasonable representations of the prototype structure. The resulting structures were large enough to allow the use of a small-aggregate concrete instead of mortar. The 13-3/4-foot square, 1/4-scale structures were nearly the maximum size that could be placed in the 22-foot 10-inch-diameter LBLG facility used for both static and dynamic loading, particularly since space was desired around the structure to include enough soil around the walls of the model to allow a realistic soil-structure interaction to develop.

2.5.2 Relationship of Quantities in the Models and in the Prototype. The load capacities of the models were nearly the same as that to be expected for the prototype. A small increase in applied load capacity of about 0.75 psi was expected in the models because the structure dead load was reduced by 75 percent, or nearly 0.75 psi. By the laws of similitude, stresses and strains scale directly between the models and prototype. The deflections and time scales of the models are one-fourth of those in the prototype with the Mach modeling used. Predicted

velocities and accelerations of the prototype are one time and one-fourth times, respectively, the corresponding values observed in the models.

2.6 DESIGN AND DESCRIPTION OF THE MODEL

2.6.1 Description and Dimensions of the Models. The 1/4-scale models of the prototype were designed to be cast as an "open-box" type unit consisting of the scaled slab, columns, and walls, which fit upon heavily reinforced base slabs. The bases of the walls were effectively fixed by bolting to the base slabs, which served as the floors and footings for the structures. The dimensions of the models were obtained by dividing the prototype dimensions by four. The column lengths were reduced and the bases pinned to allow the placement of load cells at the base of each column. Figure 2.15 shows a cross section of one-half of one model.

2.6.2 Reinforcement. The No. 6 (0.75-inch-diameter) Grade 40 reinforcement of the prototype was modeled by using 3/16-inch-diameter annealed C1018 steel rod that was soaked in muriatic acid to etch and rust the bar for improved bond characteristics. The preferred deformed or indented bar geometry could not be obtained in the desired bar size and material strength.

The spacings of the 3/16-inch-diameter reinforcement in the models were one-fourth of those of the prototype multiplied by the factor $f_y/44$ ksi, where f_y = actual yield strength of the model reinforcement. This factor was included to insure a reasonable modeling of the prototype flexural strength even if the model reinforcement might have a yield stress level different from the average yield value (as opposed to the minimum yield value of 40 ksi) of 44 ksi assumed for the Grade 40 reinforcement. (From AFDM, p. 6-5, median yield of reinforcing bar with a specified bar with a specified minimum yield of 40 ksi is approximately 45 ksi.)

The etched 3/16-inch-diameter bar used had an average yield of 36.9 ksi (see Section 3.1). The spacings used in the models were 5/6 times the one-fourth of the prototype spacings ($36.9/44 = 0.839 \approx 5/6$). These model spacings are shown in Figures 2.16 and 2.17. Details of the

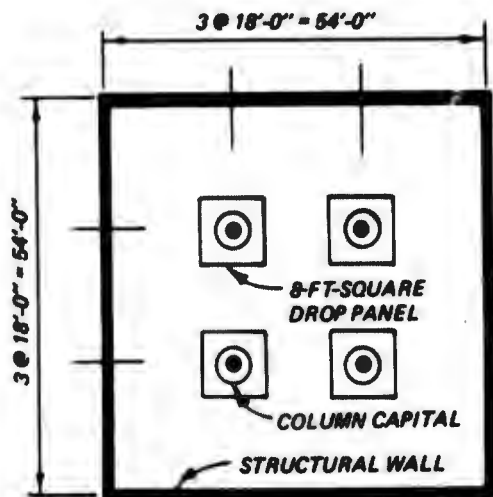
slab reinforcement placement are shown in Figures 2.18 and 2.19.

2.6.3 Column Design. The columns of the models were scaled down from the prototype columns, which were designed to carry 175 percent of the static design slab load. The reinforcement for the model columns consisted of six No. 3 Grade 40 vertical bars placed within a spiral fabricated using No. 12 wire (0.1055-inch diameter) and having a pitch of 1/2 inch. Details of a model column area are shown in Figure 2.18.

2.6.4 Wall Design. The walls were included in the models primarily to offer the fixity of the slab perimeter desired and assumed in the design and to provide the expected slab in-plane loading resulting from the horizontal soil pressures acting on the wall. The walls were designed using conservative assumptions to preclude any primary failure of the wall. The design loads assumed were both 0- and 15-psi uniform horizontal soil loads. The wall bottom was either pinned or fixed, and it was assumed that the yield moment of the slab was acting at the top of the wall.

Welded wire mat was used as reinforcement for the walls, partially to facilitate the fabrication of the wall reinforcement. A welded wire fabric (No. 4 wires (0.225-inch diameter) at 4-inch spacing in each direction) with a specified minimum yield strength of 70 ksi was placed in each face of the wall. The hooks of the top layer of the slab steel were extended 12 inches down into the wall to provide the reinforcement needed at the top outside face of the wall so that any yielding of the flexural reinforcement would occur in the slab, not at the top of the wall. Details of the reinforcement in the wall are shown in Figure 2.19.

To provide access into the model structures from the side, an entrance was provided at the center of one wall of each model. These 24- by 25-inch openings were framed with 6-inch-deep by 8-inch-wide beams reinforced with a No. 4 Grade 40 bar placed at each corner and containing closed 3/16-inch-diameter stirrups to resist torsion, particularly along the tops of the openings. Details of this type access opening are shown in Figure 2.20.

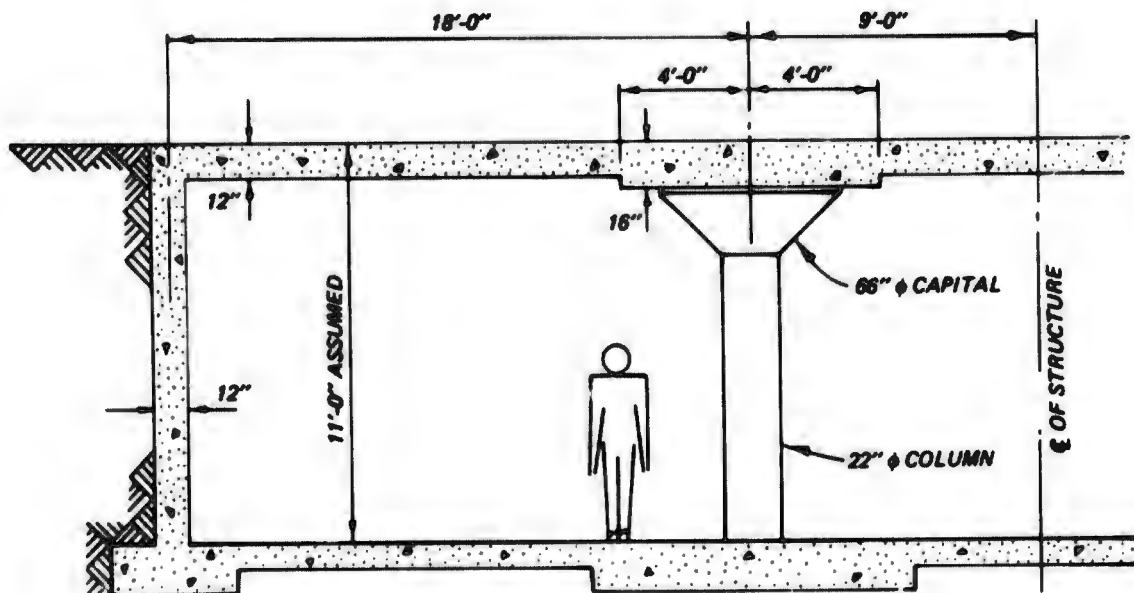


BOTTOM VIEW OF SLAB

REINFORCEMENT PERCENTAGES

IN SLAB FOR DOWNWARD LOADS:
 $0.44\% \leq \rho \leq 1.11\%$
 OVER COLUMNS, $\rho = 1.03\%$
 IN WALLS, VERTICAL DIRECTION:
 $0.41\% \leq \rho \leq 1.31\%$
 NO SHEAR REINFORCEMENT IN THE SLAB.

COMPONENT	QUANTITY	
	CONCRETE	STEEL
SLAB, COLUMNS	120 YD ³	13.1 TONS
WALLS	79 YD ³	6.4 TONS
STRUCTURE ABOVE FLOOR	199 YD ³	19.5 TONS



HALF-SECTION VIEW

Figure 2.1 Preliminary design using a flat slab system with capitals.

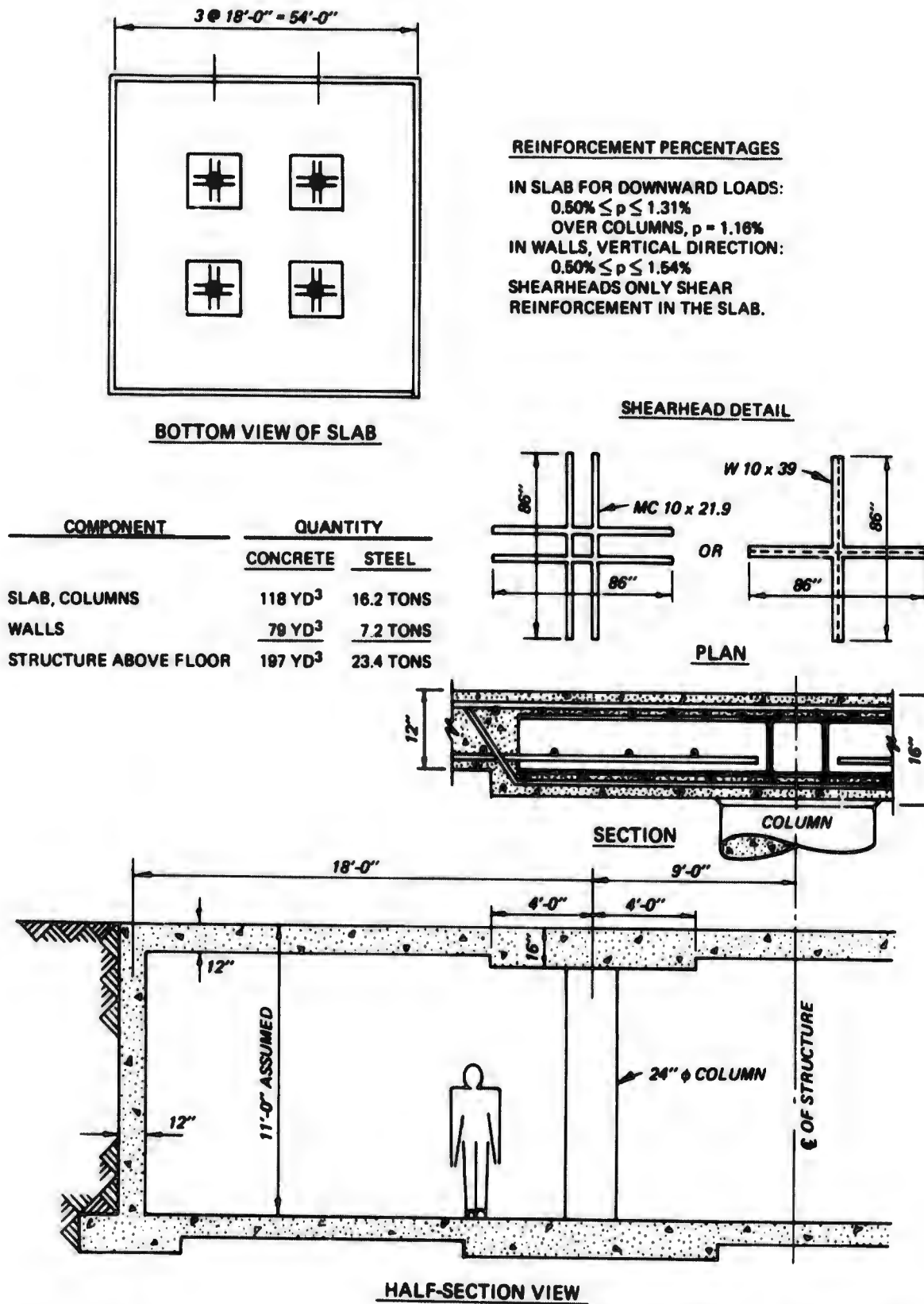
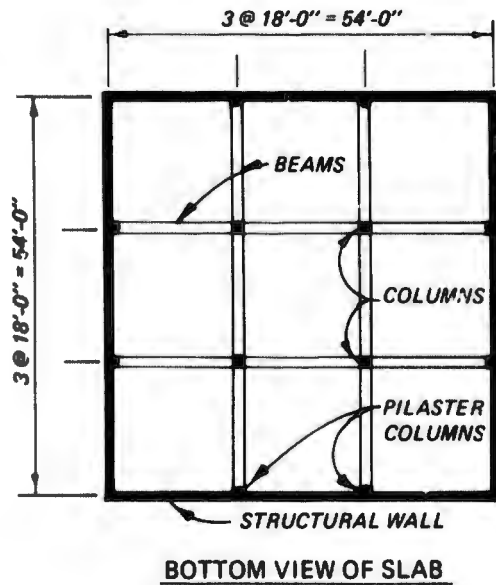


Figure 2.2 Preliminary design using a flat slab system with shearheads.



REINFORCEMENT PERCENTAGES

IN SLAB FOR DOWNWARD LOADS:

$$0.43\% \leq \rho \leq 0.84\%$$

NO SHEAR REINFORCEMENT IN THE SLAB.

IN BEAMS FOR DOWNWARD LOADS:

$$5.2 \text{ IN.}^2 \leq A_s \leq 10.8 \text{ IN.}^2 (\rho = 2.79\%)$$

STIRRUPS THROUGHOUT BEAMS FOR SHEAR

$$0.37\% \leq \rho_v \leq 1.15\%$$

IN WALLS, VERTICAL DIRECTION:

$$0.32\% \leq \rho \leq 0.62\%$$

COMPONENT	QUANTITY	
	CONCRETE	STEEL
SLAB, BEAMS, INTERIOR COLUMNS	109 YD ³	13.5 TONS
WALLS, PILASTER COLUMNS	85 YD ³	7.3 TONS
STRUCTURE ABOVE FLOOR	194 YD ³	20.8 TONS

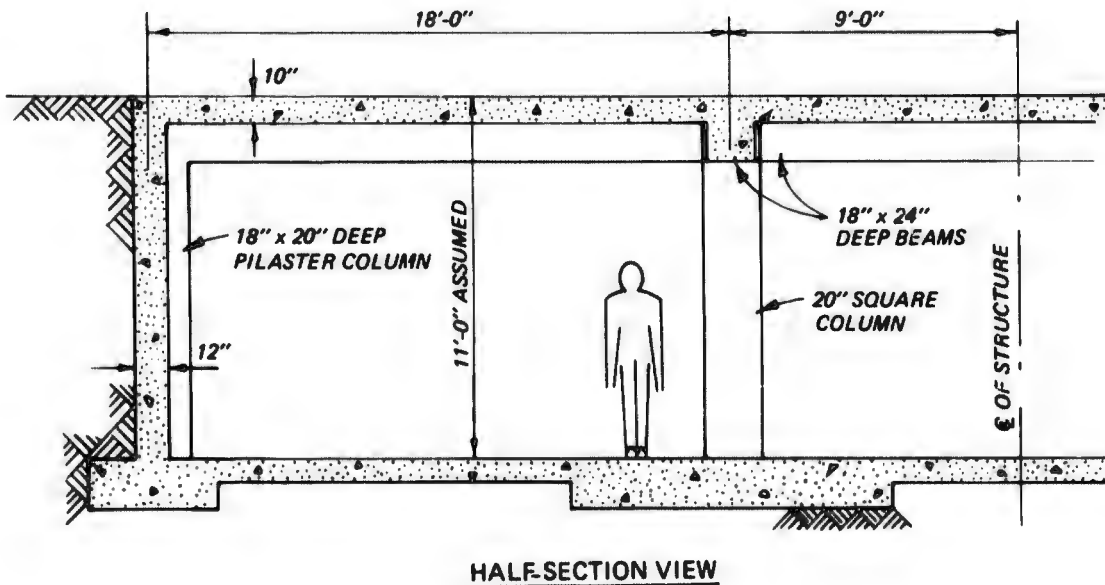
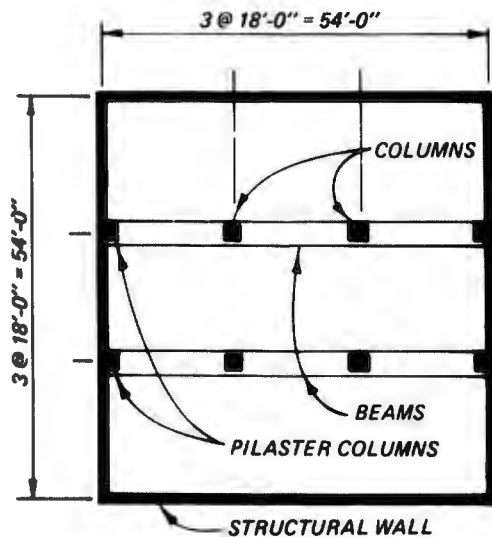


Figure 2.3 Preliminary design using a two-way slab system.



BOTTOM VIEW OF SLAB

REINFORCEMENT PERCENTAGES

IN SLAB FOR DOWNWARD LOADS:
 $0.50\% \leq \rho \leq 0.78\%$ IN SPAN
 DIRECTION, TEMPERATURE
 STEEL OF 0.20% GROSS AREA
 IN OTHER DIRECTION.

NO SHEAR REINFORCEMENT IN THE SLAB.

IN BEAMS FOR DOWNWARD LOADS:

$$7.1 \text{ IN.}^2 \leq A_s \leq 12.1 \text{ IN.}^2 \quad (\rho = 2.0\%)$$

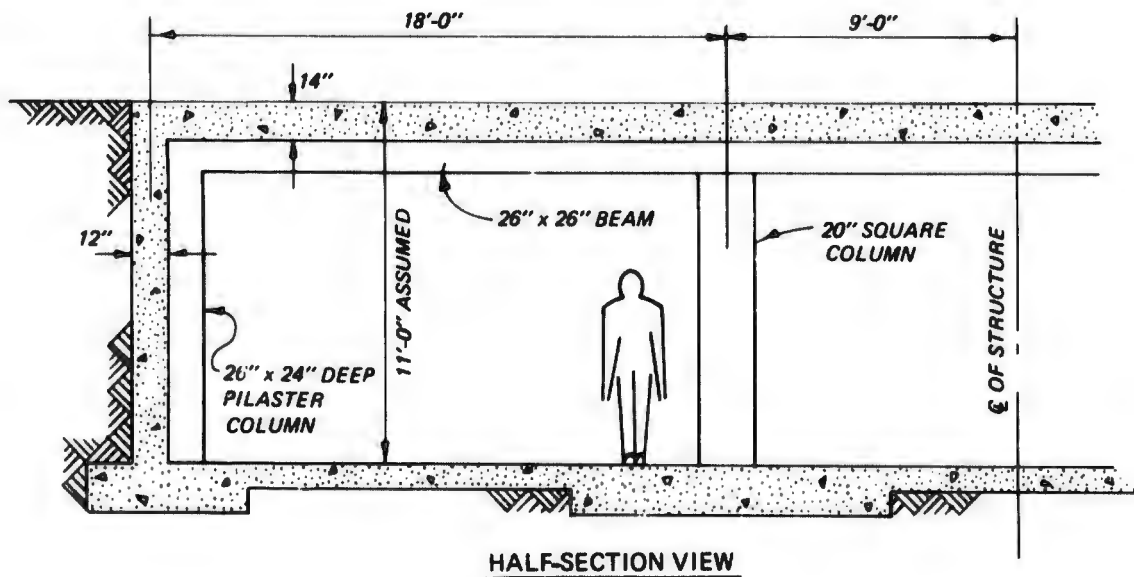
STIRRUPS THROUGHOUT BEAMS FOR SHEAR

$$0.31\% \leq \rho_v \leq 0.88\%$$

IN WALLS, VERTICAL DIRECTION:

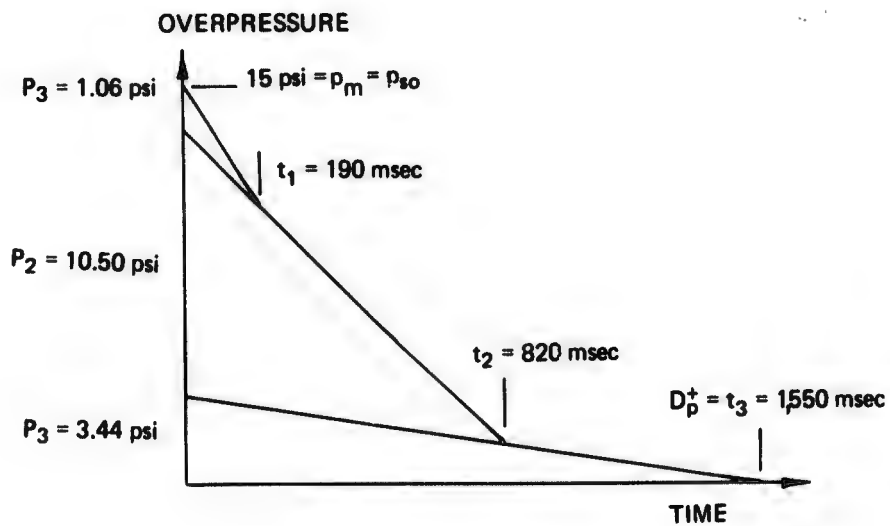
$$0.32\% \leq \rho \leq 0.88\%$$

COMPONENT	QUANTITY	
	CONCRETE	STEEL
SLAB, BEAMS, INTERIOR COLUMNS	143 YD ³	11.7 TONS
WALLS, PILASTER COLUMNS	83 YD ³	6.4 TONS
STRUCTURE ABOVE FLOOR	226 YD ³	18.1 TONS

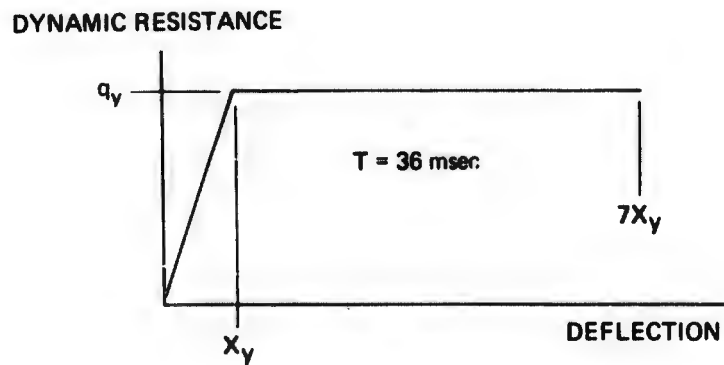


HALF-SECTION VIEW

Figure 2.4 Preliminary design using a one-way slab system.



a. PRESSURE-TIME CURVE
FOR 1-MT DEVICE



b. RESISTANCE

Figure 2.5 Loading and resistance function for dynamic analysis of slab in flexure.

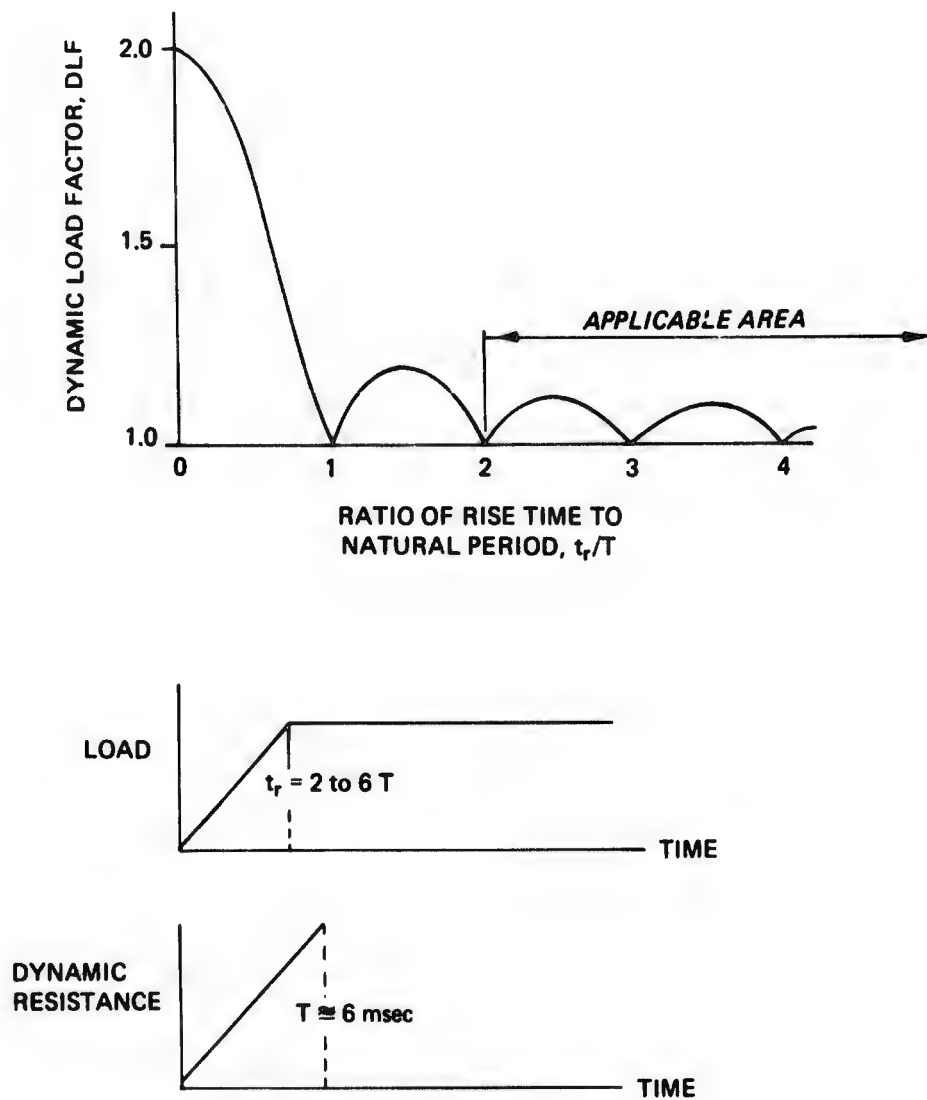


Figure 2.6 Dynamic analysis of slab in shear.

	INTERIOR CENTER OF SPAN	FIRST INTERIOR SUPPORT	CENTER OF SPAN	EXTERIOR SUPPORT	
* VALUE FOR SLAB WITHOUT DROP PANELS	+8%*	-13%*	+7%*	-8%*	HALF COLUMN (EDGE) STRIP
⊗ VALUE FOR SLAB WITH DROP PANELS	+15%†	-17%†	+20%†	-20%*	MIDDLE STRIP
† AVERAGE OF VALUES FOR SLABS WITH AND WITHOUT DROPS	-20% ⊗	-56% ⊗ 	+26%†	-32%*	COLUMN STRIP

← SPAN DIRECTION

EXTERIOR SUPPORT CONDITION: CASE B - REINFORCED CONCRETE BEARING WALLS INTEGRAL WITH SLAB.
 SIDE SUPPORT PARALLEL TO STRIP: CASE 3 - REINFORCED CONCRETE BEARING WALLS CAST INTEGRALLY WITH SLAB.

VALUES GIVEN ARE PERCENT OF TOTAL PANEL MOMENT, M_o , WHERE:

$$M_o = 0.10 WLF \left(1 - \frac{2c}{3L}\right)^2 \quad \text{FROM AC1 2101 (e) 1}$$

WHERE:

W = LOAD ON A PANEL
 L = SPAN LENGTH, COLUMN CENTER TO CENTER
 $F = 1.15 - c/L \geq 1.00$
 C = EFFECTIVE SUPPORT SIZE

a. MOMENT DISTRIBUTION COEFFICIENTS

	+384	-967	+468	-624	EDGE STRIP
	+959	-1,264	+1,560	-1,560	MIDDLE STRIP
MOMENT INCH-KIPS	+1,278	-4,163	+2,027	-2,495	COLUMN STRIP

← SPAN

b. MOMENTS FOR A 10 PSI LOAD

Figure 2.7 Distribution of flexural moments.

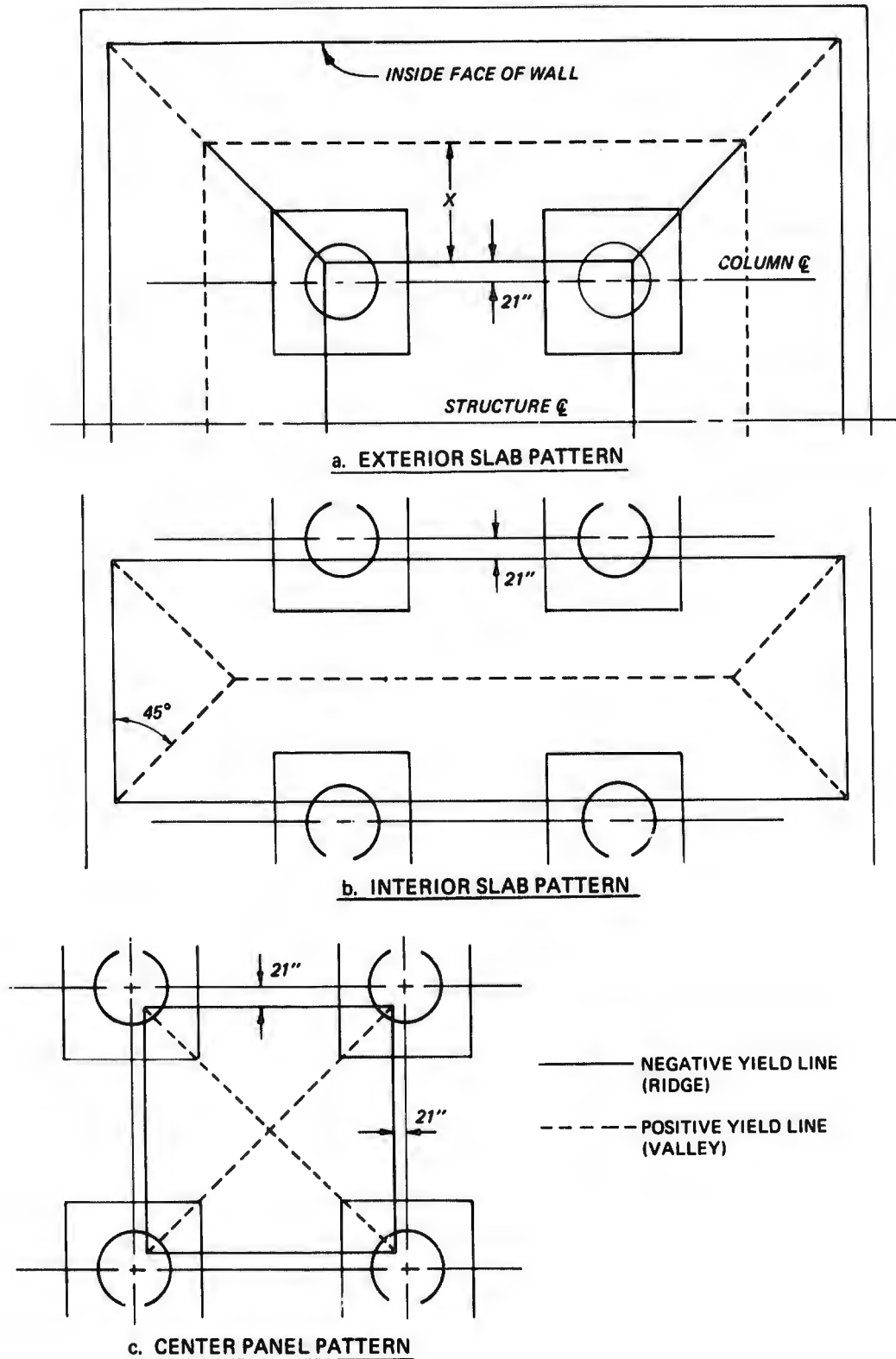
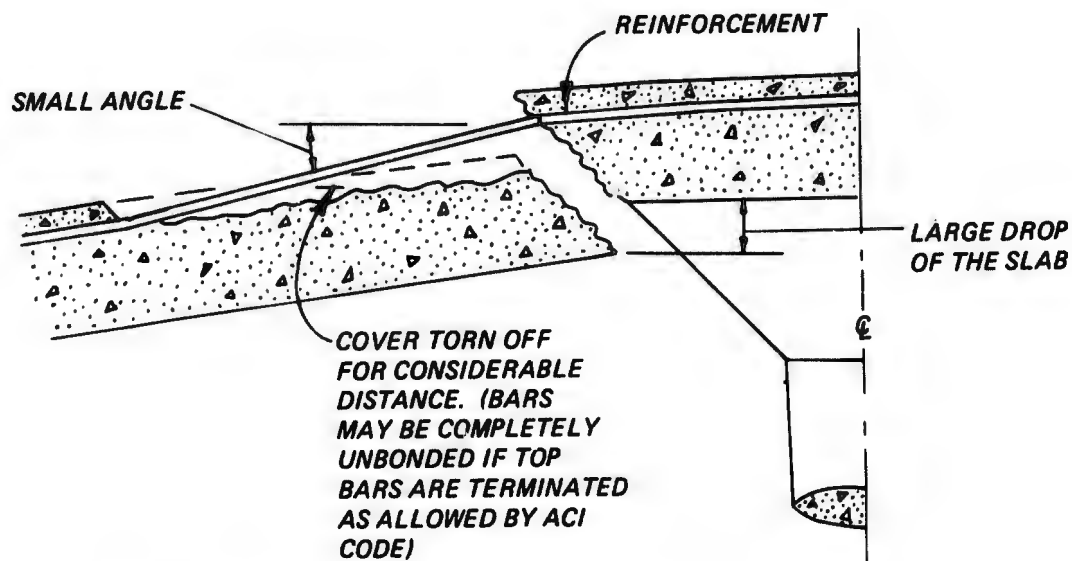
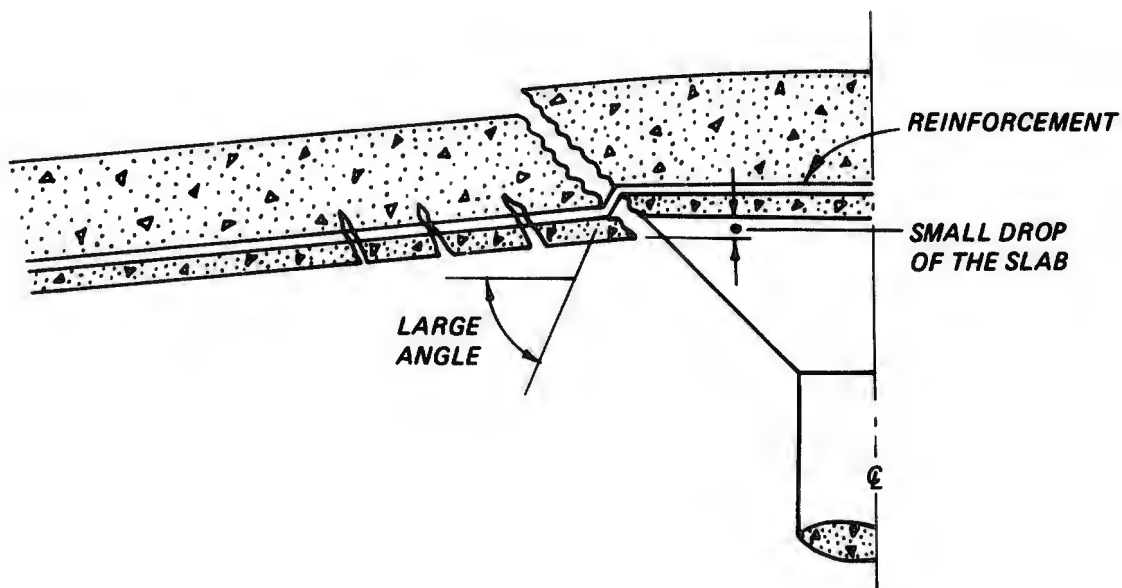


Figure 2.8 Yield-line patterns for flexural strength determination.



a. POSTFAILURE GEOMETRY; TOP REINFORCEMENT ONLY



b. POSTFAILURE GEOMETRY; BOTTOM REINFORCEMENT ONLY

Figure 2.9 Load transfer to column after punching shear distress.

SPAN DIRECTION
←→

ALL MOMENTS ARE IN INCH-KIPS

+510	-1,288	+623	-831	EDGE STRIP
+1,277	-1,684	+2,077	-2,077	MIDDLE STRIP
+1,702	-5,546	+2,701	-3,323	COLUMN STRIP
+1,277	-1,684	+2,077	-2,077	MIDDLE STRIP
 ℄ 				

a. DESIGN MOMENTS FOR PROTOTYPE

0.387 (0.290) ^a	0.761	0.482 (0.361)	0.500 (0.480)	SPAN DIRECTION ←→
0.487 (0.365)	0.500 (0.487)	0.606	0.606	
0.500 (0.492)	0.680	0.801	1.003	GRADE 40 REINFORCEMENT
 ℄ 				

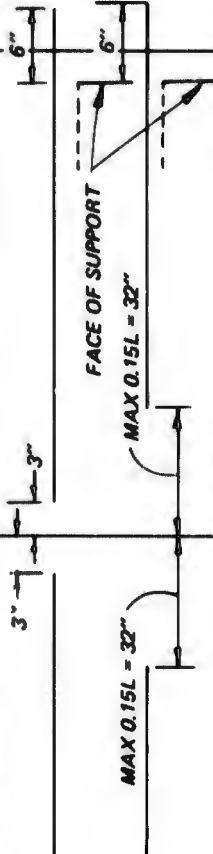
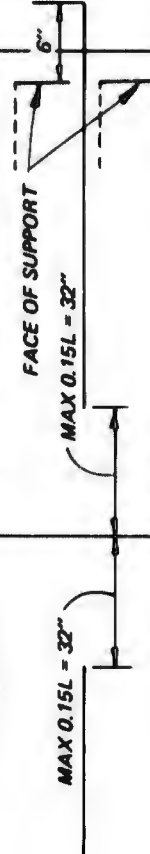
^a WHEN MINIMUM STEEL PROVISIONS CONTROLLED, CALCULATED VALUES ARE SHOWN IN PARENTHESIS

d = 9-1/8 INCH EXCEPT FOR TOP STEEL IN AREA OF DROP PANEL WHERE d = 14-1/8 INCH

b. DESIGN STEEL PERCENTAGES

Figure 2.10 Design moments and tensile reinforcement percentages.



		COLUMN & WITH DROP PANELS		WALL & NO DROP PANELS	
COLUMN STRIP, STRAIGHT BARS	TOP	50% OF REINFORCEMENT	$L/3$ $\boxed{+6''}$	$L/3$ $\boxed{+6''}$	$0.30L$ $\boxed{+6''}$
		REMAINDER	$L/4$ $\boxed{+6''}$	$L/4$ $\boxed{+6''}$	$0.25L$ $\boxed{+6''}$
	BOTTOM	100%	$16 \text{ BAR } \phi$ $\boxed{+6''}$	$16 \text{ BAR } \phi$ $\boxed{+6''}$	
		100%	$L/4$ $\boxed{+6''}$	$L/4$ $\boxed{+6''}$	$0.25L$ $\boxed{+6''}$
MIDDLE STRIP, STRAIGHT BARS, ALSO USED FOR EDGE STRIP	TOP	50%	$3''$	$3''$	
	BOTTOM	50%	$\text{MAX } 0.15L = 32''$	$\text{MAX } 0.15L = 32''$	

CHANGES

1. TOP STEEL OVER COLUMN LINE EXTENDED 6" BECAUSE OF LARGE CAPITAL SIZE.
2. BOTTOM STEEL-COLUMN STRIP EXTENDED TO RETAIN LENGTH OF BAR FOR NORMAL 7" SQUARE DROP.
3. TOP BARS AT WALLS EXTENDED 6" BECAUSE WALL WILL OFFER MUCH BETTER MOMENT RESISTANCE THAN USUALLY PROVIDED ON SLAB PERIMETER CAUSING LINE OF CONTRAFLEXURE TO MOVE TOWARD SPAN
- NOT SHOWN AND PRIMARILY TO PROVIDE LAP IN BOTTOM MAT NEAR COLUMN:
4. ALL COMPRESSION STEEL OVER COLUMN EXTENDED TO $L/3 + 6''$ FROM COLUMN
5. EVERY OTHER COLUMN STRIP EXTERIOR SPAN BOTTOM BAR EXTENDED TO WITHIN 3" OF COLUMN

CHANGES

1. TOP STEEL OVER COLUMN LINE EXTENDED 6" BECAUSE OF LARGE CAPITAL SIZE.
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4. NOT SHOWN AND PRIMARILY TO PROVIDE LAP IN BOTTOM MAT NEAR COLUMN: ALL COMPRESSION STEEL OVER COLUMN EXTENDED TO $L/3$ +6" FROM COLUMN.
5. EVERY OTHER COLUMN STRIP EXTERIOR SPAN BOTTOM BAR EXTENDED TO WITHIN 3" OF COLUMN.

CHANGES FROM 1983 ACI CODE FIGURE 2104 (g), MINIMUM LENGTH OF FLAT SLAB REINFORCEMENT, ARE SHOWN IN BOXES ABOVE.

Figure 2.13 Design minimum reinforcement lengths.

Edge Strip	173 (0.339)	-413 (0.321)	307 (0.493)	-467 (0.562)
Middle Strip	1,014 (0.785)	-1,852 (1.100)	1,772 (0.855)	-1,786 (0.861)
Column Strip	1,881 (1.104)	-5,575 (1.005)	3,280 (1.212)	-2,600 (0.782)
Middle Strip	1,252 (0.982)	-1,868 (1.108)	2,185 (1.055)	-2,600 (1.252)

Moments, kip-inches

Moments given are those resulting for a design load of 1,730 psf times 1.111 to give the expected rather than minimum moment capacity. The 1.111 value is $1/\phi$ where $\phi = 0.90$.

Numbers in parentheses are the ratios of the given moments to those resulting from the 1963 ACI Code and listed in Figure 2.10

Span Direction →

Structure

Figure 2.14 Moment capacities provided from the use of the 1971 ACI code.

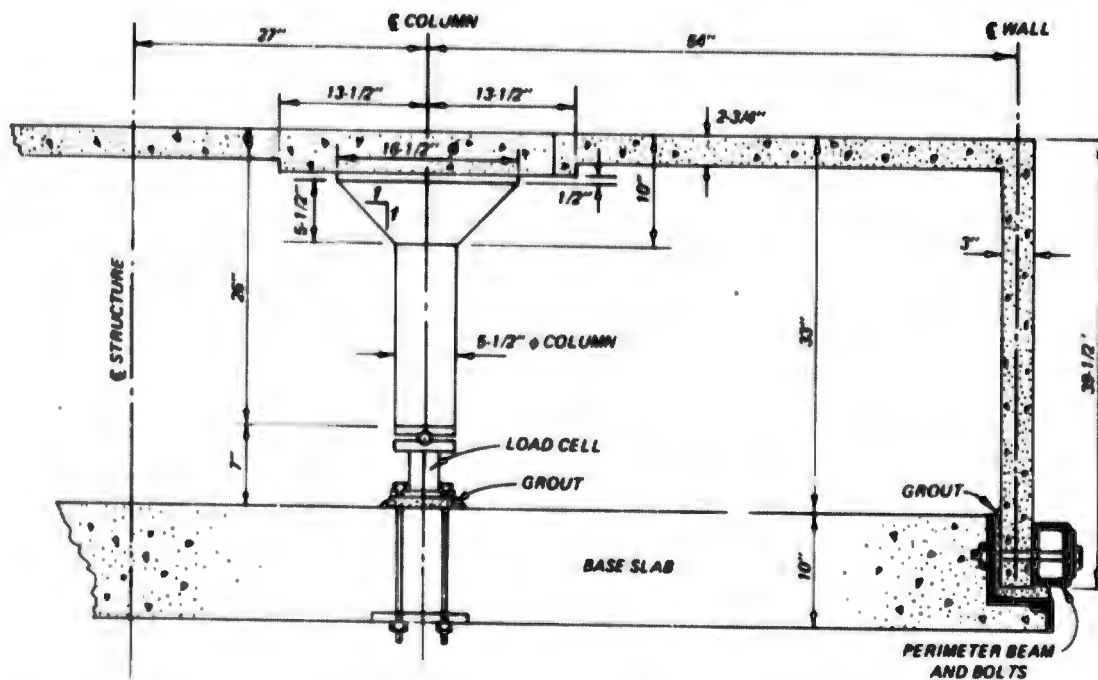


Figure 2.15 Half-section of the model.

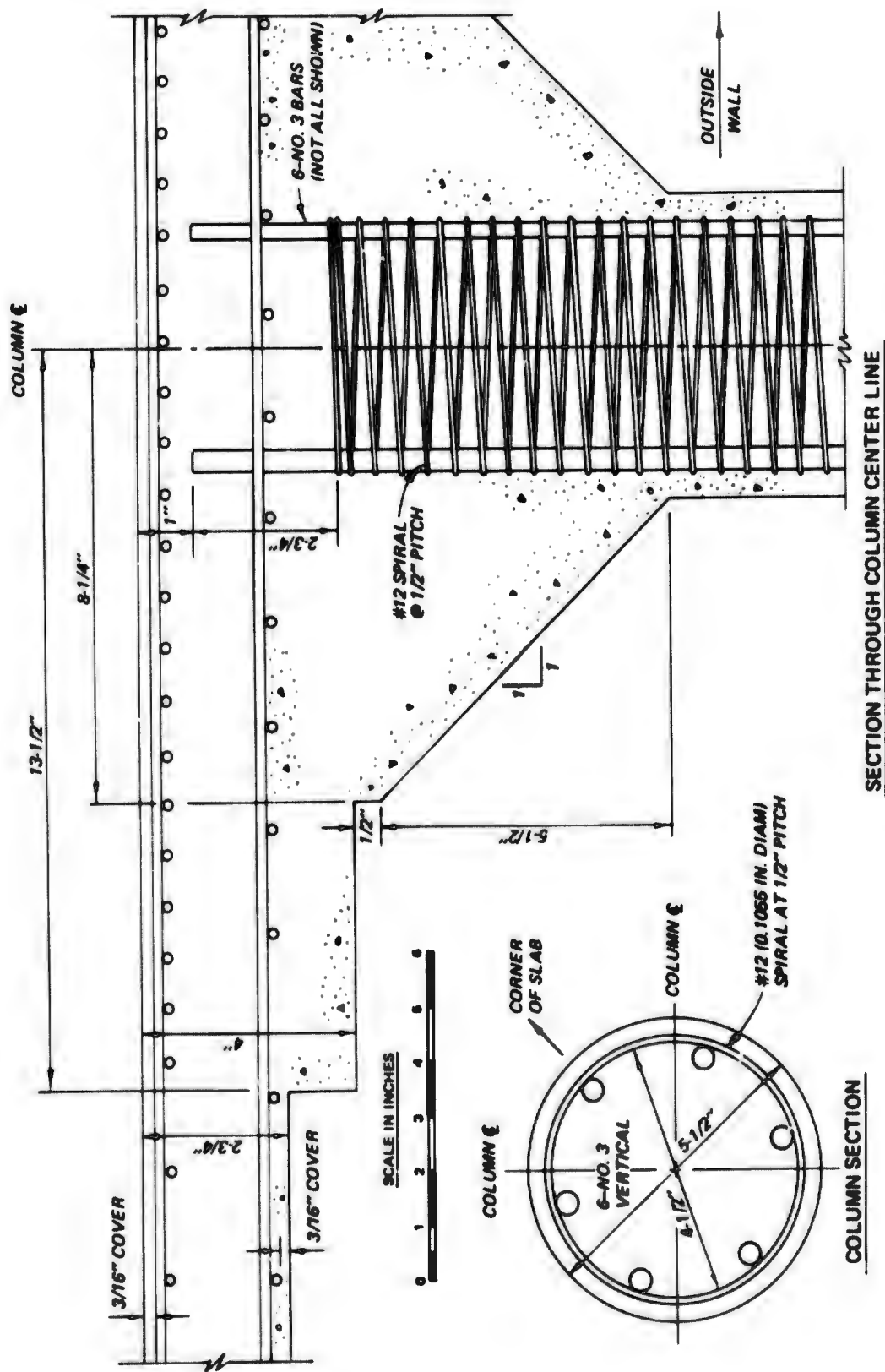


Figure 2.18 Details of reinforcement in the column area of the model.

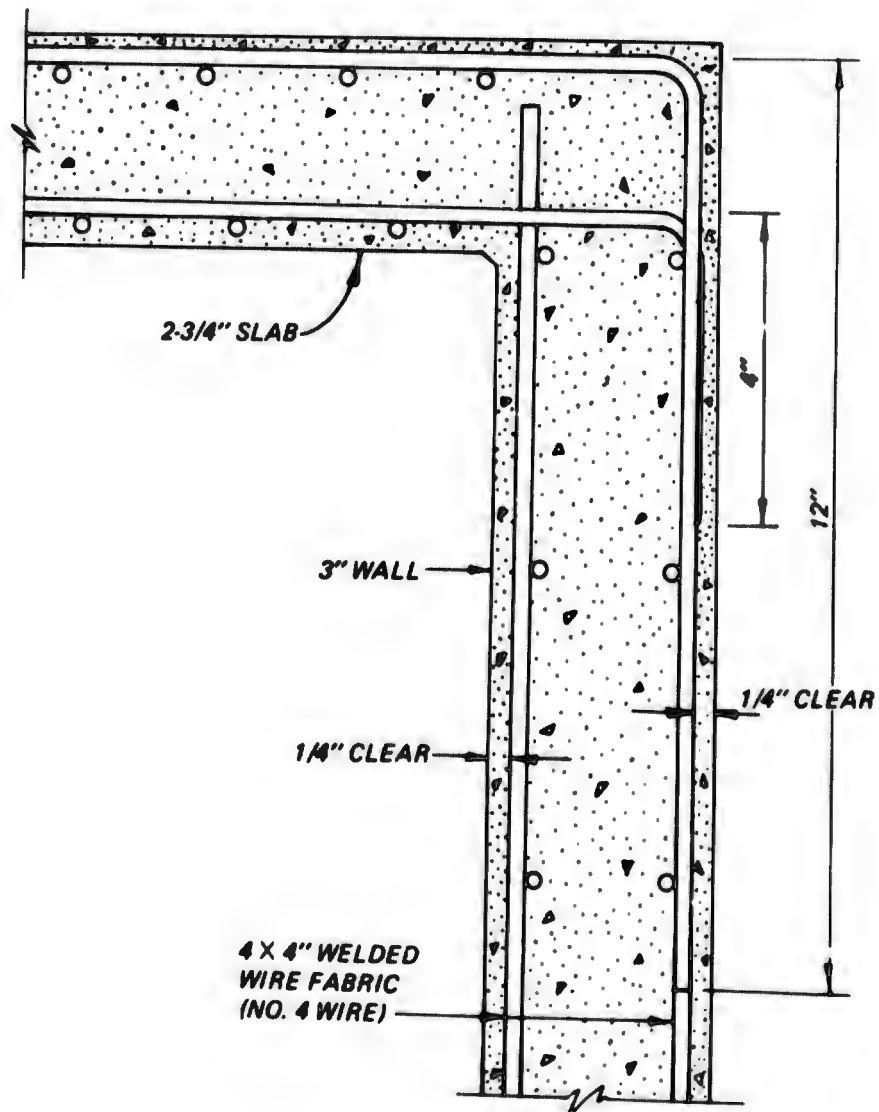


Figure 2.19 Details of the reinforcement near the top of the model walls.

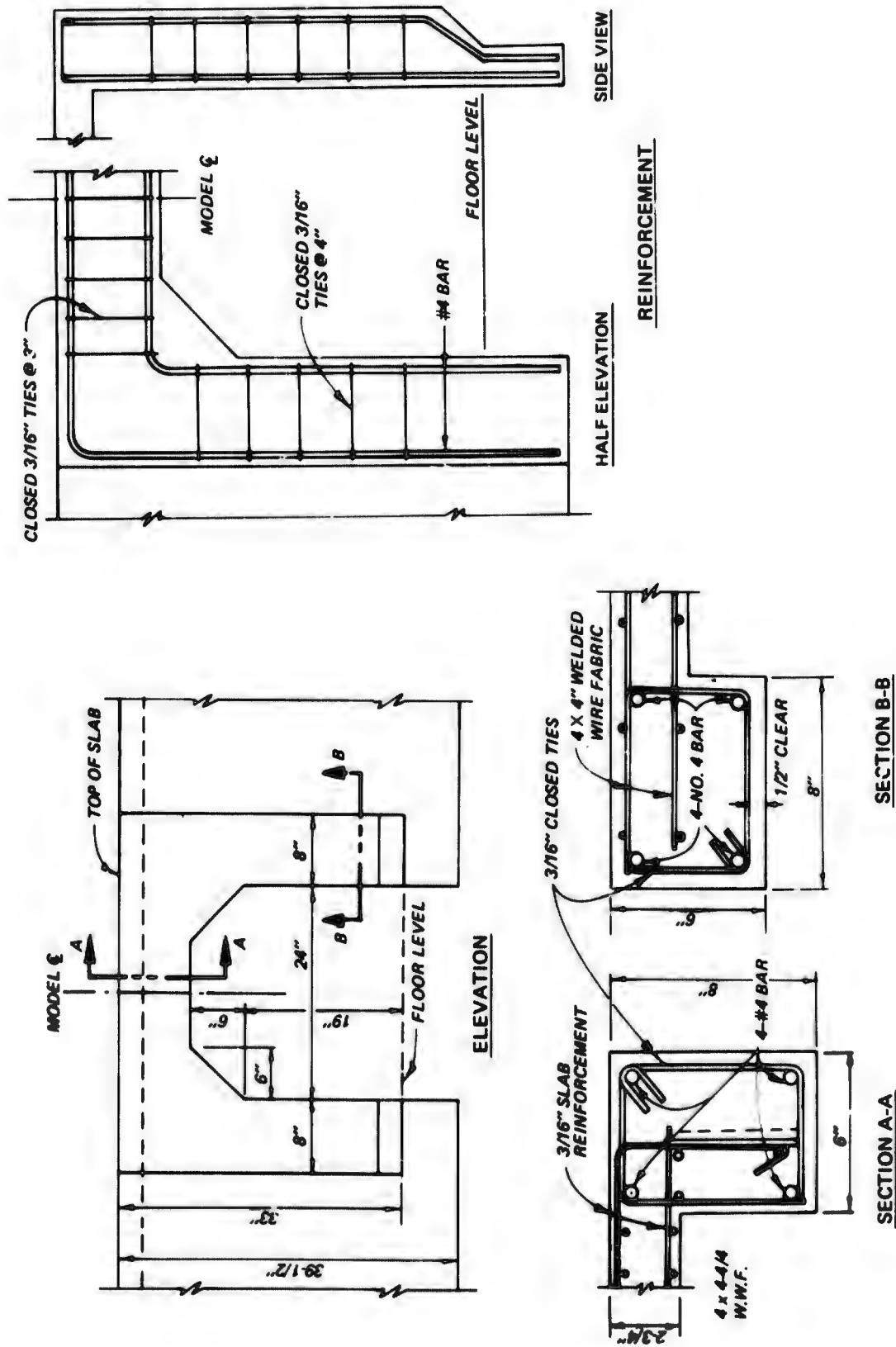


Figure 2.20 Details of side entrance in model wall.

CHAPTER 3

EXPERIMENTAL PROCEDURES

3.1 MATERIALS

3.1.1 Concrete. Through use of a trial mix procedure, both model structures were cast using concrete proportioned for a 28-day strength of 4,000 psi. Type I cement was used. The fine aggregate and coarse aggregate (3/8-inch maximum) used was crushed limestone of Tennessee origin. The mix proportion by weight was 1:3.43:3.46 (cement: fine aggregate: coarse aggregate). The water-cement ratio by weight was 0.78. The mix was proportioned to give a concrete with an average slump of 2 inches.

3.1.2 Reinforcement. Obtaining a suitable reinforcement is very often a major problem in the construction of structural models; this project was no exception. No deformed or indented wire having the required 3/16-inch diameter was available, nor was any straightened wire having the desired 42- to 46-ksi yield available. Therefore, the use of annealed wire or rod was investigated.

The effects on yield strength, ultimate strength, and stress-strain curve shape of annealing several types of wire at various temperatures were determined using a small annealing oven. The reductions in yield strength and ultimate strength at various annealing temperatures are shown in Figures 3.1 and 3.2, respectively. The wires were placed in a hot oven, held at a specified temperature for 2 hours, then allowed to cool in the oven for 12 hours.

On the basis of these results, C1018 cold drawn wire, available in straight 20-foot lengths, was selected for use and obtained in the needed quantity. The wire obtained in this larger batch, identified as C1018 No. 2 in Figures 3.1 and 3.2, was lower in strength than the first C1018 sample; its carbon content was probably lower within the allowable range for C1018 wire than was that of the first batch. This wire was annealed in a larger oven by a local manufacturing firm. The wire was heated to 1,200 F for 2 hours and cooled slowly.

In order to improve the bond characteristics, the wire was soaked in a muriatic acid solution to etch the surface and then allowed to rust. Loose rust was later removed by rubbing the wire with steel wool.

The larger bars (No. 3 and 4) used in the column and side entrance framing was Grade 40 billet steel reinforcement.

The 4- by 4-inch, 4-gage welded wire fabric used in the model walls met the requirements of ASTM Specification A 82, which specifies a minimum yield (offset) of 70 ksi.

3.1.3 Material Tests. Twenty-seven standard 6- by 12-inch concrete control cylinders were cast for each structure, three from each of nine batches of concrete placed in the structure (see Section 3.2.5). Two control cylinders from each batch of concrete were tested in compression on the test date of the corresponding model structure and the third was used to determine the tensile (split cylinder) strength also on the test date. Results of these concrete control cylinder tests are given in Table 3.1. Average concrete compressive strength was 4,190 psi for the static test structure and 4,970 psi for the dynamic test structure. Typical stress-strain curves obtained from cylinders instrumented with 6-inch-long strain gages are shown in Figure 3.3.

The properties of the 3/16-inch-diameter annealed wire used for the slab reinforcement were determined from 13 tensile tests conducted on lengths of the wire selected at random. The average yield stress and ultimate strength were 36.85 and 56.6 ksi, respectively (Table 3.2).

The stress-strain curves displayed a sharp yield point followed by a long, flat yield plateau extending to a strain of 0.030 to 0.035 in/in. The elongation at rupture over a 4-inch gage length was 35 to 40 percent.

The surface treatment and rusting of the bar resulted in a small drop (1-1/3 percent) in the yield strength from that of the annealed but untreated wire. The ultimate strength was reduced approximately 5 percent. The length of the yield plateau and the failure elongation were not changed appreciably by the surface treatment.

3.2 CONSTRUCTION OF THE MODEL

The first step in building the model structures was to construct a

base slab to serve both as a foundation for the structure during testing and as a platform upon which to construct the formwork for the test structures (slabs, walls, and columns). This formwork consisted mostly of dimension lumber and 3/4-inch-thick plywood. After the formwork for the inner surface of the model had been completed, all reinforcement was placed and tied. The outer forms for the walls were then set. The concrete for the walls, columns, and slabs was then placed in one operation.

3.2.1 Base Slab. The base slab, shown in Figure 3.4, was a 10-inch-thick slab with an approximately 40-inch-square access hole at its center and surrounded with heavy steel angles to support the walls of the model structure.

The primary function of the base slab was to provide a very rigid common foundation for the walls and four columns of the model during testing and to provide a method of restraining the bases of the walls in order that the desired complete fixity condition could be approximated. The base slab, when elevated on wooden cribbing, also served as a convenient work platform and support for the formwork. This arrangement gave the necessary access to the interior of the formwork and the advantages of constructing the formwork directly above the structure about which the model walls would later have to fit. In addition, the base slab provided the lifting points for handling the assemblage of the model structure and foundation when they were placed in the testing apparatus.

This slab, basically 14-foot-1-inch square (Figure 3.5), was reinforced with No. 5, Grade 40 bars placed in both faces and around the 4-foot-square access hole. The angles were reinforced with 1-inch-thick plates at the lifting locations, and blockouts were provided at eight locations in the slab for pressure transducers along with sufficient holes for bolting down the load cells to be placed under each column. Details of the slab edge region including the 6- by 4- by 1/2-inch-thick angles surrounding and anchored to the slab and the method of fixing the model walls are shown in Figure 3.6. The slab, ready to be cast in an inverted position, and details of the reinforcement are shown in Figures 3.7 and 3.8, respectively.

The 28-day strength of the ready-mix concrete placed in the base

slab, as indicated by three control cylinders, was 5,400 psi.

3.2.2 Formwork for Slab Model Structures. The inner formwork was constructed atop the base slab using kiln-dried dimension lumber and 3/4-inch exterior A-C grade plywood. The formwork was designed to be a very rigid structure that would withstand the process of placing the reinforcement and the concreting of the structure without undergoing detectable deflections. Special efforts were made to keep all dimensions within the desired tolerance of 1/16 inch or less. The formwork was constructed about 12 inches above the final position of the structure on the base slab. Blocking bolted to the heavy perimeter angle of the base slab (Figure 3.9a) formed the support for the wall forms. The inner wall forms (Figure 3.9b) were split vertically at the center of each side so that the sides could be collapsed inward when the forms were removed. These walls were braced diagonally to the column locations of the base slab.

The column and column capital forms were constructed of sheet metal and split into two halves (Figure 3.10). A steel column base plate with a hemispherical recess to receive a 1-inch-diameter ball to be placed atop the load cell and with four 6-inch-long No. 4 anchor bars welded to it was attached to the base of the column forms with machine screws. The top flange of the capital area form was attached with wood screws to a 1/2-inch plywood sheet the size of the drop panel.

The 2- by 6-inch joints for the bottom slab form rested upon the inner wall forms and were spaced an average of 12 inches apart (Figure 3.11a). Four double 2- by 6-inch stringers placed on each side of the drop panel locations and under the single 2- by 6-inch joints spanned the other direction and helped support and stiffen the single 2- by 6-inch joint system. One column form in place and the framing and blocking to support the other three column forms are shown in Figure 3.11a along with the diagonal bracing of the inner wall forms. Extreme care was taken to keep the formwork framing as rigid and true as possible.

The interior of the formwork is shown in Figure 3.12. Two rows of wales were placed on the inside wall forms to anchor the wall ties (Figure 3.12a). The bases of the column forms were supported against the

base slab, and posts were placed on each side of the column form to support the double 2- by 6-inch stringers.

Next, plywood sheeting was placed atop the joints and around the tops of the walls, and then the plywood joints were filled. Figure 3.13 shows the formwork at this stage. The blockouts at the bottom of the wall are to allow the bolts fixing the wall base and the lifting protection at each corner to pass through the walls.

The levelness of the slab surface was then examined by measuring elevations at approximately 75 scattered locations including points in the drop panel areas. To a limited extent, discrepancies could be corrected by shimming of the formwork. The extreme measured variations in elevation from the desired values over the lower slab and drop panel surface area were 0.009 and 0.006 foot (0.108 and 0.072 inch) for the first (static) and second (dynamic) models, respectively.

Before the reinforcement was placed, the outer wall forms were constructed in four pieces (one forming each side), assembled on the formwork structure, checked for dimension and fit, and then removed. Figure 3.14 shows the outer wall formwork for the side containing the wall access opening.

3.2.3 Fabricating and Placing the Reinforcement. The top and bottom bar mats were spaced, assembled, and tied using plywood jigs, and then positioned on the model formwork.

After loose rust had been removed from the bars by rubbing with steel wool, the bars were cut to length and bent, if necessary, using a 3/4-inch inner radius bend. These lengths were then placed in the jigs and tied securely at about 2-inch intervals to form the mat. Figure 3.15 shows the jig for the bottom mat with most bars in place. Bars to be strain-gaged were not included in the mat at this stage.

The first reinforcement placed on the formwork was that for the four columns. The assembled column reinforcement, consisting of six No. 4 Grade 40 vertical bars and a No. 12 cold-drawn steel wire spiral, is shown in Figure 3.16.

After the hardware needed to mount deflection gages and accelerometers had been attached to the formwork, the bottom mat was positioned

and nailed to the formwork using small staples. The 3/16-inch cover was insured by supporting the lower bars of the bottom reinforcing mat on short lengths of 3/16-inch-diameter wire at the more than 200 locations of the staples. Bars in the drop panels were supported on short sheet-metal angle chairs. Bars with strain gages were then tied into the mat. The bottom mat secured to the slab is shown in Figure 3.17. Some of the lifting lugs placed over each column and at the column lines on the wall are also visible. The entire bottom mat is shown in Figure 3.18.

The welded wire fabric for the wall reinforcement was next placed in two layers. These two layers were spaced by tying them to slotted 3-inch-long sheet-metal angles designed to fit against the inner and outer wall forms and to maintain a one-bar-diameter (0.225-inch) cover for both wall faces. These spacers can be seen along with blockouts for the soil pressure gages in Figure 3.19. The reinforcement around the access opening is shown in Figure 3.20.

The assembled top bar mat was then positioned on the slab. This mat was supported at approximately 130 positions using sheet-metal angle chairs that also served to hold down the bottom mat. The wires tying the top mat to these chairs extended through the formwork, thus insuring that the mat would not be lifted from its position during the concreting and vibrating operations. The top bar mat in the vicinity of a column along with the blockouts for the overpressure transducers for the dynamic tests are shown in Figure 3.21.

Placing and tying the top bars with strain gages and tying the hooks of the top bar mat to the wall reinforcement (Figure 3.22) completed the placing of the reinforcement.

The completed slab reinforcement is shown in Figure 3.23.

3.2.4 Preparation of Model for Casting. After the placement of the reinforcement had been checked and all bars of the top mat that were instrumented with strain gages had been installed, the wall forms were set in place and secured by nailing down the bottom of the forms and by installing ten or twelve 1/4-inch-diameter threaded rods per side used as wall ties on two rows of wales. The sheet-metal angles spacing the wall reinforcement also held the wall forms the desired 3 inches apart.

A 2-1/2- by 2-1/2-inch angle was bolted to the top of the wall forms flush with an inner face of the outer form. This angle was then leveled using shims to within ± 0.002 foot of the desired elevation with the aid of a precision level. This perimeter angle, shown in Figure 3.24, formed the support for the slab screed.

The completed formwork and model reinforcement ready for the casting of the concrete are shown in Figures 3.25 and 3.26.

3.2.5 Casting of the Model. The walls and slab of the model were cast in one continuous operation taking about 4 hours.

The concrete was mixed in 12-ft³ batches using a 1/2-yd³ capacity cyclo-turbine mixer. Nine batches were used for each model. The proportions of the mix used are given in Section 3.1.1.

Three control cylinders were cast from each batch of concrete. These cylinders were cast in standard 6- by 12-inch steel molds and vibrated 15 seconds using an internal vibrator with a 1-1/4-inch-diameter round head.

The concrete was placed first in the walls, then in the column and slab area. The sequence of placing of the concrete is shown in Figure 3.27. The concrete was consolidated as it was placed using 1-1/4-inch square-headed internal vibrators. Planks with bolt-legs that sat upon the slab form were used to provide access to the top of the slab. The concreting operation is shown under way in Figure 3.28.

After the concrete had been placed and vibrated, the top surface was struck and screeded using a 6- by 2-inch aluminum structural tubing section supported on the perimeter angle bolted to the top of the outer wall forms. The surface was finished several hours later.

3.2.6 Curing of the Models, Removal of the Formwork, and Condition of the Model. The models and the control cylinders were cured under wet burlap for 7 days and then air-cured until testing.

The formwork was removed after 21 to 27 days. After the outer wall formwork had been removed, the four lifting lugs over the columns and the eight lifting lugs in the walls were located and exposed. A 6- by 4- by 3/16-inch rectangular structural tubing strongback was then attached to the two columns and two wall lifting lugs along each of two

parallel column lines. This strongback served both to support the slab column areas, thus preventing possible damage to the model when the supports under the columns were removed, and also to facilitate lifting the 13,000-pound model before it was attached to the base slab.

The ties holding the slab mats to the formwork were then cut on the inside of the form, and the liner formwork was removed. All of the inner formwork except the plywood decking was reused for the second model.

A close inspection of each model structure showed that both were free of honeycomb-like cells and voids except for a very minor amount along the base of part of the walls. The only cracks found in either slab structure model were short (1 to 3 inch), fine cracks extending from the corners of one or more of the four blockouts to accommodate the four lifting lugs of the base slab. The sharp corner at the perimeter of the drop panels of the first structure (for static testing) was broken off in places (Figure 3.29) and led to a small modification of the formwork for the second model structure.

Each of the model structures was removed from the base slab and stored indoors until time to attach the model to the base slab for testing. Figures 3.29 and 3.30 show the interior of the first (static) model structure.

3.3 TESTING PROCEDURE

3.3.1 Preparation of the Test Specimen. Before the slab structure model was connected to the base slab, an aluminum frame to support the deflection instrumentation for the slab was fabricated and attached to the base slab. The frame on the base slab is shown in Figure 3.31. The two model structures can be seen in the background in the figure.

The model was then fitted on the base slab. The walls were supported on the step formed around the perimeter of the base slab by the 6- by 8- by 1/2-inch angle sections and shimmed to the desired height above the horizontal leg of the angle. The load cells placed under each of the four interior columns were shimmed from the base slab snugly against the column. A nonshrink grout was then placed in the voids

under the four interior load cells and between the model walls and the base slab (see Figure 3.6). The mix design for the grout used was as follows:

Type III cement	94 pounds
Fine sand (Cook's Bayou sand, $D_{50} = 0.33$ mm)	100 pounds
Water	6 gallons
Powered aluminum	2 grams
7-day design strength	3,000 psi

Seven days after the grouting operation, the 24 high-strength, 7/8-inch-diameter bolts connecting the 5- by 3- by 3/8-inch structural tubing perimeter beam to the base slab (Figure 3.6) were tightened. The model at this stage is shown in Figure 3.32. Figure 3.33 shows details of one corner of the structural model attached to the base slab.

The average load on the four interior column load cells after the grouting operation and before the removal of the supporting strongback beams over the slab was 335 pounds.

Most instrumentation, including deflection transducers, pressure transducers, and accelerometers, was installed while the models were in the position shown in Figure 3.32.

3.3.2 Loading Device. The LBLG at WES was used to test both the statically and dynamically loaded models. This device, shown in Figure 3.34, consists of two basic components, the Central Firing Station (CFS) and the test chamber. The test chamber consists of three 22-foot-10-inch-ID high-strength steel rings stacked on a movable platen to form a cylindrical chamber 10 feet deep. After the model to be tested and backfill material are placed in this chamber, a ring containing baffled firing tubes and a lid are placed on the three rings of the tank below and moved into the CFS, which is a massive posttensioned concrete reaction structure.

To obtain dynamic loadings, Primacord explosive is placed in the firing tubes and detonated, resulting in a downward-traveling plane wave.

The LBLG can simulate the pressures produced by kiloton and megaton nuclear devices. Surface airblast overpressures from less than 10 to 500 psi can be produced with rise times of 1 to 10 msec and durations in the range of 1/2 to 2 seconds.

Slowly applied or static loadings of up to 1,000 psi can be obtained by sealing the exhaust ports of the top ring of the tank and using the tank as a pressure vessel.

More complete descriptions of the LBLG, its performance, and calibration are given in References 22 and 23.

3.3.3 Placement of the Model and Backfill. Before the model structure was placed in the test chamber, the chamber was filled to the elevation of the base slab with a fine, fairly uniform sand and leveled to provide a uniform support for the base slab foundation of the model structure. The base slab elevation was selected so that the top of the structure would be slightly below the top of the three tank rings of the test chamber.

A steel entranceway box approximately 3 feet square, 4 feet deep, and with one side and top open was placed adjacent to the access hole at the center of one wall of the model.

The test chamber was then backfilled to the level of the slab surface. Care was taken to uniformly compact the backfill material.

The fairly uniform, fine sand used for backfill material is known locally as Cook's Bayou sand and has been used for several previous studies conducted in the LBLG. The sand has a specific gravity of 2.65, minimum and maximum laboratory dry unit weights of 93.3 and 110.3 pcf, and a D_{50} value of 0.33 mm; 87 percent of the material is between a number 30 and a number 100 U. S. standard sieve size and consists of predominantly subround to round shapes. Other properties of the sand are presented in References 24 and 25. The density of the backfill as placed was approximately 103 pcf.

The electrical leads for the transducers were connected to cables running from the instrumentation room adjacent to the LBLG, through the base of the test chamber, and through the hole in the base slab serving as a foundation for the model structure. After the instrumentation had

been checked, the lid of the entranceway box was closed, and the model and soil surface were sealed as required for the type of test to follow.

3.3.4 Static Test. For the static test, water pressure was used to apply a uniform load. Thus, it was necessary to seal the backfill material and structure from the water. A layer of 1/16-inch-thick unreinforced neoprene was placed over the model area. A 6-mil-thick layer of polyethylene was placed atop this and taped to the sides of the chamber. The top layer of the water seal was a 0.035-inch-thick polyurethane diaphragm covering the test area surface and glued to the sides of the chamber.

Water pressure rather than air pressure was chosen as a method of loading to prevent a sudden failure that air expansion would cause when the resistance of the structure began to decrease. Water loading allows a better determination of the entire deflection-resistance curve.

After the water seal had been completed, the top ring and lid of the test chamber were installed and the chamber was moved into the CFS. The air void above the model was then filled with water, although it was determined after the test that an approximately 3-inch-thick layer of air remained at the top of the tank and above the exhaust ports.

Water pressure for loading was obtained by passing tap water through a pressure regulator. The water pressure in the water mains at the time of testing was approximately 80 psi. The first step in the static test was to record calibration steps for the 85 transducers. Water pressure was then applied to the slab surface at a rate of approximately 2 psi per minute until a load peak was obtained at nearly 14 minutes into the test. Loading was continued for another 4-1/2 minutes before the water sealing membrane ruptured. One water pressure transducer, one slab panel center deflection channel, and one Bourdon-type mechanical pressure gage connected to the water chamber above the model were monitored throughout the test.

The test chamber was then rolled out of the CFS, the top rings, lid, and sealing membranes removed, and the damaged model structure examined and photographed. The backfill was excavated to the level of the base slab, and the model was removed from the test chamber to allow

inspection of the walls and further inspection and photographing of the slab portion of the model.

3.3.5 Dynamic Tests. The first of several differences between the procedures for the static and dynamic tests was in the sealing.

A 6-mil-thick layer of polyethylene was taped to the edge of the model and the test chamber walls to serve as a barrier to prevent the airblast pressure from entering into the sand. Approximately 1 inch of sand was placed over the polyethylene to protect it from heat. The roof of the model was not covered with the polyethylene. A fine mesh copper screen was placed over the surface to shield the model instrumentation from electrical disturbances resulting from the explosion.

The exhaust ports in the top ring were left open for the two dynamic tests.

Before the top ring and lid were placed on the test chamber, the amount of Primacord explosive necessary to produce the desired pressure, based on calibration curves and past use of the LBLG, was strung in the firing tubes atop the model location in the test chamber. The test chamber was then closed and placed in the CFS.

Overpressure was generated by detonation of the Primacord after the calibration of the transducers had been completed and the magnetic-tape recorders had reached operating speed.

The first dynamic test resulted in only slight damage to the model. The backfill and model structure were removed from the test chamber only after the second dynamic test.

3.4 TEST MEASUREMENTS AND INSTRUMENTATION

The total number of transducers used during any one of the static or dynamic tests was limited to a maximum of about 90 (the number of magnetic-tape channels available for the continuous recording of data).

Several variations of transducer layouts and mixtures of types of measurements were considered before the layout presented herein was accepted as the one that would potentially give the optimum amount of useful information.

Much of the instrumentation, especially strain gages, was

concentrated in one-quarter of the slab (the northwest quarter), and, in the case of the strain gages, placed in one direction only. This was done to allow a reasonably accurate determination of the stress pattern and moment distribution in that part of the slab. All other quarters and the other direction are, by symmetry, nearly identical. Other transducers, especially air pressure and slab deflection gages, were scattered over the slab surface in order to determine the gross loading and behavior of the entire slab system.

All instrumentation on the walls was concentrated on the north half of the west wall adjacent to the slab quarter containing the most instrumentation.

3.4.1 Pressure and Load Measurements. The airblast and interface pressures acting on the structures were measured using commercially available pressure transducers. Four load cells were constructed to measure the vertical load supported by the four interior columns.

The magnitude of the applied load during the static test was measured with three 100-psi-range Consolidated Electrodynamics Corporation (CEC) pressure transducers placed around the test chamber perimeter.

Nine CEC pressure transducers of 100-psi range were employed to measure airblast pressures in the dynamic tests. Seven were placed at various locations on the slab surface, as shown in Figure 3.35, and two were placed inside the structure on the base slab to determine any pressure leakage into the structure. All were placed with the axis of the transducer vertical and the sensing face upward. The transducers were held flush with the slab surface by hollow Teflon mounting bolts placed in formed holes in the slab. This mounting system served to reduce the acceleration and ringing experienced by the transducer.

The pressure transducers used to measure the overpressure and the mounting apparatus are shown in Figure 3.36 along with the pressure transducers for measuring interface pressures discussed below.

The interface pressure acting on the model wall was monitored at eight locations on the north half of the west wall. The locations shown in Figure 3.35 were used for both structures.

Eight CEC pressure transducers (25- and 50-psi ranges) were mounted

in threaded steel mounts cast into the wall of the model for the static test. For the dynamic tests, the same pressure transducers were held flush with the wall surface using the same hollow Teflon mounting bolts used with the pressure transducers on the slab surface.

The four load cells, placed one under each interior column, consisted of an instrumented 4-inch-long section of 2-1/4-inch-ID by 2-3/4-inch-OD seamless steel tubing press fitted onto two 6-inch-square steel end plates 1-1/2 inches thick and machined to a 3/4-inch thickness except for the area fitting into the steel tubing. One of the end plates contained a centrally located depression to receive a 1-inch-diameter hardened steel ball, and the other plate contained four holes for bolting the load cell to the base slab. Each load cell was instrumented with four 1/4-inch gage length, 120-ohm, epoxy-backed, foil strain gages (Budd C6-141-B) placed in a full-bridge configuration at the midheight of the tubing. The two vertical gages were placed diametrically opposite and 90 degrees from the two horizontal strain gages.

The calibration of the load cells showed a linear response up to the maximum calibration load of 100 kips. A strain of 1 $\mu\text{in/in}$ corresponded to from 22.92 to 23.27 pounds for the four load cells. Two of the load cells are shown in Figure 3.37.

3.4.2 Deflection and Acceleration Measurements. Deflections of the slab were measured at the center of each of the nine panels and at four midspan locations along column lines. The horizontal deflection of a point at the center of the west model was also determined. The deflection transducer locations are shown in Figure 3.35.

Slab deflection measurements were obtained using 7-inch-capacity linear potentiometers manufactured by Topp Industries. These transducers were clamped in mounting brackets bolted to the deflection framework placed inside the model (Figure 3.31) and attached to the lower slab surface by one of two methods. For the statically tested model, the probes of the deflection transducers were attached with Shrinktubing flexible plastic tubing to short studs centrally located on a 1-1/2-inch-square pull pad attached with epoxy to the slab bottom surface. For the dynamic tests, the probes were connected with Shrinktubing to the

exposed threads of 3/16-inch-diameter bolts with the head ends anchored into the slab.

A $\pm 5/8$ -inch-capacity Collins linear variable differential transformer (LVDT) was clamped onto the deflection frame and used to measure the horizontal (inward and outward) wall motion.

The deflection transducers and the acceleration transducers used for the dynamic tests are shown in Figure 3.38.

Accelerations were recorded at nine locations during the dynamic tests. Vertical accelerations were measured at five locations on the slab surface adjacent to deflection transducers at either the center of the span along column centerlines or at panel centers, at the base of the drop panel 1 inch from the northwest column capital, at the center of the west wall, and at the base of the northwest column. The last two transducers were included primarily to determine the vertical motion of the model structure base slab during the dynamic tests. The horizontal acceleration of the center of the west wall was also recorded. The locations of these transducers are shown in Figure 3.35.

The accelerometers (CEC, Model No. 4-202-001 with a range of ± 500 g's and Statham, Model No. A69TC-250-350 with a range of ± 250 g's) were rigidly mounted to the structure. Those on the bottom slab surface were attached with four machine screws to steel inserts cast into the slab. The accelerometer at the base of the northwest column was similarly attached to the bottom load cell plate. One small aluminum block bolted to an insert cast in the wall held the two accelerometers on the west wall.

3.4.3 Strain Measurements. The locations of the strain gages placed on the reinforcement and the slab surfaces are shown in Figures 3.39 through 3.42. The number of gages used was reduced for the dynamic tests to allow the addition of the necessary surface pressure and acceleration instrumentation. Because of the bar spacings used, many gages on the reinforcement were placed on the bar closest to the location shown rather than precisely at the location shown.

The steel strains were measured using Budd C6-141-B epoxy-backed foil gages. These gages had a nominal grid size of 1/4-inch length by

1/8-inch width and a resistance of 120 ± 0.2 ohms.

The bars to be gaged in the slab reinforcement were prepared for gaging by cleaning and lightly sanding with emery cloth at the gage location. This resulted in a very small reduction of the bar at the gage location. The gages were then bonded to the bar with a heat-curing epoxy. After the installed strain gages had been inspected, lead wires were attached to the gage tabs, and the locations were waterproofed using several layers of Gagekote synthetic rubber compounds.

The gages on bars with hooks were placed at 90-degree angles to the hook. Straight bars were tied in the reinforcement layers with the strain gage on the side of the bar rather than at the top or bottom. The above-described gage placements were used to reduce the influence of any bending of the bar as much as possible without using two strain gages per gage location.

The Budd C6-141-B strain gages installed on the welded wire fabric for the wall reinforcement were also installed on the side of the wire (see Figure 3.41) and with the same procedure used for the slab flexural reinforcement.

The same installation procedure and strain gage size and type were also used for the gaged locations on the column steel except that several deformations of the No. 3 bar at the gage location were removed. The bars with strain gages were placed in the column reinforcement cage with the gage to the outside of the cage.

Budd C6-1161-B, epoxy-backed foil gages having a nominal grid size of 1-inch length by 3/32-inch width and a gage resistance of 120 ± 0.2 ohms were used to determine the concrete strains at the locations shown in Figure 3.42. After the concrete voids at the gage location had been filled with a thin layer of Epocast epoxy, the location was sanded until the concrete surface was exposed. The gage was bonded to the concrete with Eastman 910 cement and waterproofed with Gagekote synthetic rubber.

3.4.4 Data Recording and Processing. All data for the static and dynamic tests were recorded on magnetic tape using seven 14-channel magnetic-tape recorders (Sangamo Model 4784 and Sangamo Model 3564).

One channel was used internally by each recorder in all tests and a 1,000 Hz timing signal was placed on another channel in the dynamic tests. Timing and correlation for the static test were supplied by interrupting the signal from a pressure transducer (P1) and recording this signal on the first channel of each tape. A block diagram of the recording system used is presented in Figure 3.43.

The signals from all transducers except the Collins LVDT were amplified by WES differential analog module (DAM 103) amplifiers, which also provided excitation for the gages and contained the calibration and balancing circuitry. The sensitivity of the Topp deflection transducers used for measuring the slab surface deflections was multiplied by a factor of 10 during the early part of the static test in order to provide a more usable signal from the small deflections expected during the first part of the static test. To obtain this multiplication, the gain control on the amplifiers was manually changed from 100 to 10 when the applied pressure reached a designated level.

The magnetic-tape speed was 7-1/2 in/sec during the static test and 60 in/sec during the dynamic tests.

Calibration for the data was provided by electrically inserting a known equivalent gage output into the circuitry of each data channel and recording the resulting output for a brief interval prior to each test. The effect of cable resistance was included in the calibration.

The data were later reproduced on photographic paper using the tape output as the input to a CEC Data-Graph 5-133 recorder. The data from the static test were digitized using an Edwin plotter. An analog-to-digital converter was used to digitize the records from the dynamic test. The data were then processed and plotted with the aid of a computer. Several accelerations were doubly integrated (numerically) to obtain deflections.

TABLE 3.1 RESULTS OF CONCRETE CONTROL CYLINDER TESTS

Model for Static Testing				Model for Dynamic Testing			
Concrete Batch	Slump	Compressive Strength at Time of Model Test ^a	Split Cylinder Strength at Time of Model Test ^a	Concrete Batch	Slump	Compressive Strength at Time of Model Test ^b	Split Cylinder Strength at Time of Model Test ^b
	inches	psi	psi		inches	psi	psi
1	2	4,460 4,140	455	1	2-3/4	4,730 4,640	520
2	2	4,320 3,890	450	2	2-1/4	4,780 4,980	540
3	2-1/2	4,040 3,890	405	3	2-1/2	4,910 5,040	515
4	3	4,140 4,070	500	4	2-3/4	4,790 4,680	555
5	2-1/4	4,310 4,270	460	5	2-1/2	4,810 4,990	470
6	2-1/4	4,210 4,390	415	6	2-1/2	-- 4,930	495
7	2-1/4	4,050 4,020	485	7	2-1/2	5,550 5,960	530
8	2-1/2	4,070 4,730	455	8	2-1/4	4,910 4,930	410
9	2-1/4	c c	c	9	2-1/2	5,060 4,840	475
Average	2-1/4	4,190	455		2-1/2	4,970	500
Coefficient of Variation		0.051	0.069			0.064	0.084

^a Cylinders broken at time of first static test (181 days).

^b Cylinders broken at time of first dynamic test (201 days).

^c Cylinders misplaced or damaged.

TABLE 3.2 RESULTS OF STATIC TENSILE TESTS ON 3/16-INCH-DIAMETER REINFORCEMENT

Specimen Number ^a	Load at Yield ^b	Yield Stress ^c	Ultimate Load	Ultimate Strength
	pound	psi	pound	psi
1	985	35,700	1,540	55,800
2	1,005	36,400	1,540	55,800
3	1,035	37,500	1,595	57,800
4	990	35,900	1,550	56,100
5	1,025	37,100	1,590	57,600
6	1,035	37,500	1,600	58,000
7	1,035	37,500	1,605	58,100
8	1,025	37,100	1,595	57,800
9	1,040	37,700	1,600	58,000
10	1,040	37,700	1,595	57,800
11	1,005	36,400	1,505	54,500
12	1,000	36,200	1,485	53,800
13	<u>1,010</u>	<u>36,600</u>	<u>1,515</u>	<u>54,900</u>
Average	1,018	36,850	1,563	56,600

^a Specimens 1 through 10 from annealed bar before surface treatment; Specimens 11 through 13 from bar with surface etched and treated.

^b Loading rate for all specimens in elastic range was approximately 1,500 psi/sec.

^c Stresses based on nominal area of 3/16-inch diameter (area = 0.0276 in²).

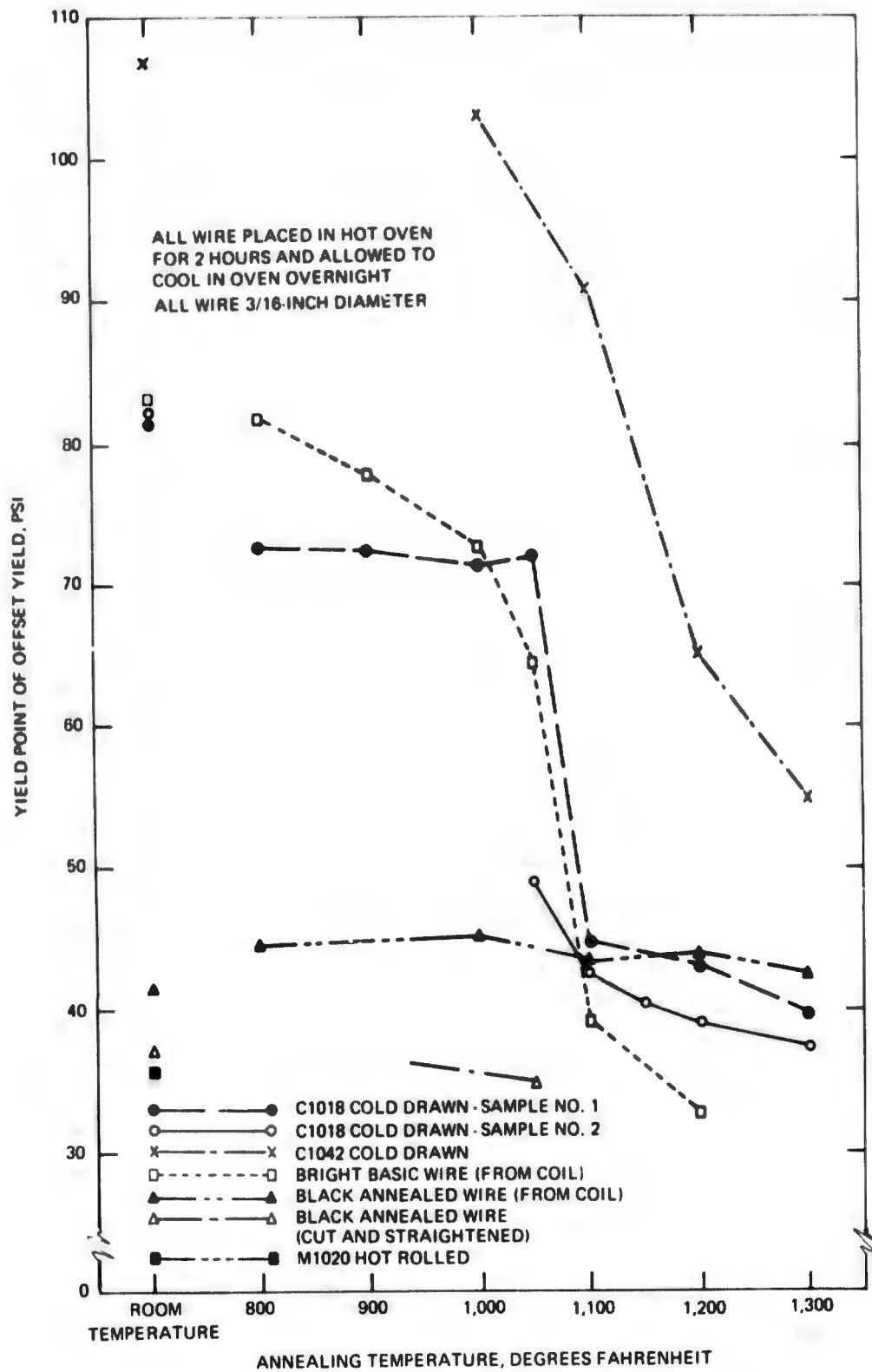


Figure 3.1 Effect of annealing temperature on yield strength.

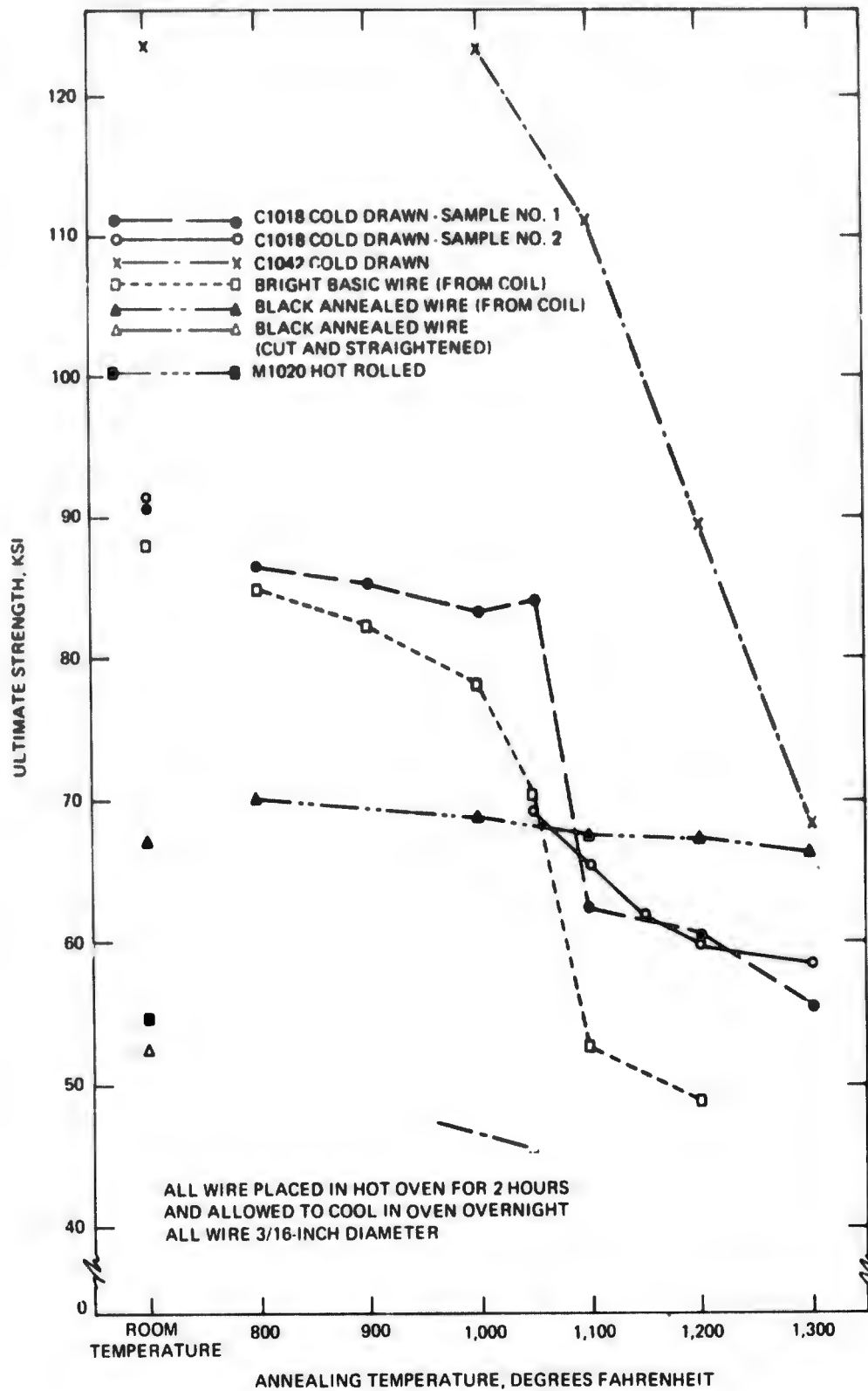
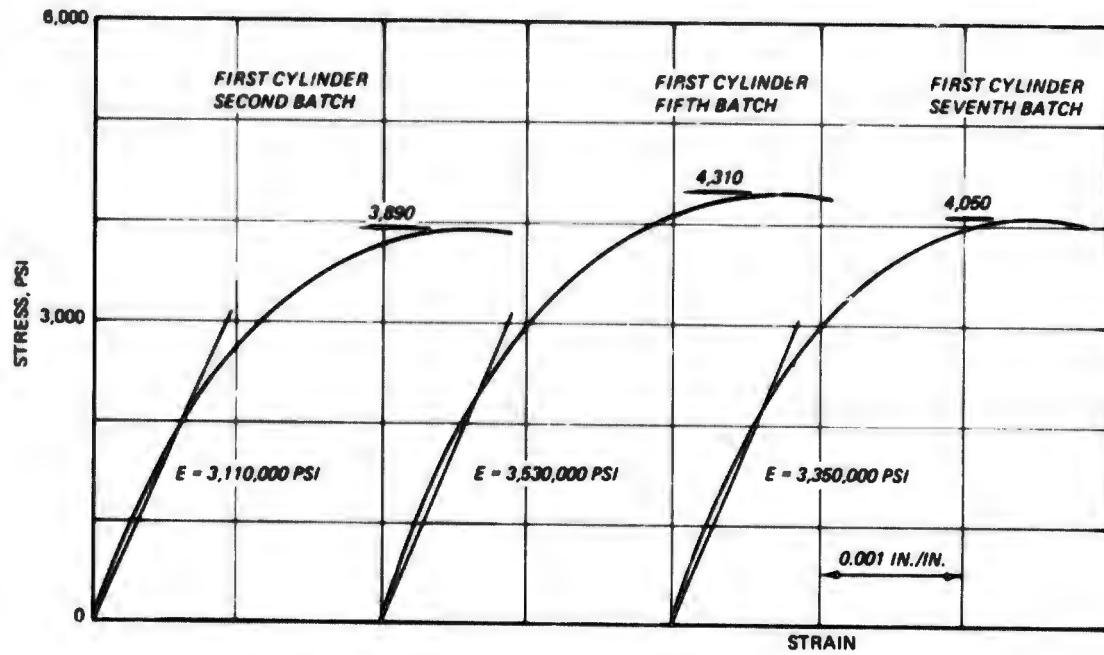
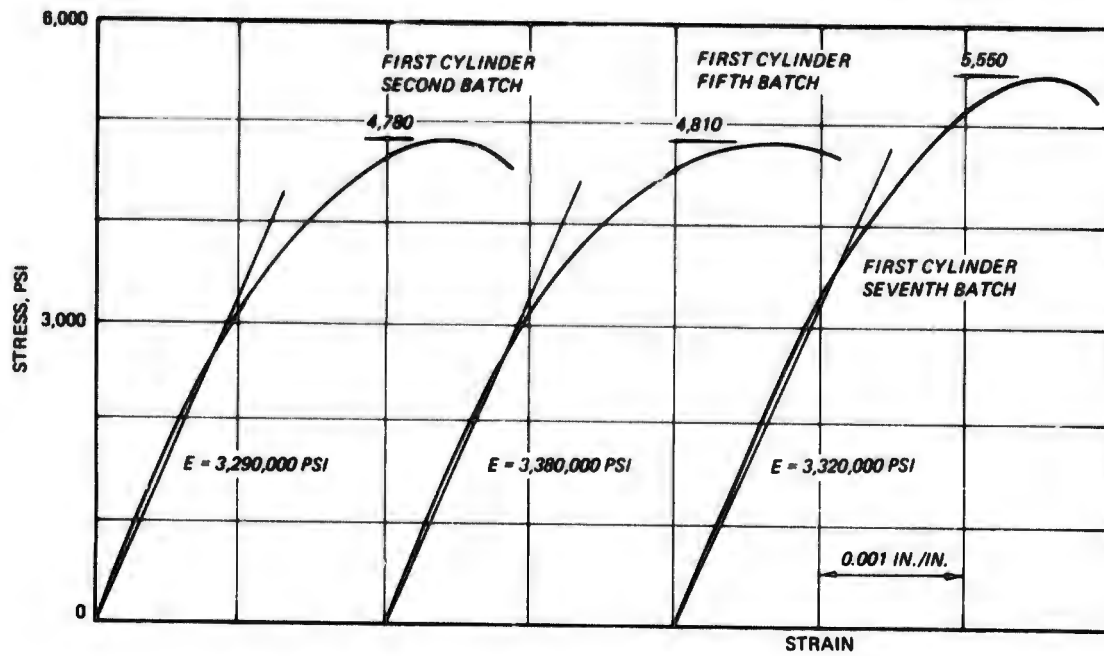


Figure 3.2 Effect of annealing temperature on ultimate strength.



a. CYLINDERS CORRESPONDING TO STRUCTURE FOR STATIC TESTING



b. CYLINDERS CORRESPONDING TO STRUCTURE FOR DYNAMIC TESTING

Figure 3.3 Typical static stress-strain curves for concrete control cylinders.



Figure 3.4 Base slab ready for start of model formwork construction.

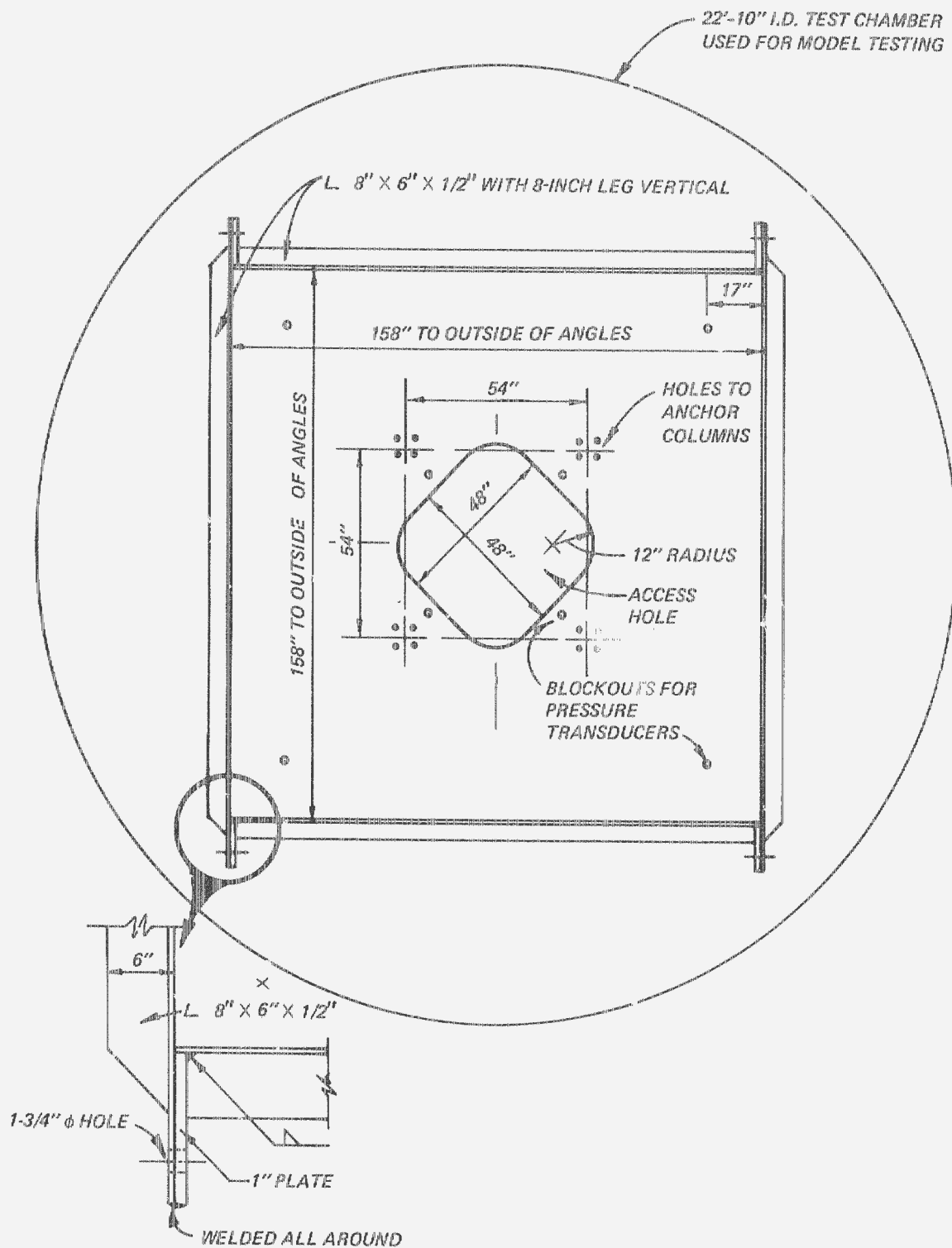


Figure 3.5 Plan view of the base slab.

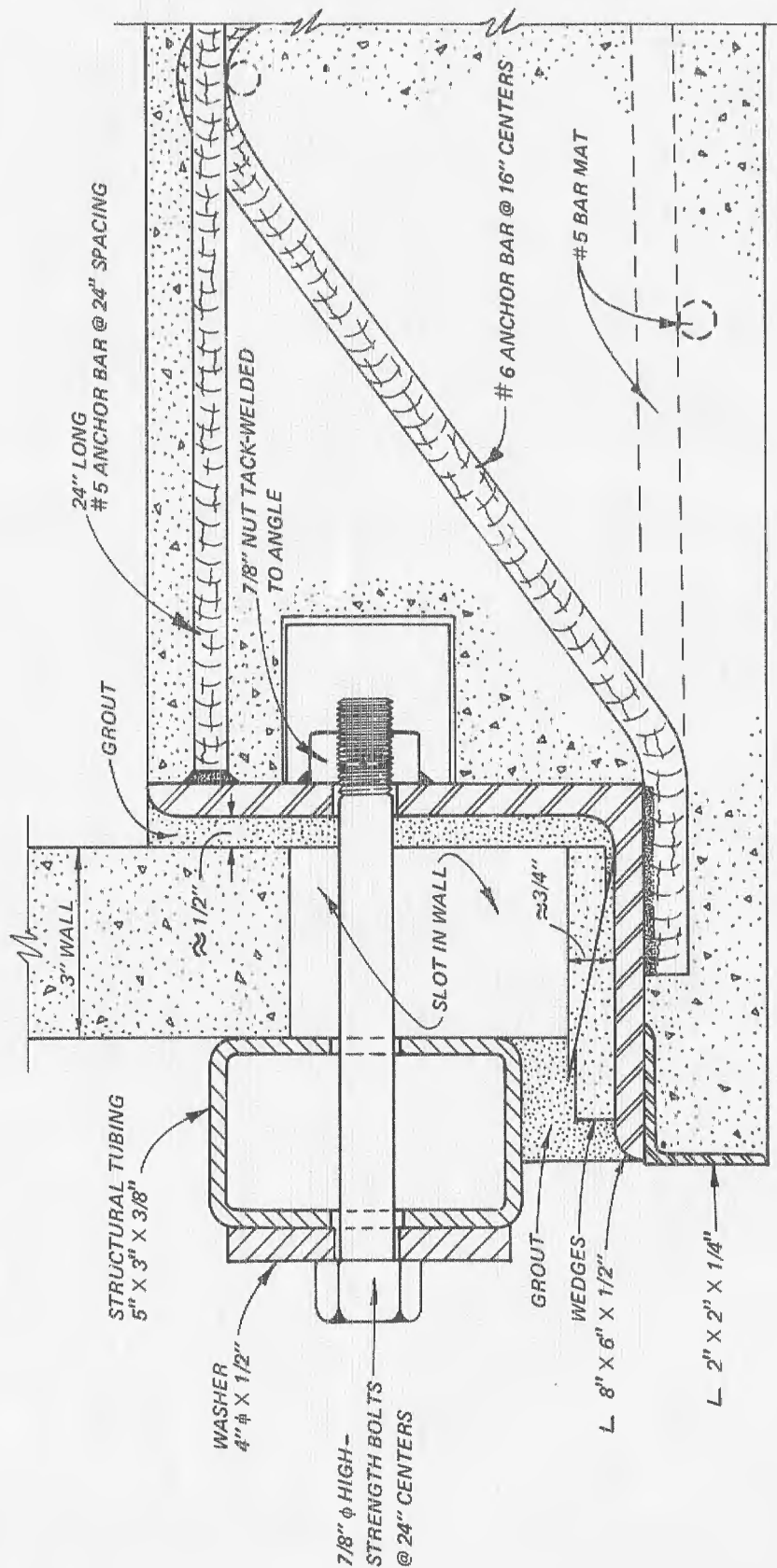
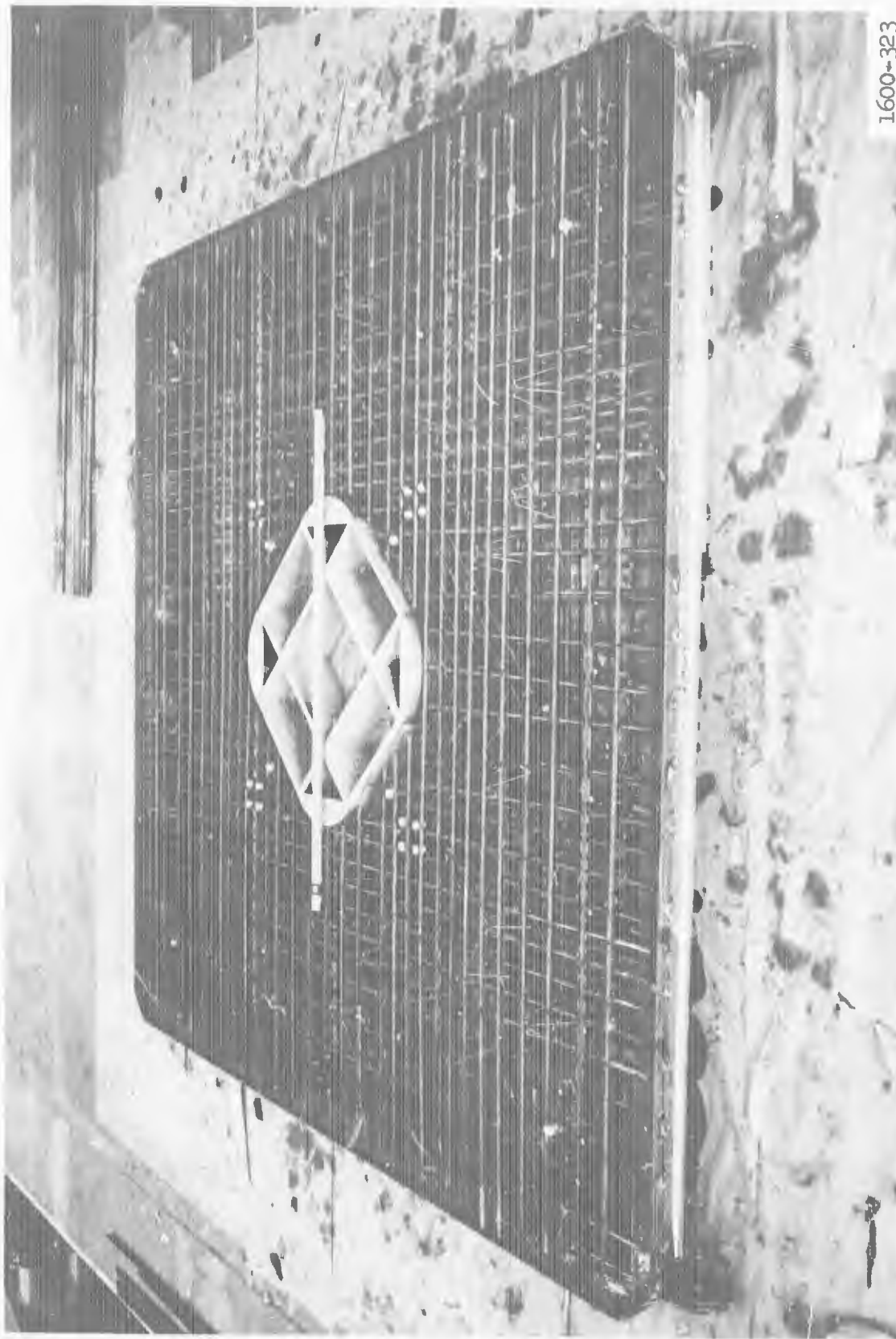
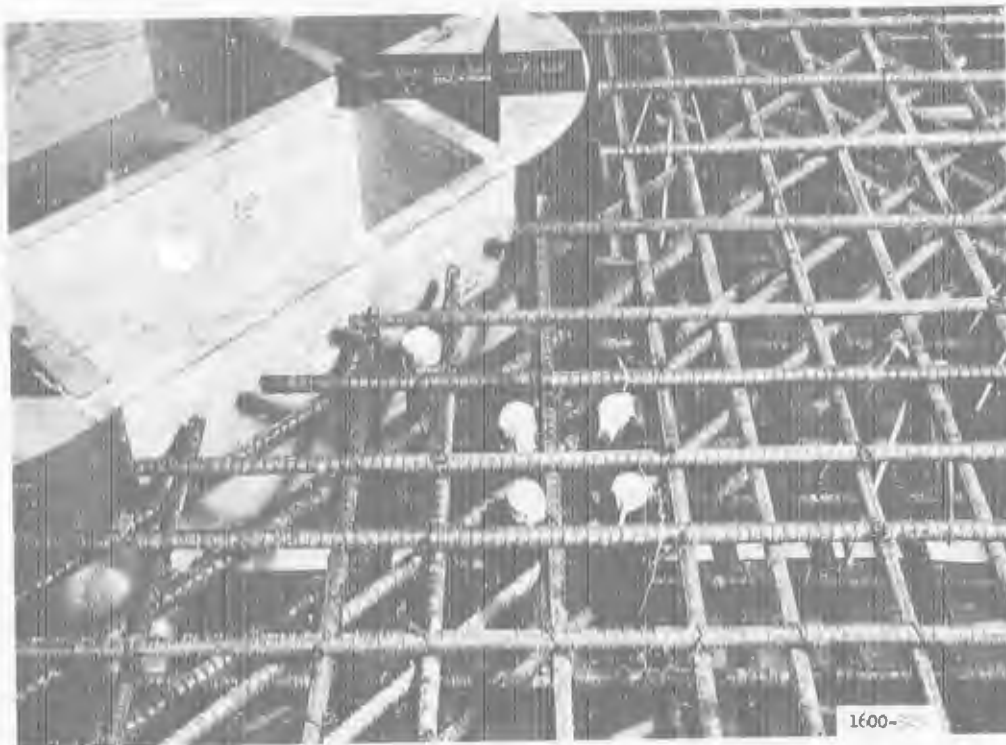


Figure 3.6 Details of the base slab edge and method of fixing base of wall.

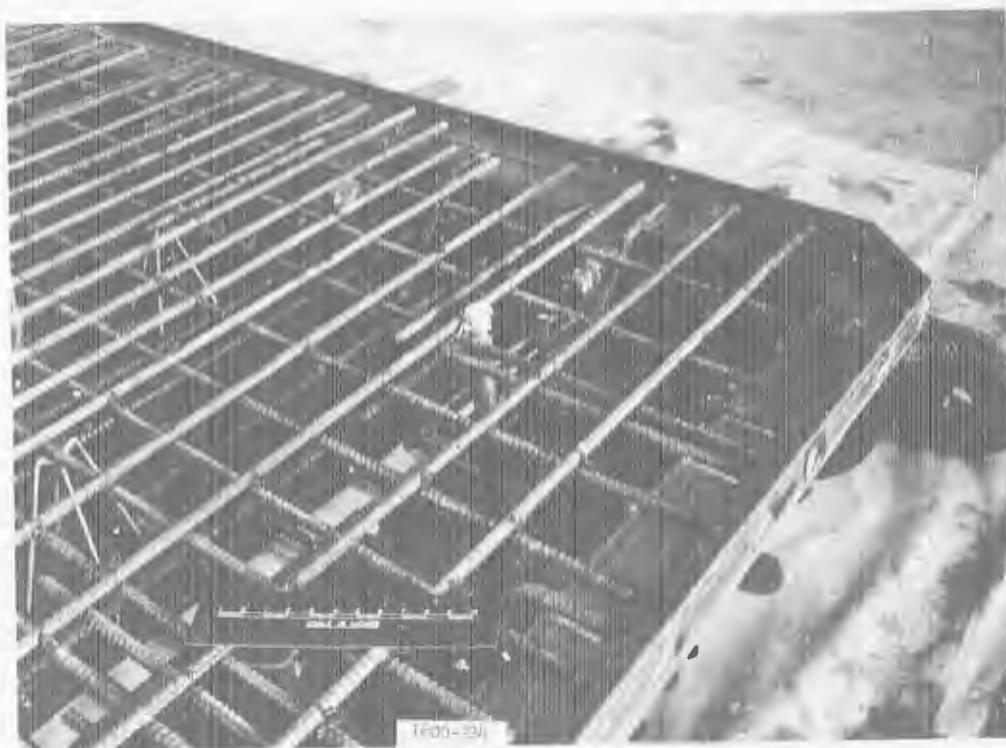


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Figure 3.7 Base slab (inverted) before casting of concrete.



a. At a column location and the center access hole.



b. At a corner of the slab.

Figure 3.8 Details of base slab reinforcement and construction.



a. Base of wall forms.

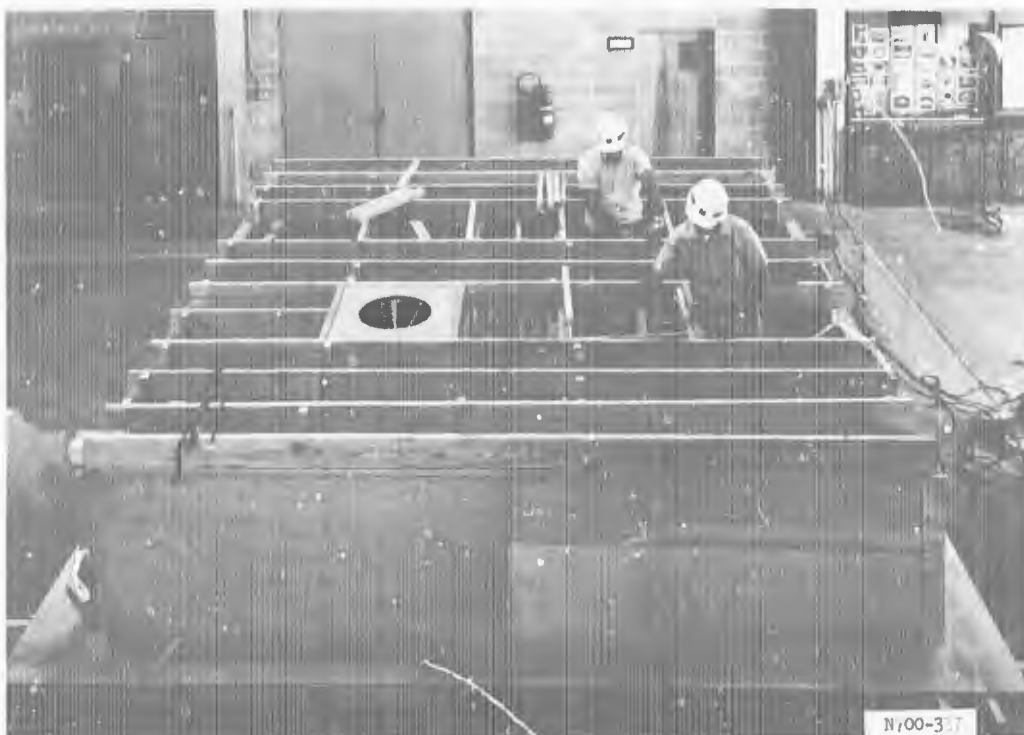


b. Overhead view of inner wall forms.

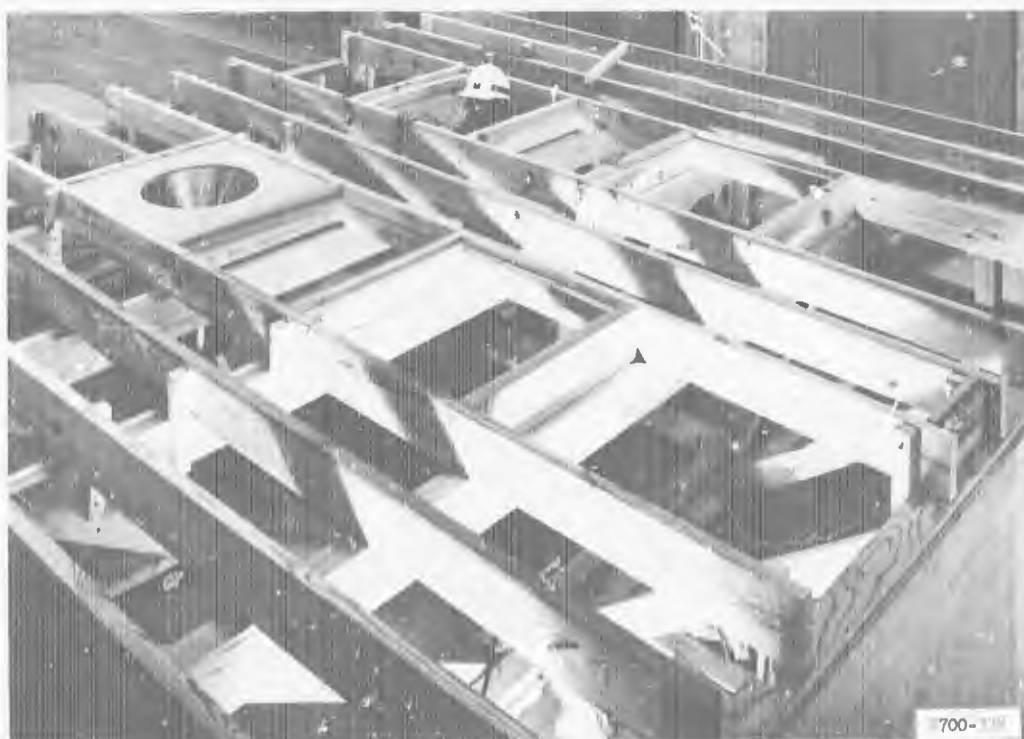
Figure 3.9 Construction of model formwork, inner wall forms.



Figure 3.10 Column and capital forms and column base plates.

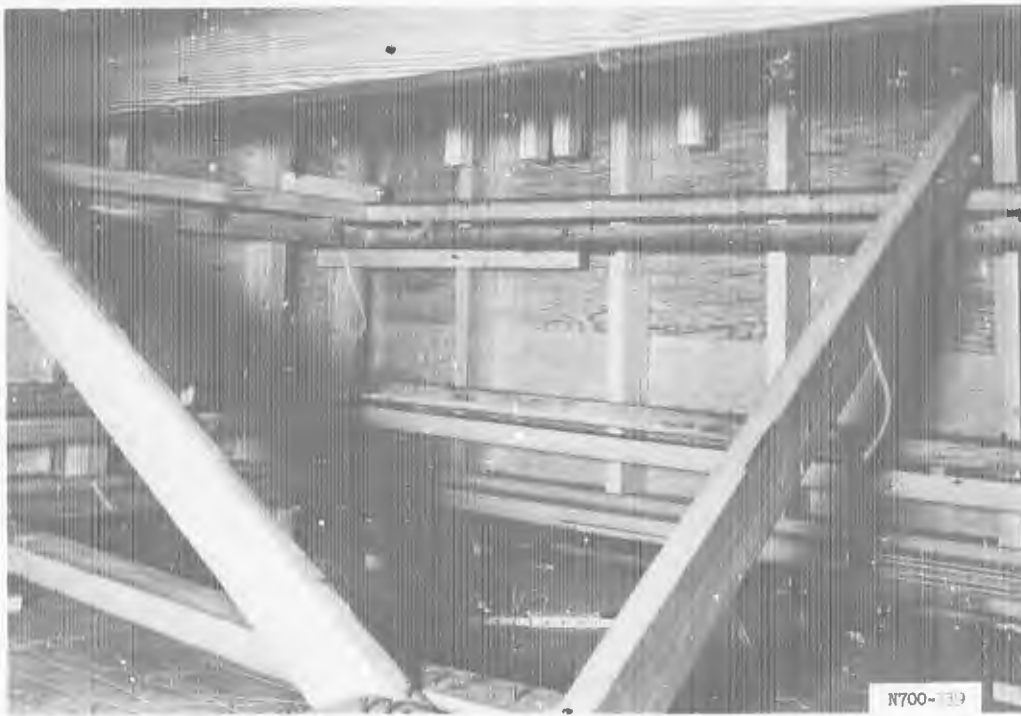


a. Overall view.



b. Closeup view.

Figure 3.11 Framing for the slab and drop panel forms.



a. Corner area.



b. Column area.

Figure 3.12 Interior of the inner model formwork.

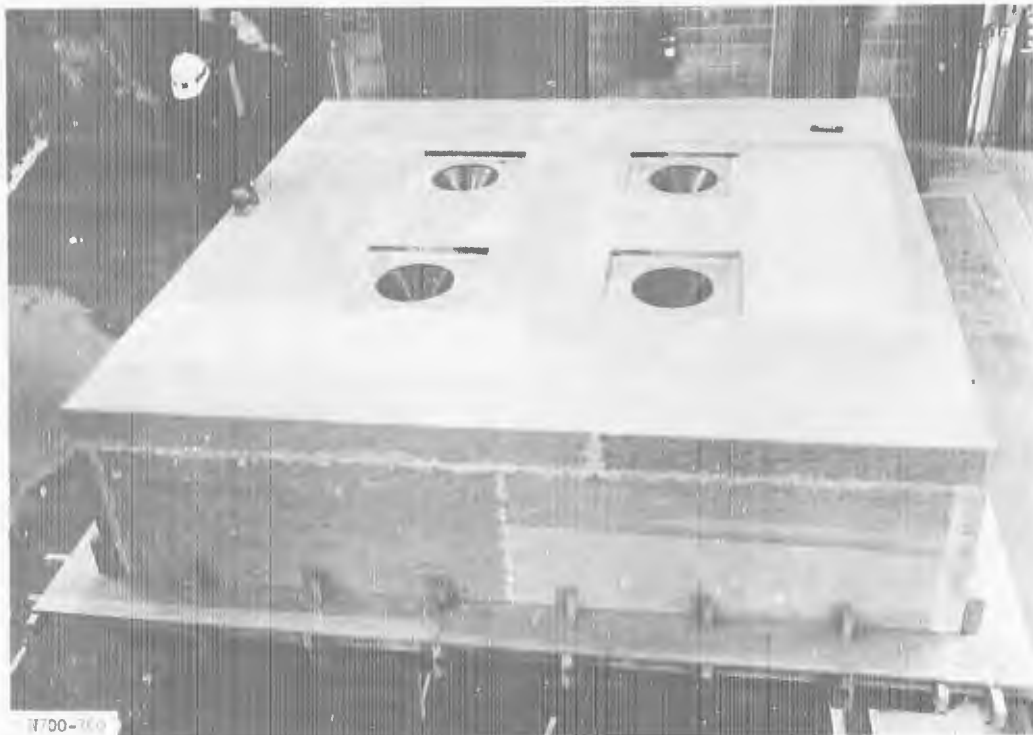


Figure 3.13 Complete inner formwork.

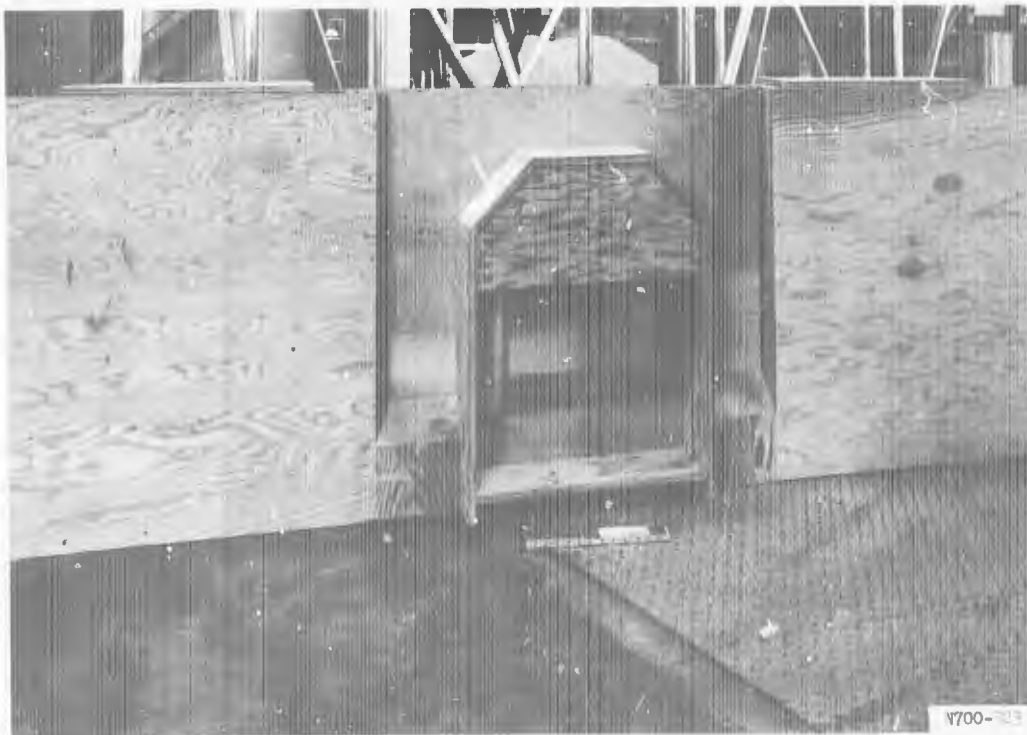


Figure 3.14 Outer wall formwork for the side containing access opening.

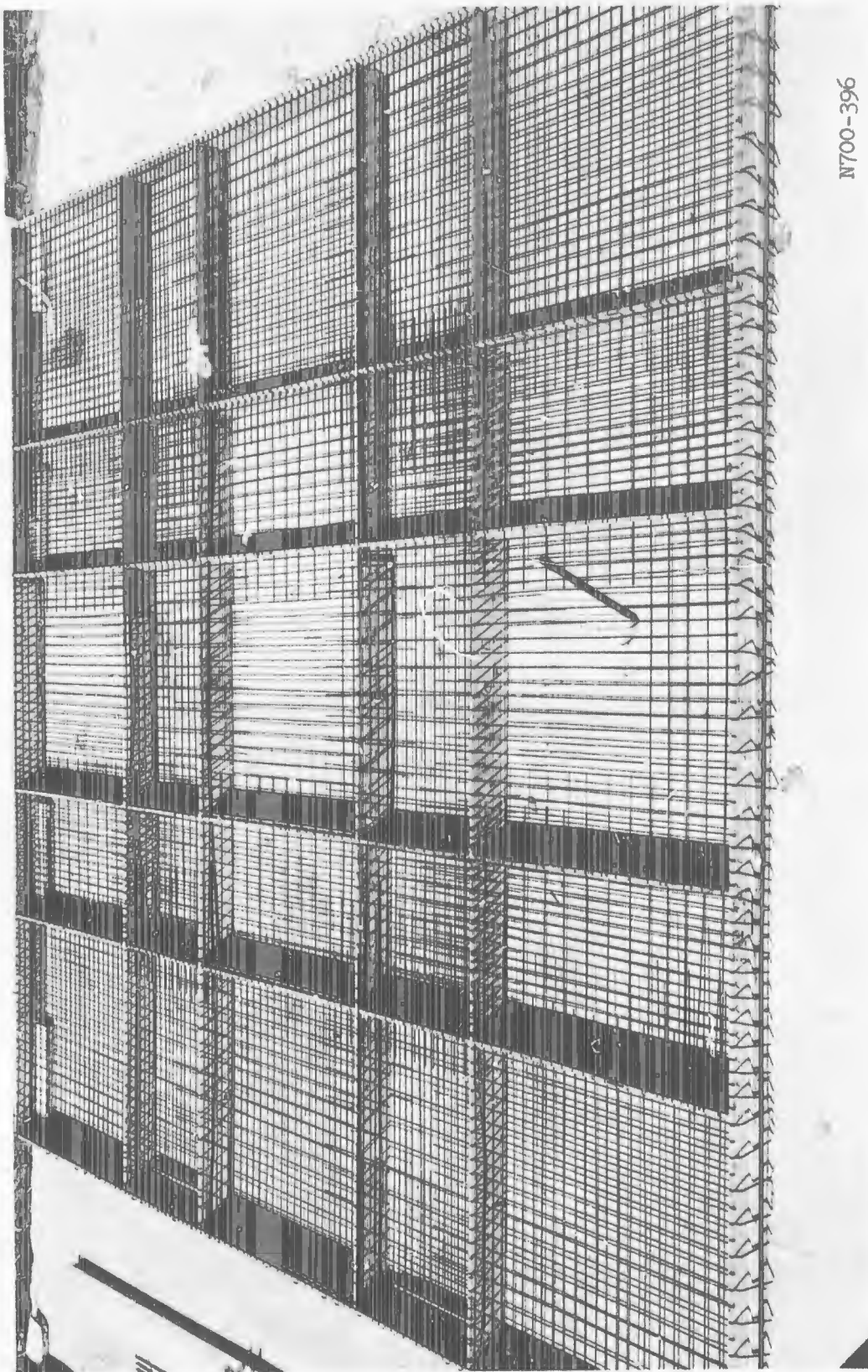
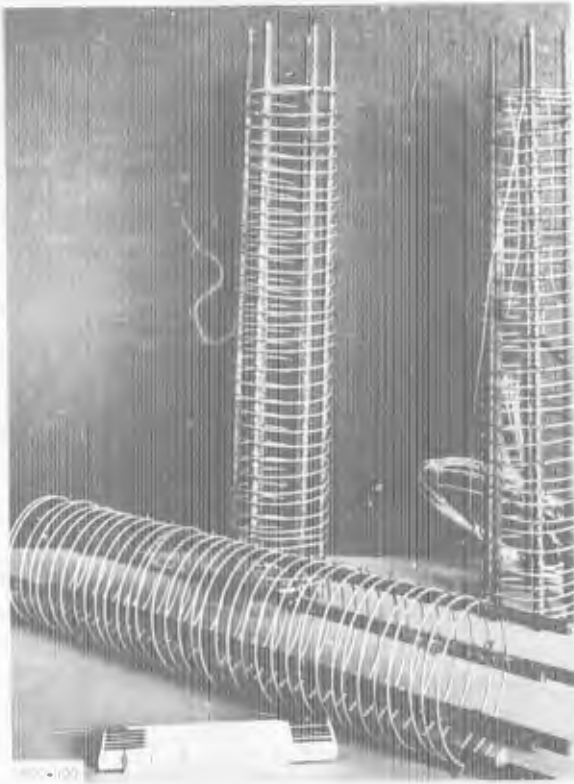


Figure 3.15 Jig for assembling the bottom reinforcement mat with most bars in place.



a. Assembled.



b. In-place.

Fig. 3.16 Column reinforcement.

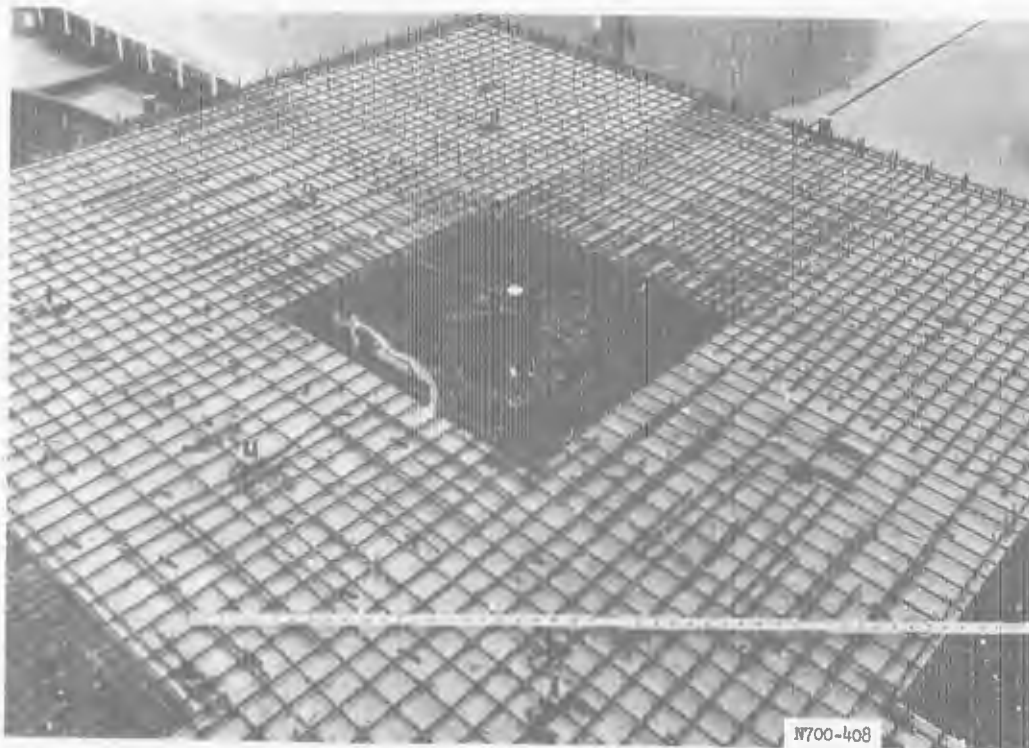


Figure 3.17 Closeup of bottom reinforcement mat in place.

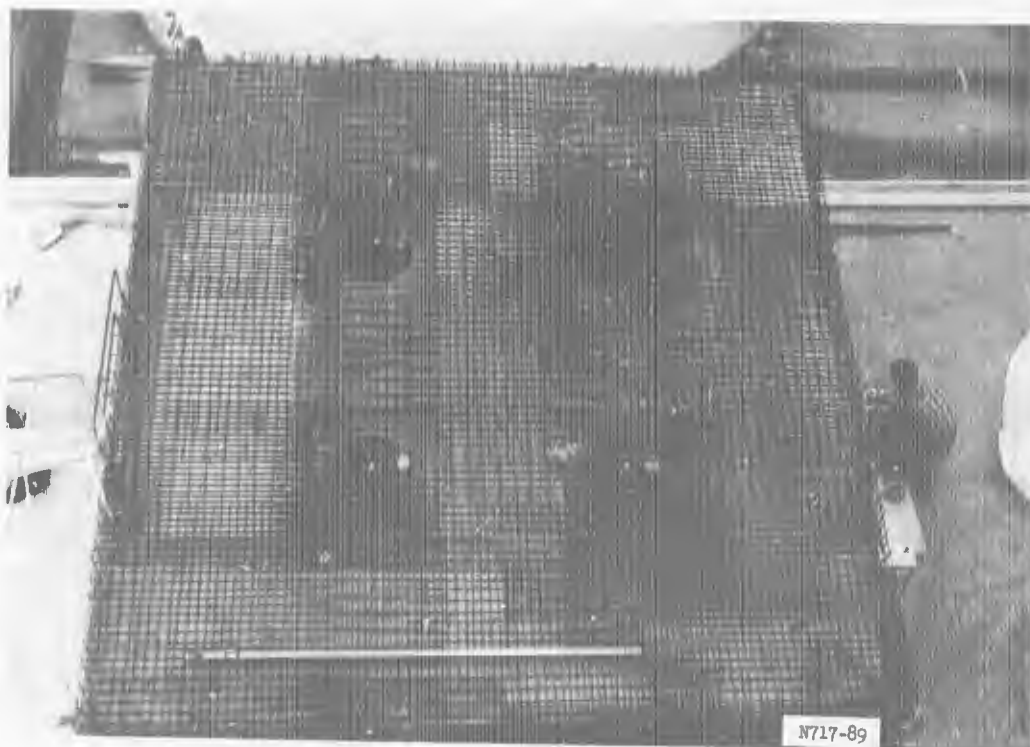


Figure 3.18 Bottom reinforcement mat in place.

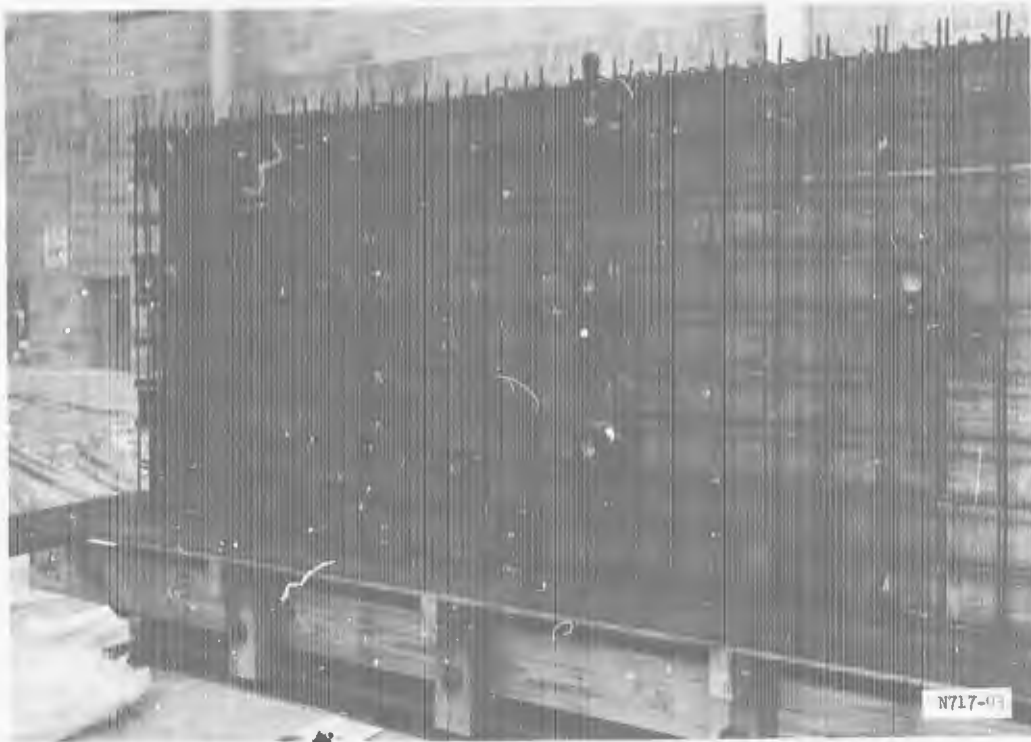


Figure 3.19 Wall reinforcement and soil pressure gage blockouts (model for static tests).

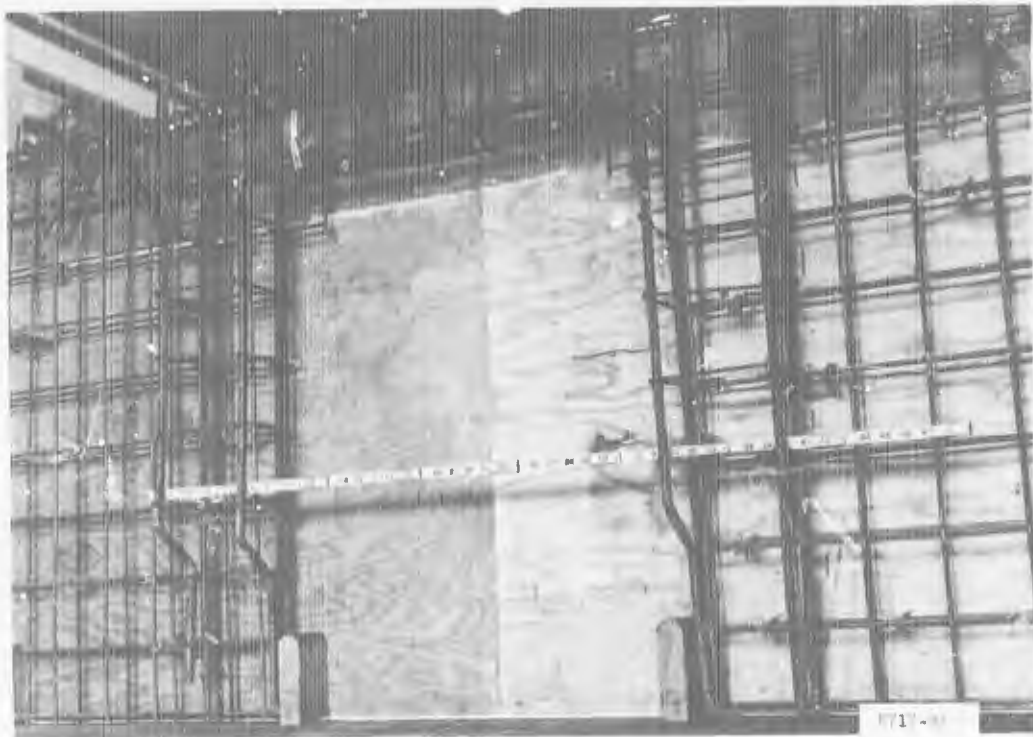


Figure 3.20 Reinforcement in wall and around side entrance.

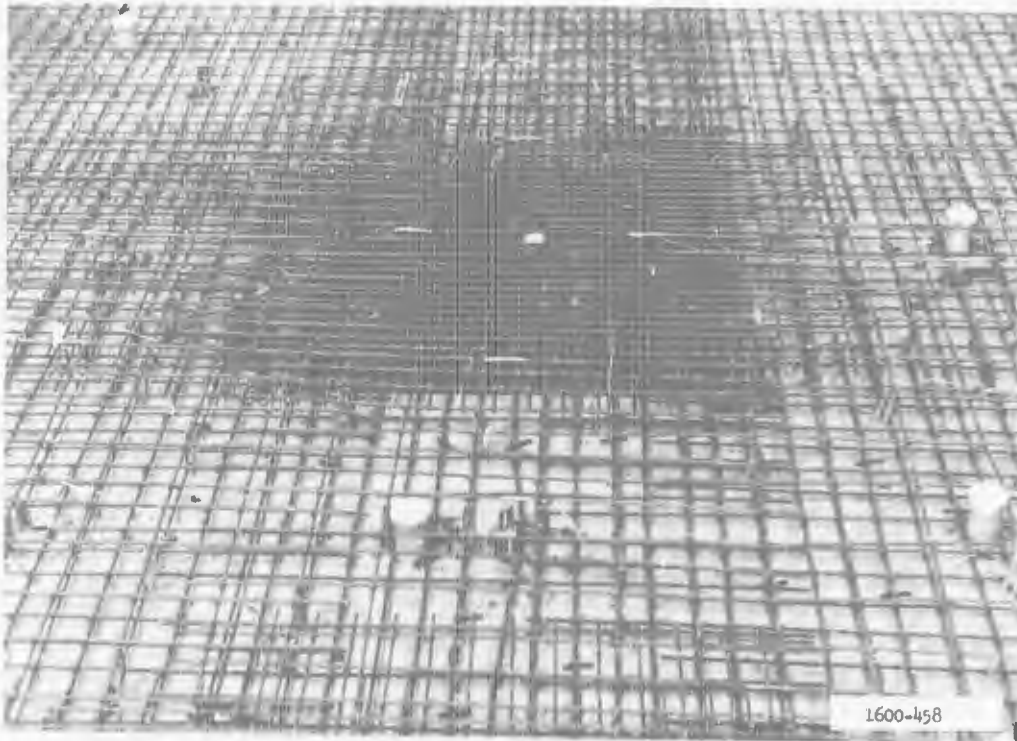


Figure 3.21 Column area with top mat in place (model for dynamic tests).

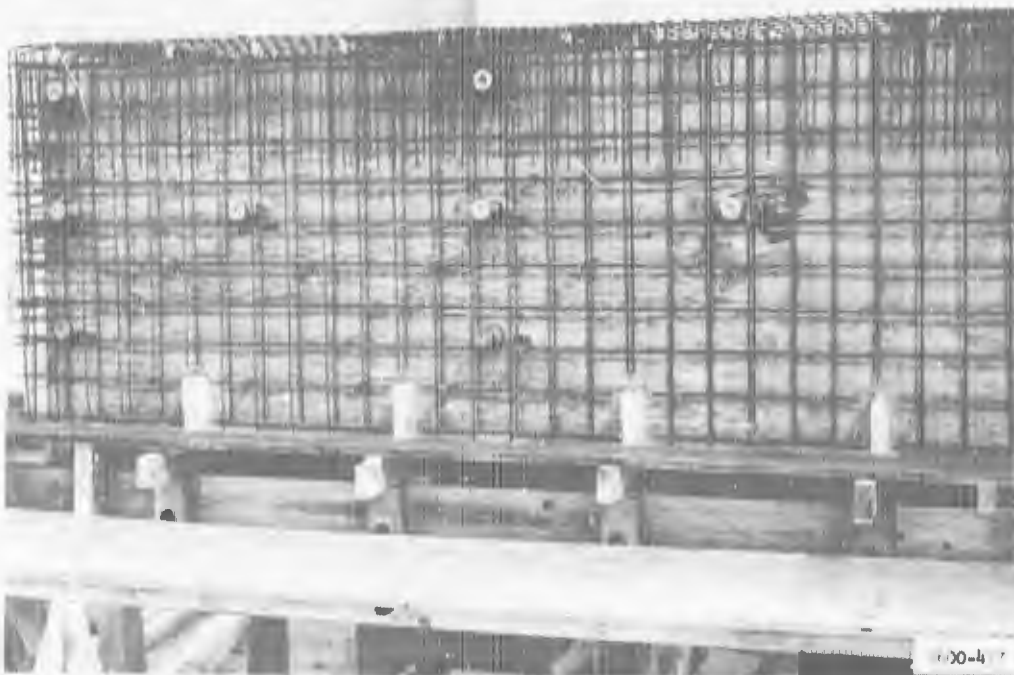


Figure 3.22 Wall reinforcement with hooks from top mat tied.

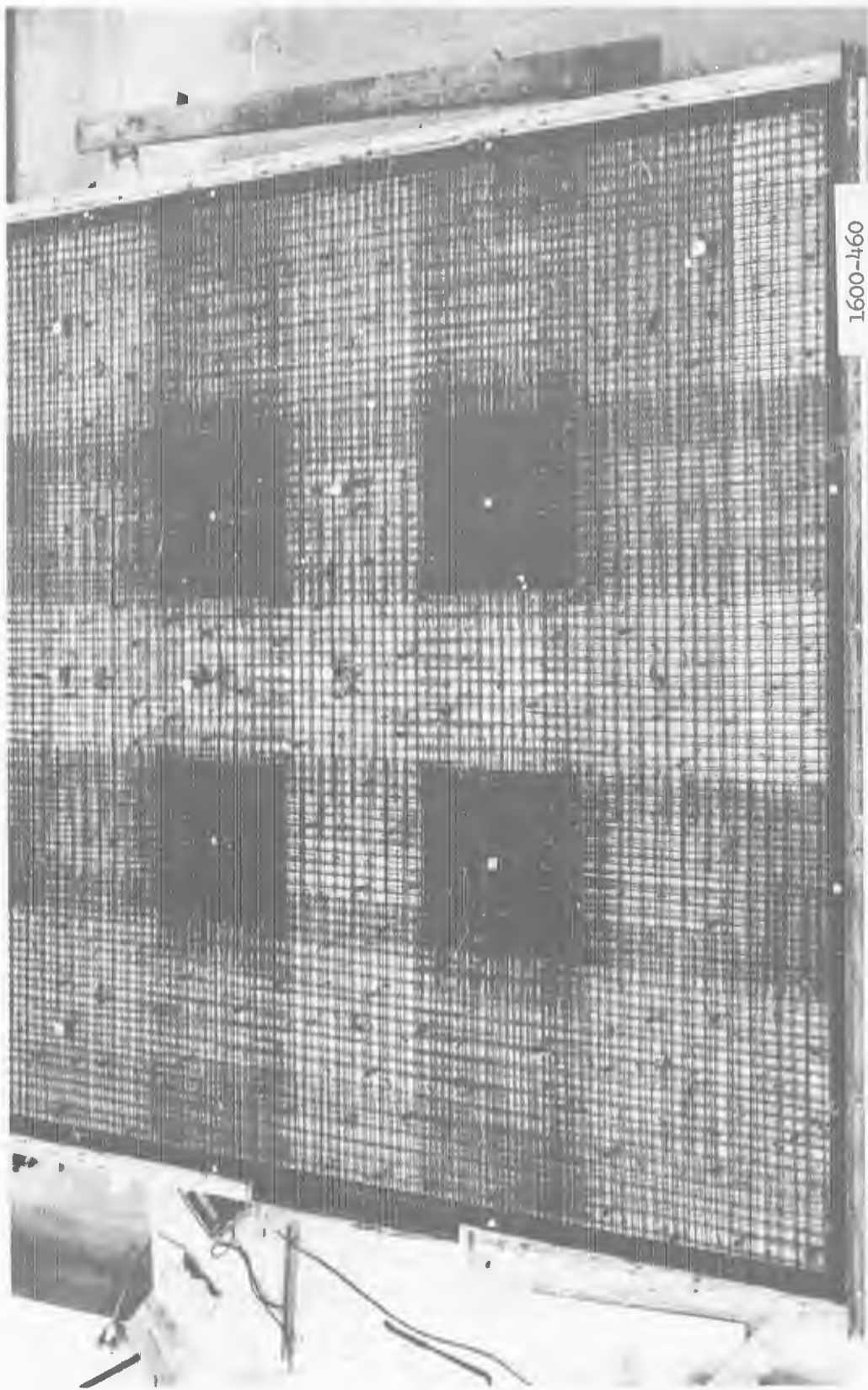


Figure 3.23 Completed slab reinforcement.

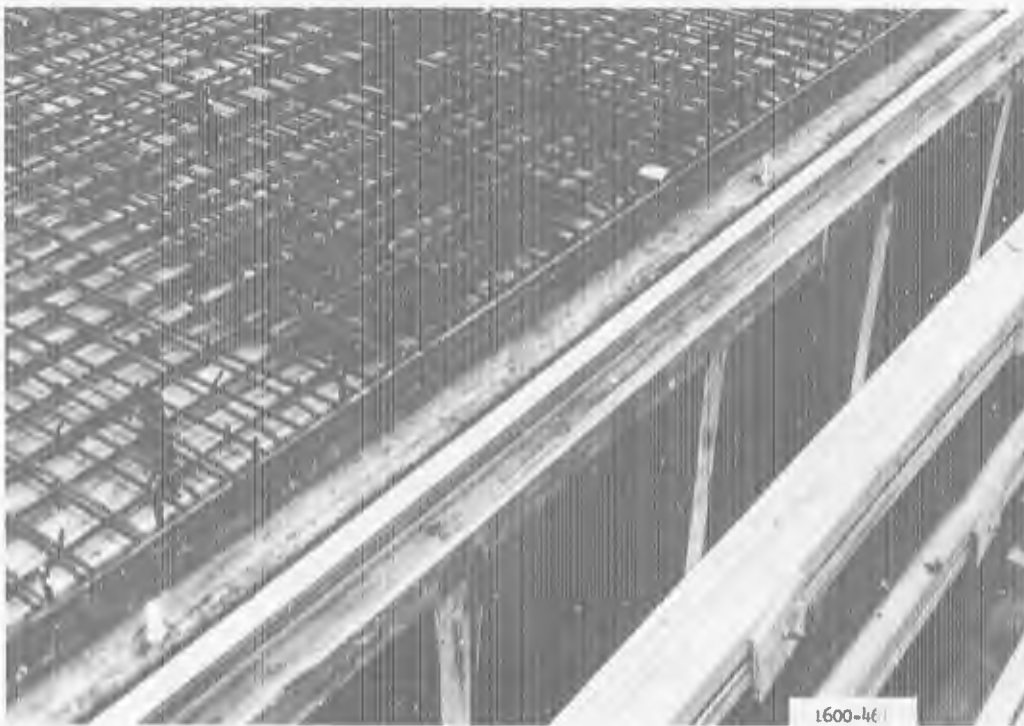


Figure 3.24 Detail of edge of form and reinforcement with screed support angle in place.

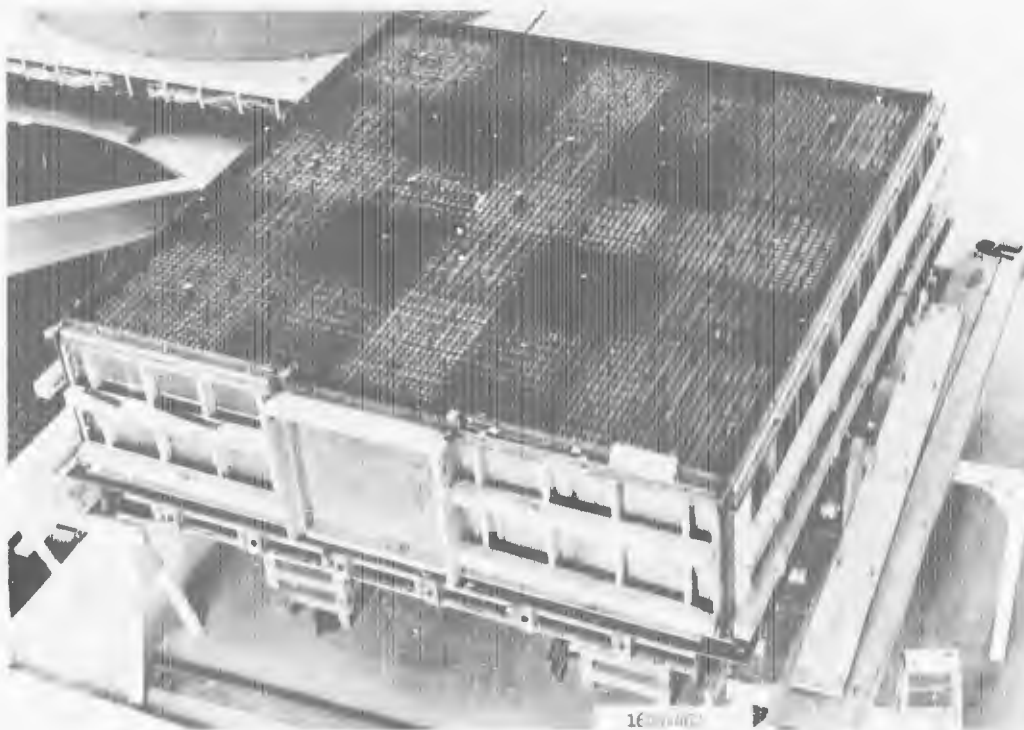
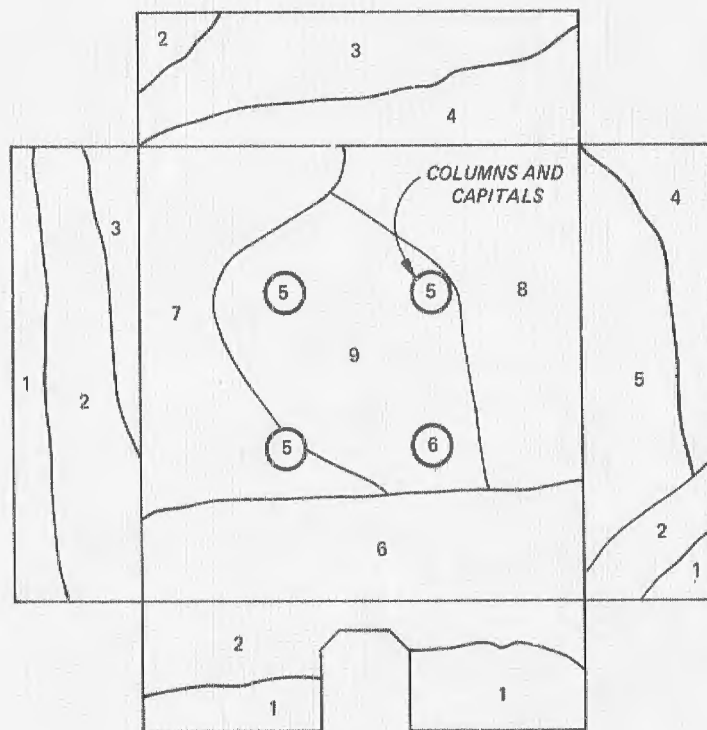


Figure 3.25 Overall view of slab reinforcement and formwork.

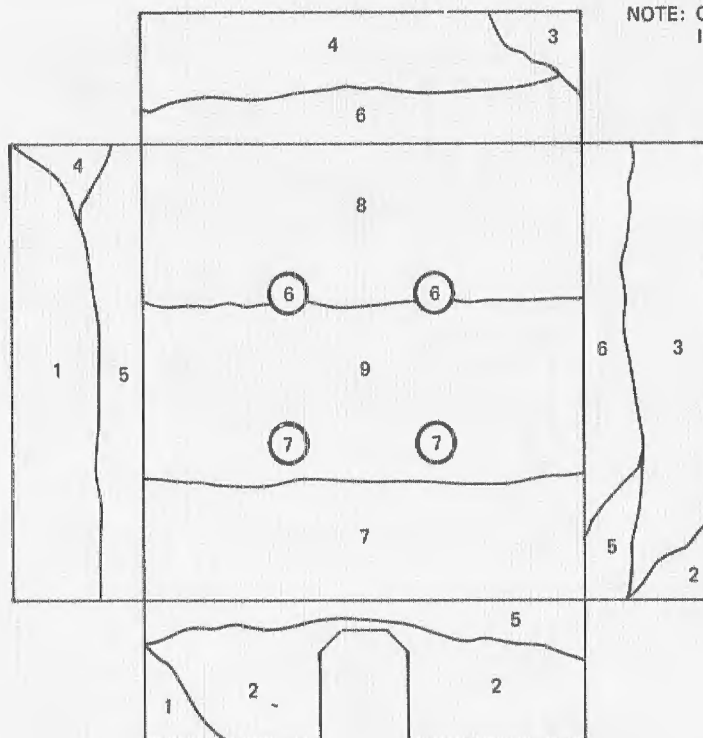


Figure 3.26 Slab model and formwork ready for casting of concrete.



a. MODEL FOR STATIC TESTING

NOTE: CONCRETE BATCHES PLACED
IN NUMERICAL ORDER



b. MODEL FOR DYNAMIC TESTING

Figure 3.27 Placement sequences and approximate locations of concrete batches in models.

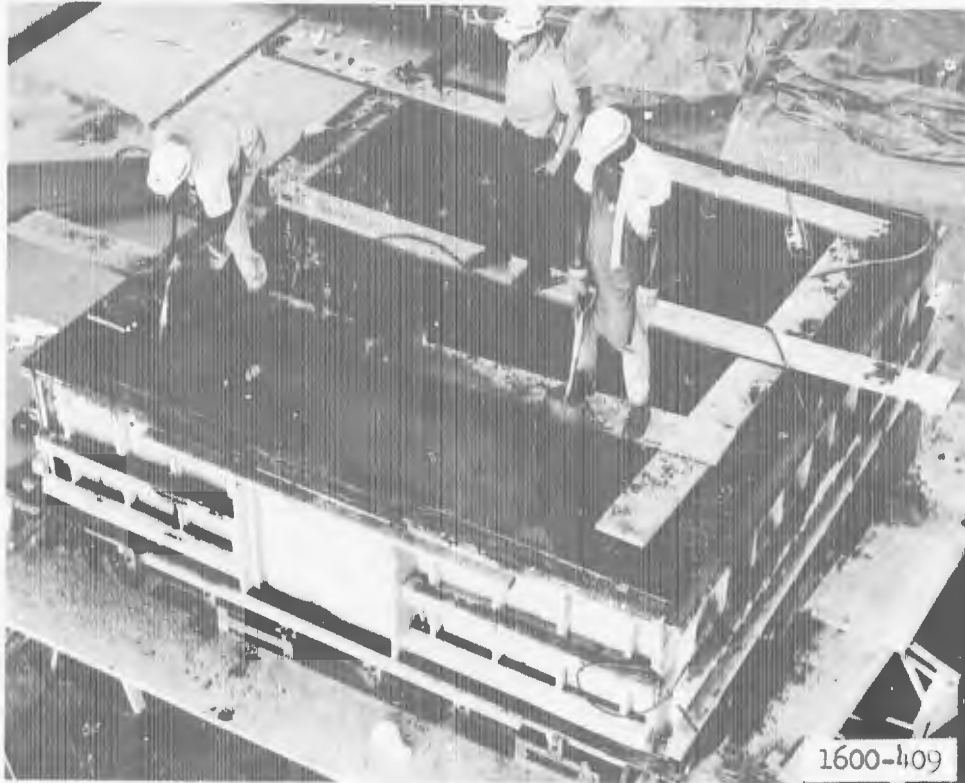


Figure 3.28 Concreting of the model structure.



Figure 3.29 Interior of the first (static) model structure looking toward the most heavily instrumented corner.



Figure 3.30 Bottom view of the first (static) model structure.

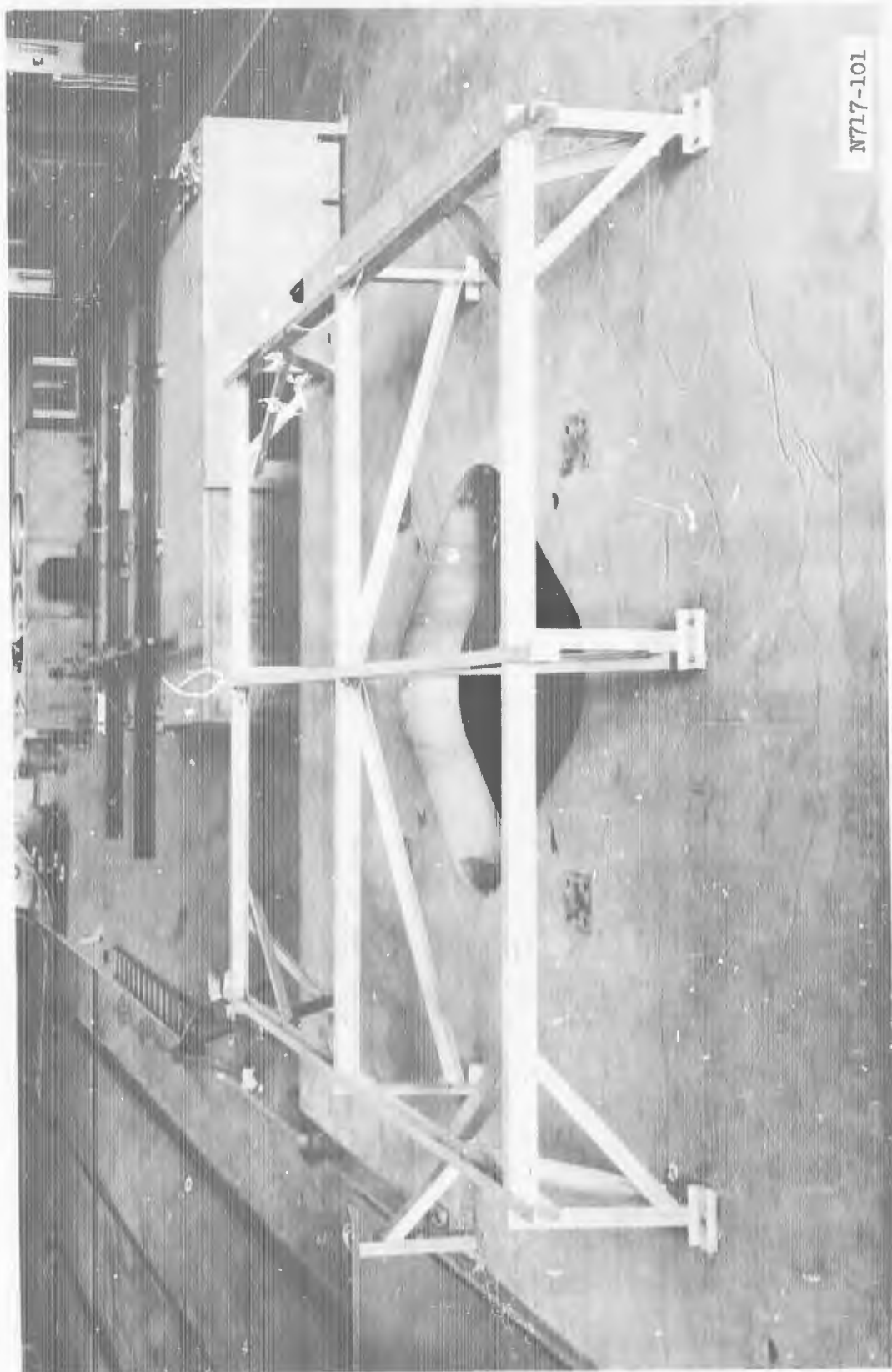


Figure 3.31 Deflection frame on base slab (model structures in background).

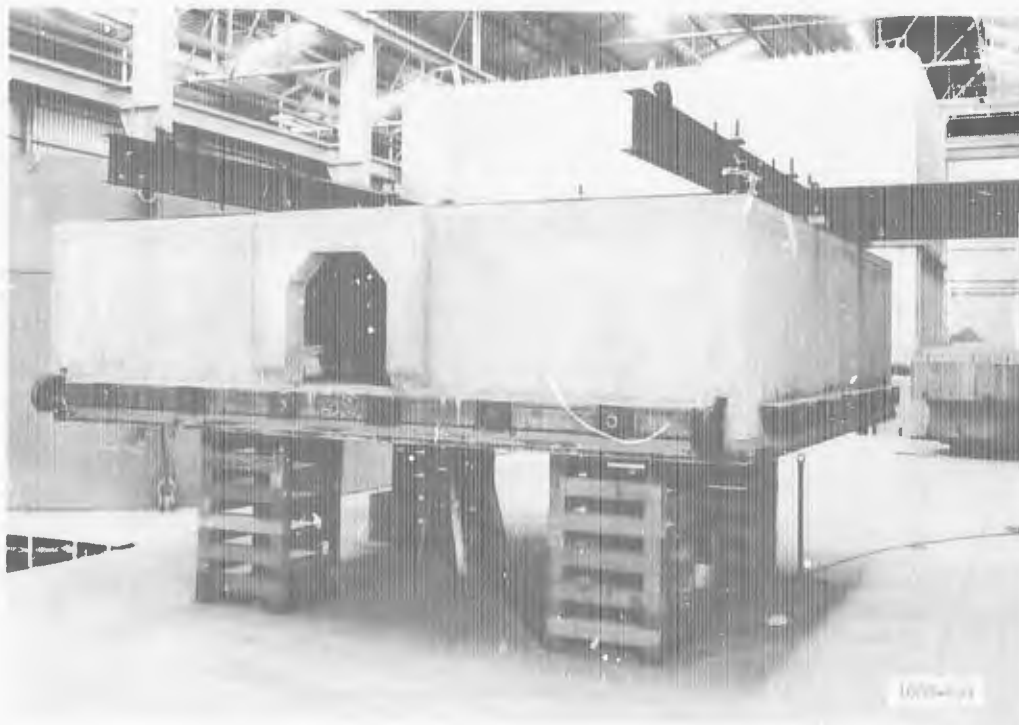


Figure 3.32 Model structure attached to the base slab.

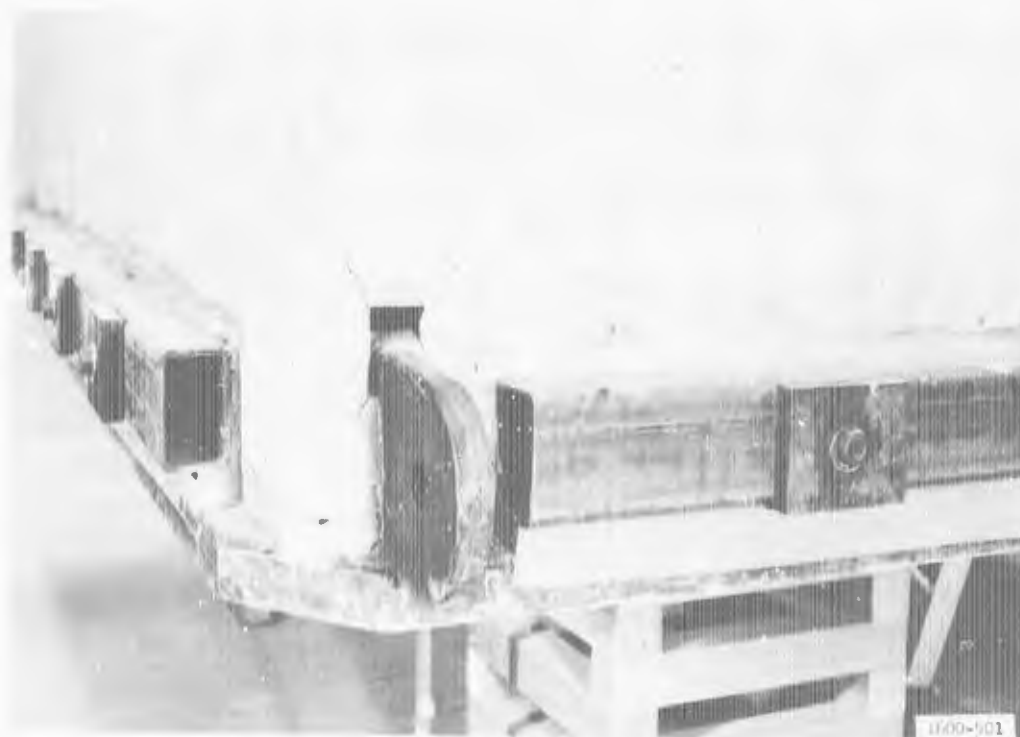
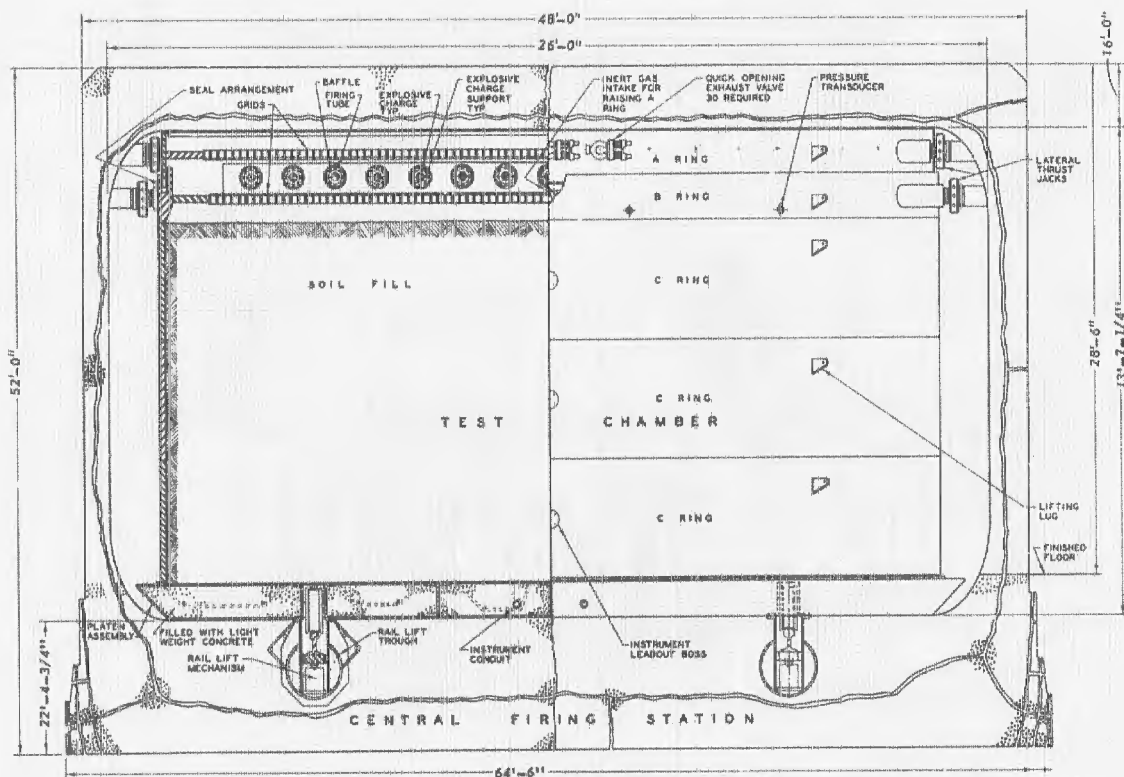
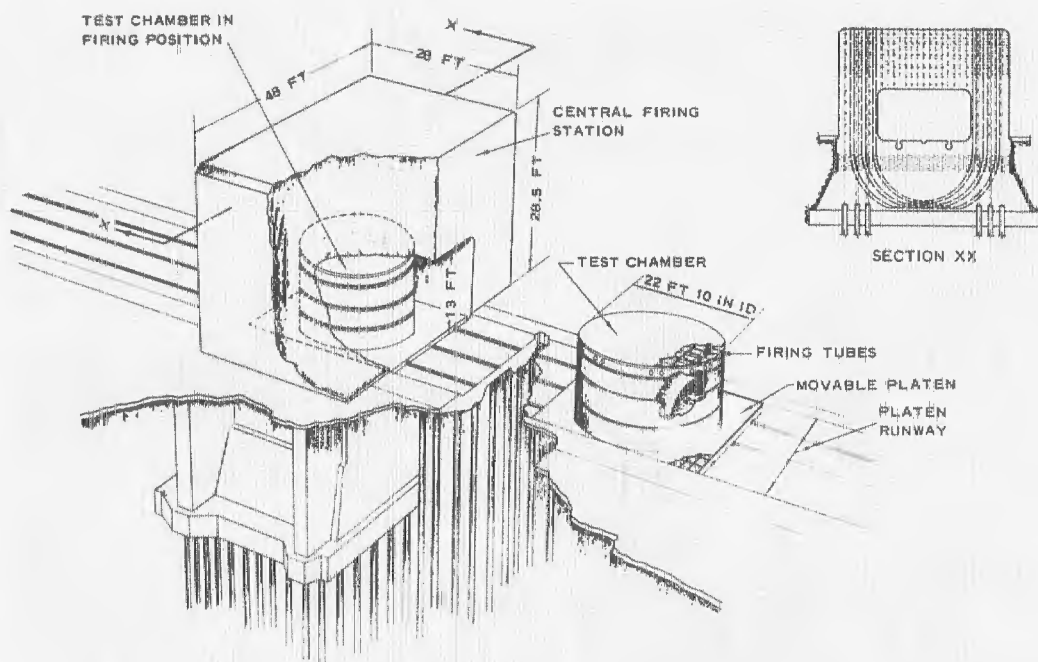


Figure 3.33 Corner of the model structure when attached to the base slab.

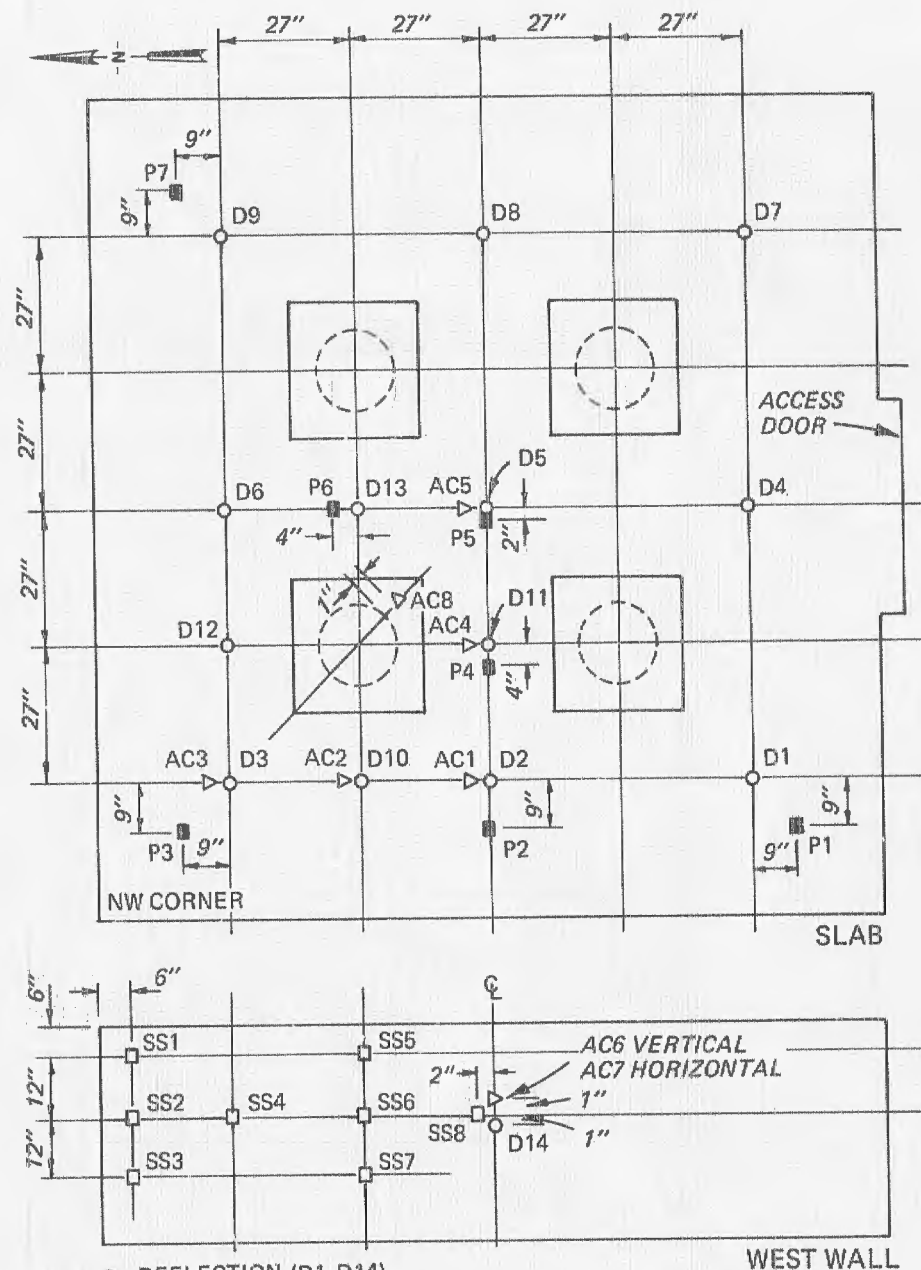


a. Half-section of the Large Blast Load Generator.



b. Cutaway view of the Large Blast Load Generator.

Figure 3.34 Large Blast Load Generator.



- DEFLECTION (D1-D14)
- ▷ ACCELERATION (AC1-AC9) - DYNAMIC TESTS ONLY
- SURFACE PRESSURE (P1-P9) - DYNAMIC TESTS ONLY
- SOIL PRESSURE (SS1-SS8)
- GAGES NOT SHOWN FOR DYNAMIC TESTS ONLY
- P8 - PLACED ON SAND SURFACE AT THE STRUCTURE CENTER IN BASE SLAB HOLE
- P9 - PLACED IN BASE SLAB IN BLOCKOUT (SEE FIG. 3.5) INSIDE NE COLUMN LOCATION
- AC9 - PLACED ON LOWER PLATE OF LOAD CELL UNDER NW COLUMN

Figure 3.35 Locations of deflection and pressure gages and accelerometers.

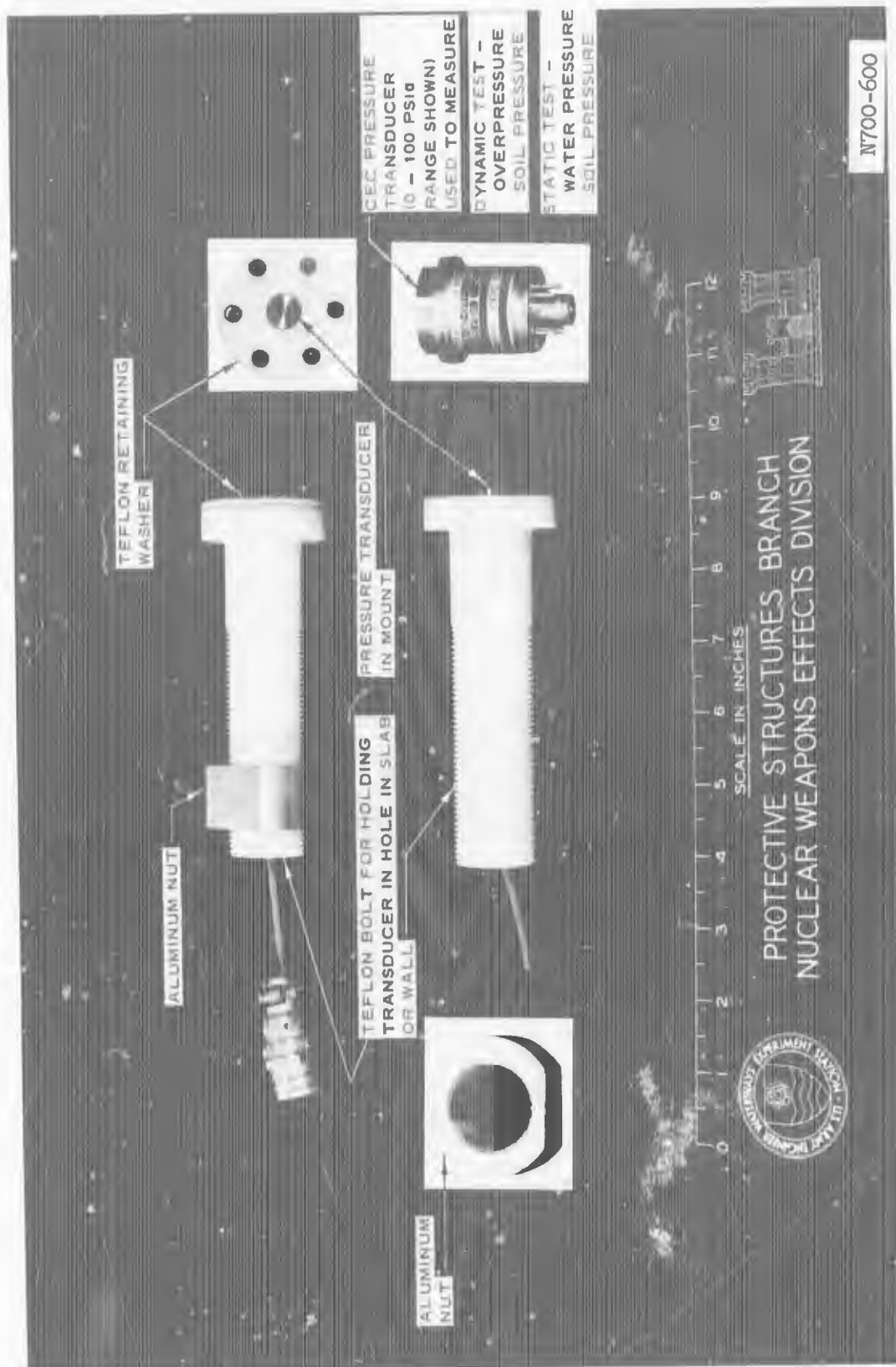


Figure 3.36 Pressure transducers.

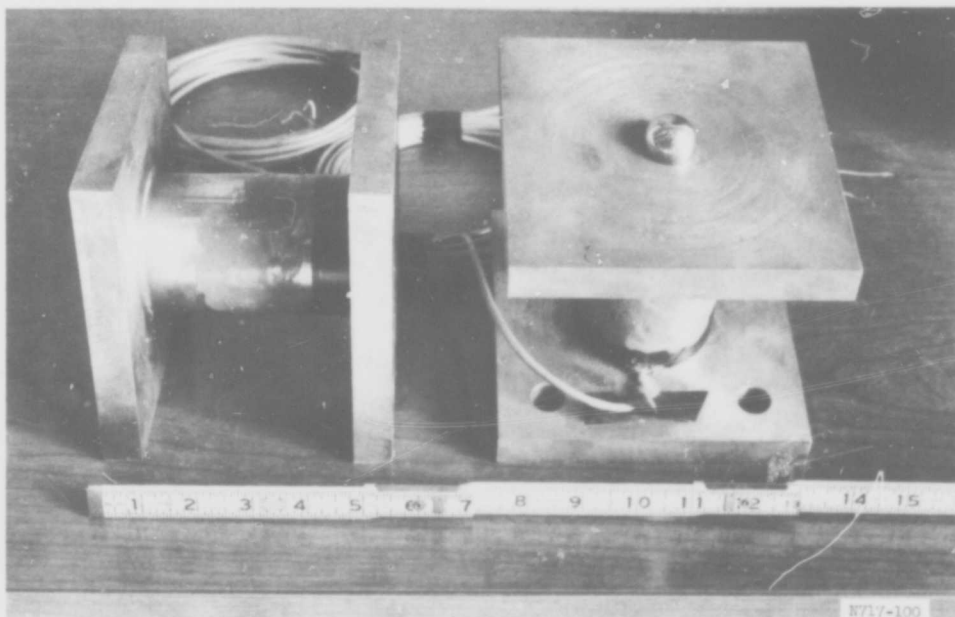
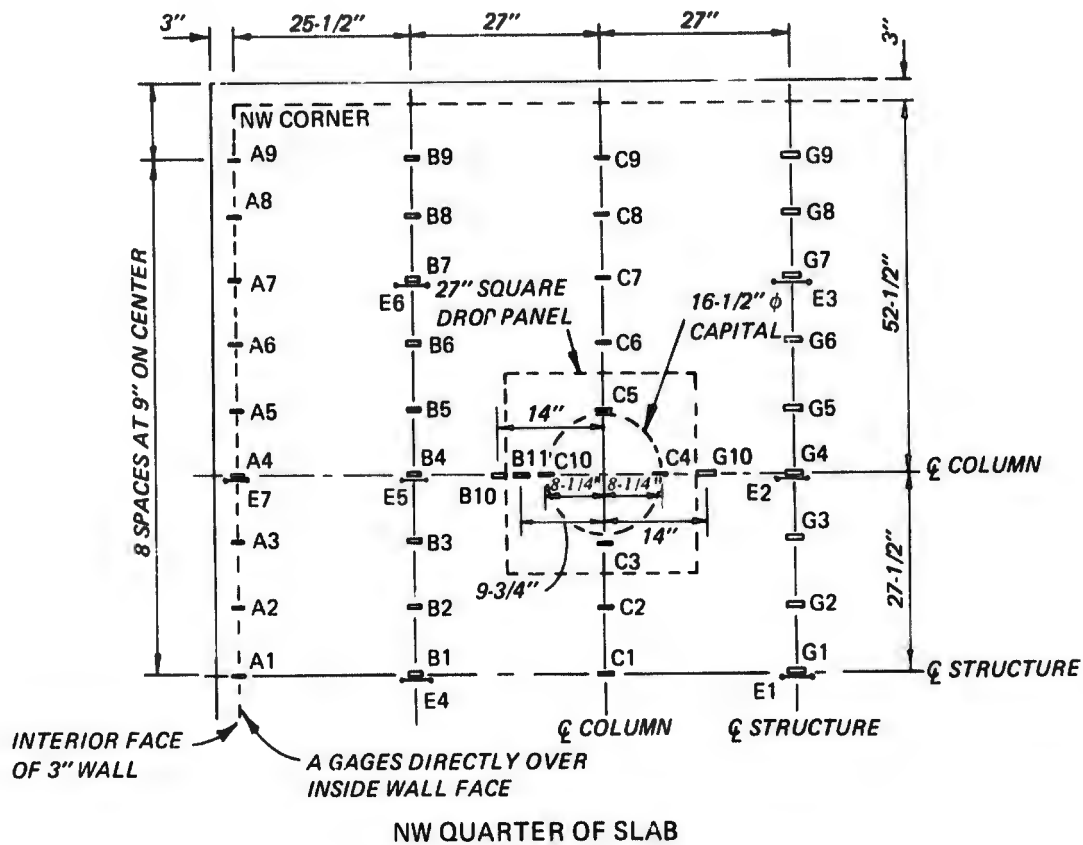


Figure 3.37 Load cells for determining column loads.



Figure 3.38 Acceleration and deflection transducers.



- A1-A9 STRAIN GAGES ON TENSILE REINFORCEMENT; TOP STEEL
- ▬ B1-B11 STRAIN GAGES ON TENSILE REINFORCEMENT; BOTTOM STEEL
- E1-E7 STRAIN GAGES ON COMPRESSIVE REINFORCEMENT

NOTE: STRAIN GAGES PLACED ON REINFORCEMENT BAR CLOSEST IN NORTH-SOUTH DIRECTION TO THE LOCATIONS SHOWN.

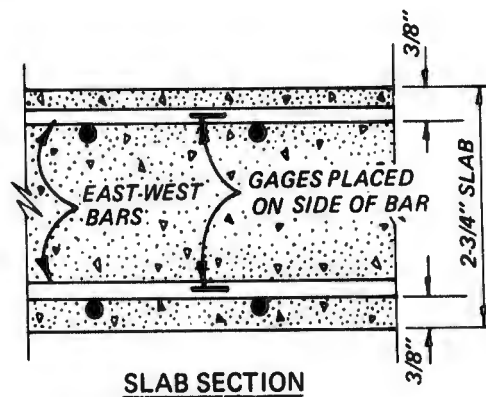
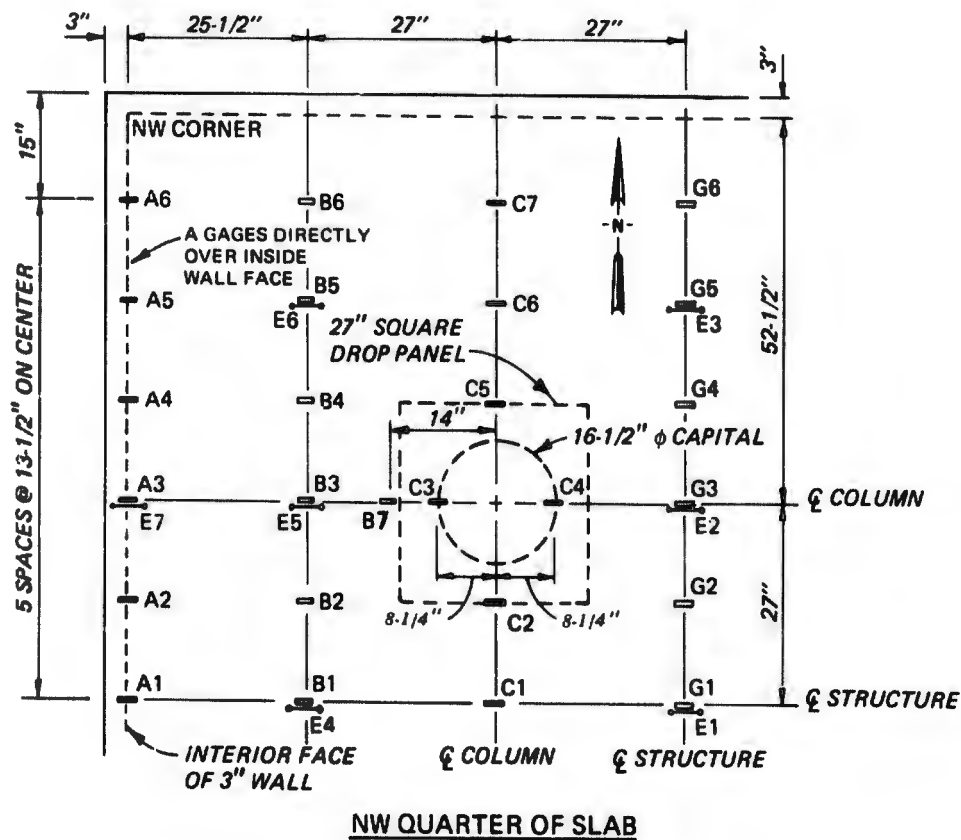


Figure 3.39 Locations of strain gages on reinforcement (static test structure).



- { A1-A6 STRAIN GAGES ON TENSILE
C1-C7 REINFORCEMENT; TOP STEEL
- { B1-B7 STRAIN GAGES ON TENSILE
G1-G6 REINFORCEMENT; BOTTOM STEEL
- E1-E7 STRAIN GAGES ON COMPRESSIVE
REINFORCEMENT

NOTE: STRAIN GAGES PLACED ON REINFORCEMENT BAR CLOSEST IN NORTH-SOUTH DIRECTION TO THE LOCATIONS SHOWN.

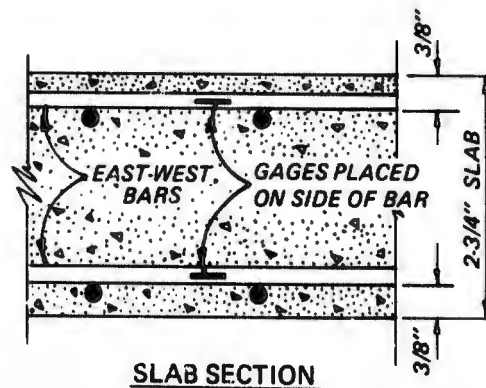
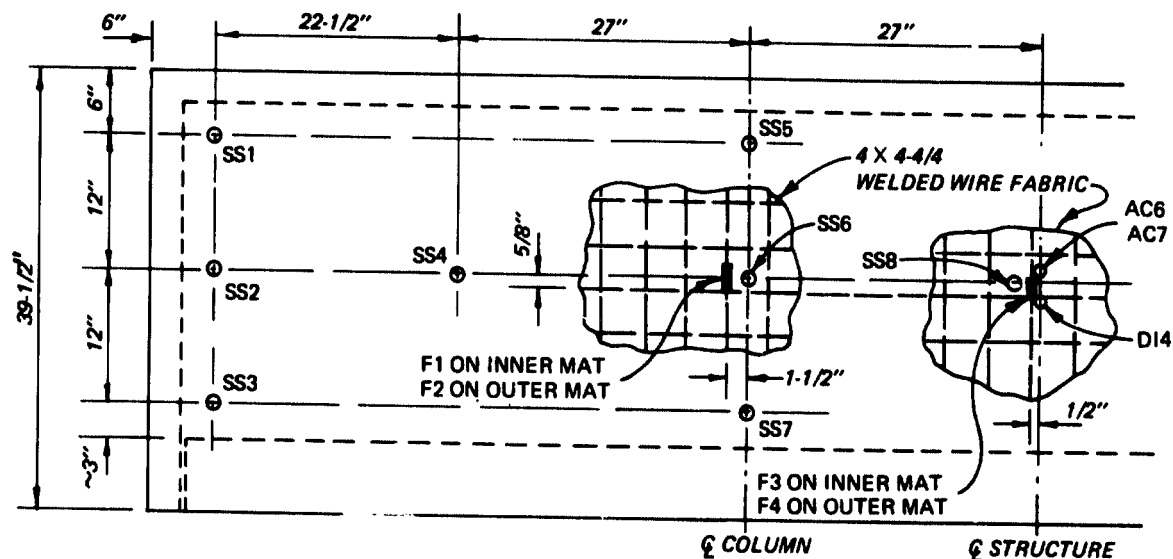
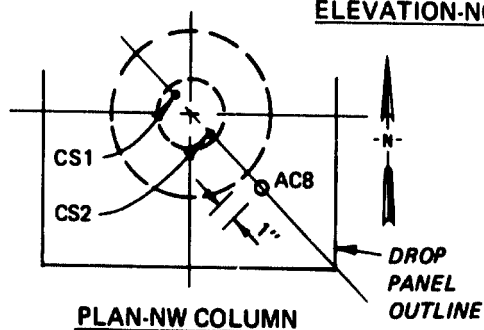


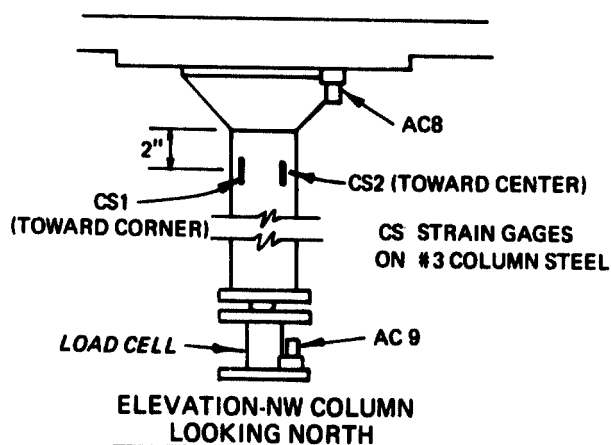
Figure 3.40 Locations of strain gages on reinforcement (dynamic test structure).



ELEVATION-NORTH HALF OF WEST WALL

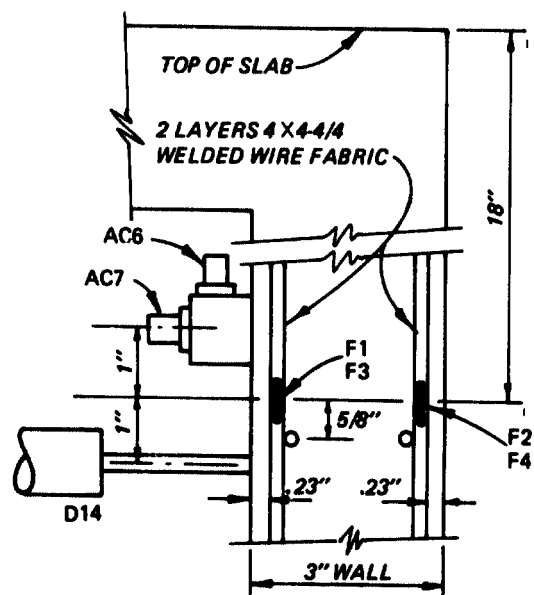


PLAN-NW COLUMN



ELEVATION-NW COLUMN
LOOKING NORTH

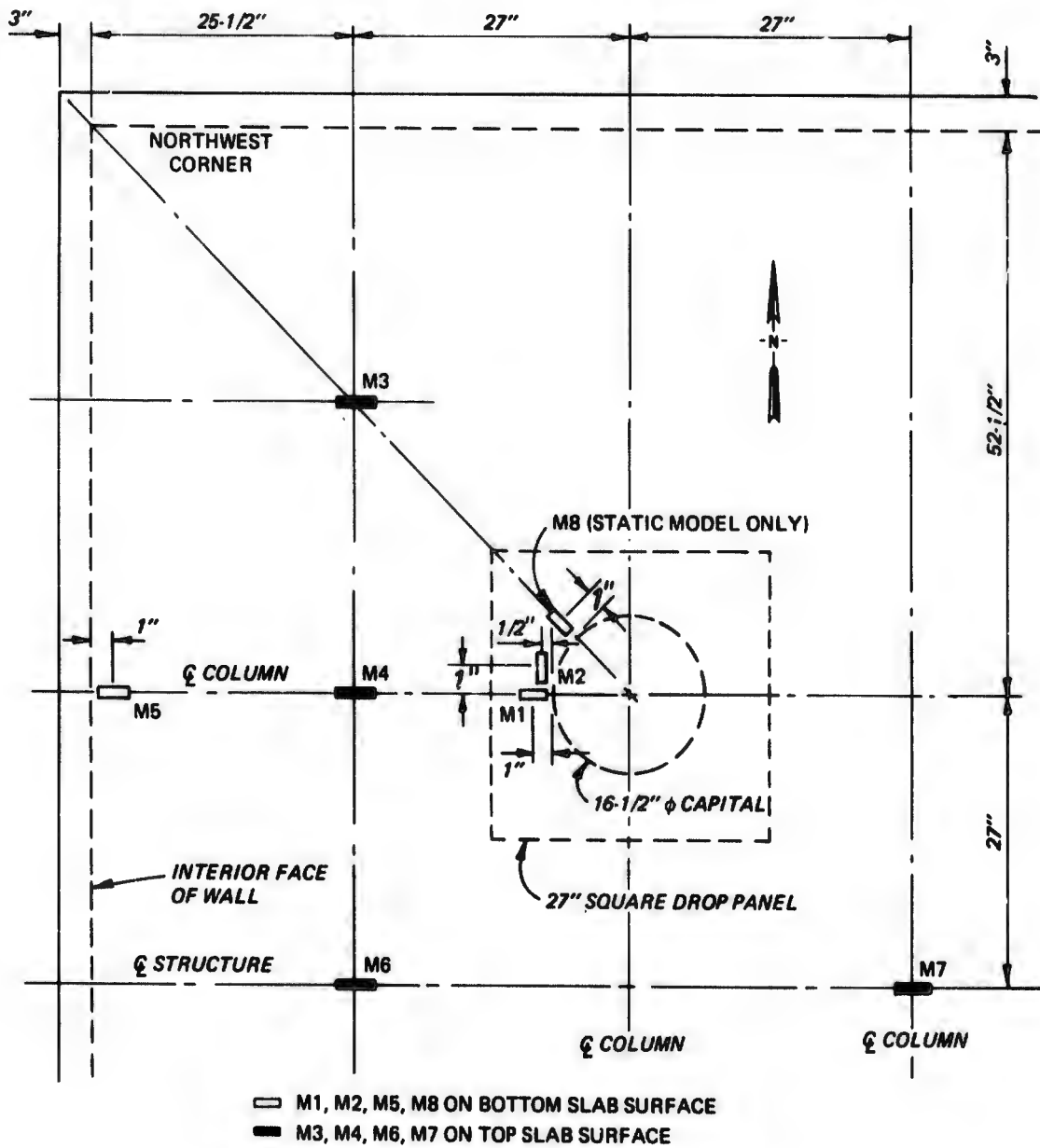
F1-F4 STEEL STRAIN-WALL
CS1-CS2 STEEL STRAIN-COLUMN
D14 DEFLECTION
AC8-AC9 ACCELEROMETER
SS1-SS9 SOIL PRESSURES



NOTE: F-SERIES GAGES MOUNTED
ON SIDE OF BAR
SS7 LOCATED 2" NORTH
OF STRUCTURE $\bar{C}$
F3 AND F4 1/2" NORTH
OF STRUCTURE $\bar{C}$

SECTION OF WALL

Figure 3.41 Locations of strain gages and assorted other transducers not on the slab.



NW QUARTER OF SLAB

Figure 3.42 Locations of strain gages on the concrete surface.

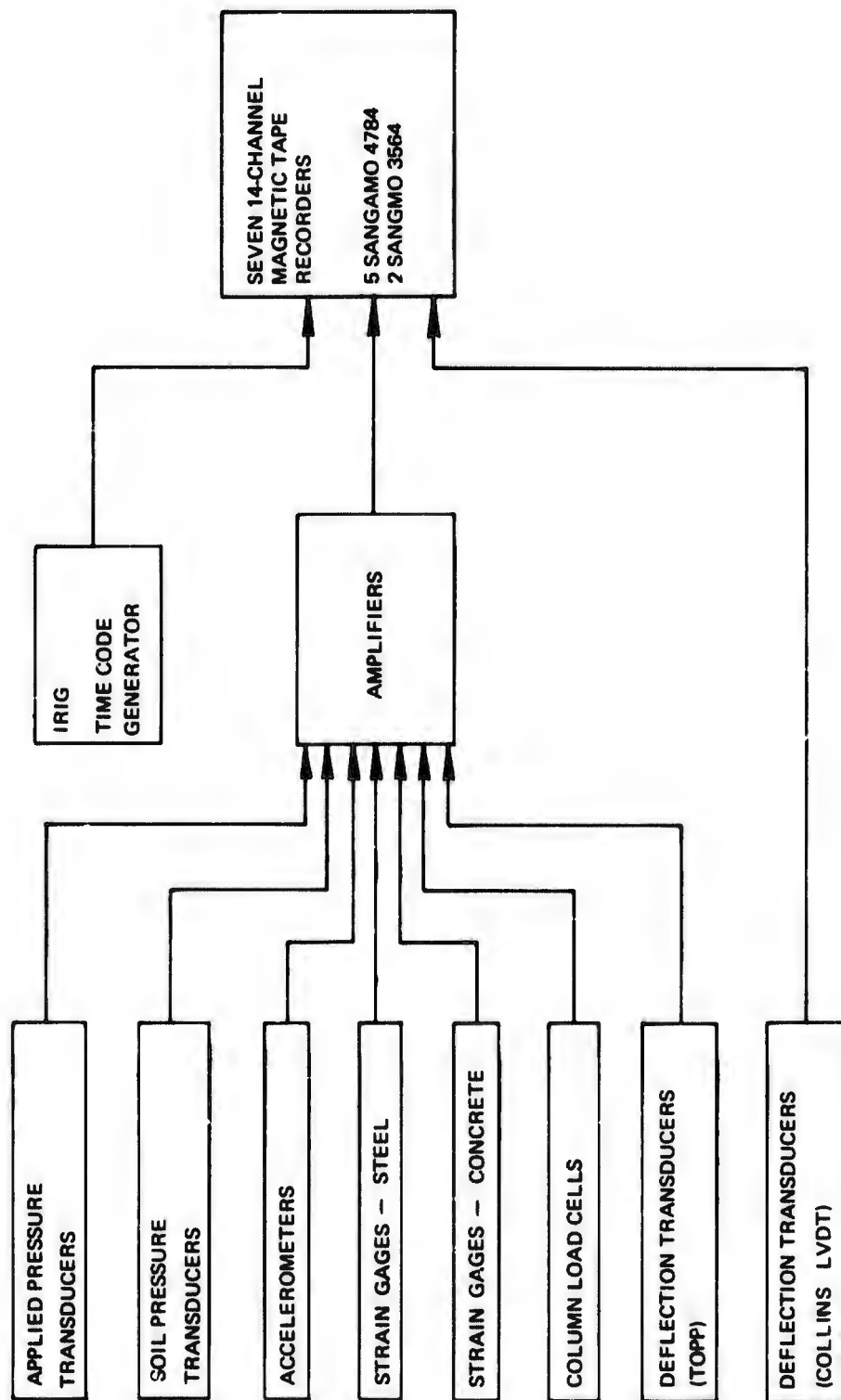


Figure 3.43 Diagram of instrumentation system.

CHAPTER 4

EXPERIMENTAL RESULTS

The results of the static and dynamic tests are presented in this chapter and in Appendixes A and B. A general description of the performance of each structure, the characteristics of the loads placed on the slabs, and the performance of the instrumentation system and data produced from each test are presented herein. Further discussion and analyses of the results are presented in Chapters 5 and 6.

4.1 STATIC TEST

The first model was loaded to failure during a test of approximately 19-minutes duration. Recorded data from this test are presented in Appendix A.

4.1.1 Performance of the Statically Tested Structure. The pressure on the structure was increased continually until the peak pressure supported by the slab reached a value of 26.6 psi. The pressure then suddenly dropped to a value of 15 to 17 psi, which was maintained until the test was terminated when the sealing membranes over the slab surface ruptured allowing water to enter the interior of the structure and the sand backfill material in the testing chamber.

The slab model as it appeared after removal of the sealing membranes is shown in Figure 4.1. The slab reached large deflections in all panels and displayed considerable distress around the column areas, although it did not drop from the columns. The rupture of the slab along the north row of panels, resulting in seal failure, led to termination of the test. This slab rupture extended through two areas where the main tensile and compressive reinforcement bars were spliced near the exterior span side of each column (see Figures 2.16 and 2.17). The bars in the rupture zone were pulled out of the concrete and did not break in tension.

Lines of crushed concrete indicating the formation of flexural yield lines, particularly along the north and east rows of panels and the northeast corner, are shown in Figure 4.2, a top view of the slab

surface. The curvatures of the west, east, and especially the north boundaries of the slab visible in this photograph reflect the pulling in of the tops of the walls by the slab late in the test.

The magnitude of the slab deflections along with the size of the slab rupture can be seen better in Figure 4.3. The slab appears to be draped over the columns with much cracking, and slopes on the order of 30 to 45 degrees are evident in a 7- to 10-inch wide ring around each column capital. Except for this cracking and a concentrated line of cracks over the inside wall location, the top slab surface away from the rupture zone was comparatively free of major cracks.

The tops of the two north columns, those adjacent to the slab rupture zone, were pulled toward the center of the structure by the centers of panels that remained attached to these columns. This is shown clearly in Figure 4.4. The cracking in one corner of the slab and along parts of two walls is also shown in Figure 4.4b. A number of top bars failed in tension at the crack over the north wall. These were the only broken bars found in the structure. The rotation of the slab at the wall line appeared to be sufficient to crush the concrete on the bottom side of the slab and destroy the capacity of the slab to transfer shear in the usual way at this section. In several locations, the slab appeared to be largely suspended from the wall by the reinforcement.

The deflections of the slab surface measured along nine lines after completion of the static test are displayed in Figure 4.5. The permanent deflection of the center panel, which was located away from the rupture area, was approximately 10 inches (18 percent of the column spacing). Deflections along exterior panel centerlines away from the rupture area (rows 1 and 6 in Figure 4.5) were 6 to 7 inches (11 to 13 percent of the column spacing) over the center three-fourths of the slab. Thus, the slab displayed a sizable ductility, although, as will be discussed in Chapter 6, much of this deflection capacity is not usable for resisting dynamic loadings.

Figure 4.6 presents a view of the deflected roof slab and the interior of the shelter area as it would appear to a hypothetical scaled-down person standing on the shelter floor near the south wall opening.

The framework supporting the deflection gages and the instrumentation has been removed. Considerable debris, some of fairly large size, is visible, especially in the vicinity of the columns. Some cracking and splitting are evident near the bases of the columns, indicating that the columns were heavily loaded.

Figure 4.7 shows the posttest condition of the interior of the model after it was removed from the base slab. Cracking resulting from positive moments can be seen along panel centerlines. The cracks generally follow the placement of the reinforcement and are quite wide along the east (right) row of panels, the location of a yield line. This yield-line location is also evident from the crushing of the concrete that occurred on the upper slab surface at the center of the east row of panels (Figure 4.2). Much distress and spalling of the concrete below the bottom reinforcement mat, including most of the drop panel area, is visible, along with some splitting of the column capital at the southeast (near right) column.

A closer view of the crack in the slab along the north wall is shown in Figure 4.8. This crack was over 1-1/2 inches wide in places and is located at the supported end of a portion of slab extending out to the line of rupture and which folded down into the structure.

The cracking of the bottom surface of the south half of the middle panel and beyond is shown in Figure 4.9. Extensive crushing of the concrete occurred between the two south columns (shown) and the two north columns.

The crack pattern on the bottom slab surface near the two north columns and details of the reinforcement near where the slab opened up are shown in Figure 4.10. Note that the drop panel around these two columns has completely fallen away and that the entire bottom reinforcement mat is exposed all around the column.

Figure 4.11 shows the condition of the four columns and load cells after the static test. The northeast column (Figure 4.11b) tilted more than any other column and jammed the plate on the column base against the load cell.

The column capital of the southeast column (Figure 4.12) split more

than any other capital. Two fairly large portions of the drop panel remained in place near this column, but appeared to be structurally useless and ready to drop to the floor. The bottom (compression) steel placed over the column area and some of the tensile steel extending from the positive moment regions resemble a net of suspension cables supported on the column capital in Figure 4.12.

The debris from the drop panel around the northeast column (Figure 4.13) included some sizable fragments. These might have been even larger if several sheet-metal chairs supporting the top reinforcement mat in the column area had not been extended through the drop panels. The drop panels seem to be appropriately named; these areas dropped from the structure. (The wording "drop panel" originated to describe the portions of the slab for which the lower panel surface drops or is located below that of the rest of the slab.)

The four walls of the structure after completion of the static test are shown in Figure 4.14. The cracks have been marked. The crack pattern is predominantly that expected for a rectangular plate fixed on three sides and loaded perpendicular to the plate (wall, in this case) along the long, unsupported side. The framing around the south wall entrance and the rupture of the slab far from this wall both contributed to the lack of extensive cracking in the south wall. The widest crack in each of the other three walls was located above the perimeter beam and at the floor level. This is evidence that the perimeter box beams and bolts did provide significant fixity to the base of the wall.

In the middle third of each wall, except the side with the opening, the crack pattern followed the horizontal wires of the welded wire fabric placed in the wall.

The reinforcement in the wall was sufficient to force the cracking and subsequent yield-line formation around the perimeter of the structure to occur in the slab rather than at the top of the wall.

The top halves of the walls away from the ends were relatively free of detectable cracks, more so than might be expected considering the large moments placed on the tops of the walls by the slab system.

4.1.2 Loading Characteristics of the Static Test. The 26.6-psi

maximum pressure on the slab consisted of the dead load of the slab and water placed over the slab as well as the water pressure applied to the model during the test.

The applied water pressure reached a maximum of 25.5 psi. This pressure represents the average from the two pressure transducers recording maximum values of 25.4 and 25.6 psi. The calibration of the third transducer was deemed questionable, as it consistently produced values about ten percent above those of the other two transducers. The Bourdon pressure gage monitored throughout the static test gave a maximum value consistent with those from the two pressure transducers yielding the 25.5 psi average.

The dead load of the 2 feet of water placed in the test chamber above the slab surface and before pressurization of the chamber was 0.9 psi, and the slab dead load was approximately 0.2 psi. Thus, the total maximum loading on the slab was 26.6 psi.

After completion of the static test, it was determined that air had remained in the top 3-inch space above the level of the ports around the top of the test chamber when the rest of the space above the slab was filled with water prior to the test. This air was compressed in volume during the loading (to nearly one-third of its original volume at the maximum water pressure) and expanded immediately following the resistance drop of the slab system, thus causing some relatively large jumps in deflections. The very stiff loading system desired was not entirely realized.

4.1.3 Records from the Static Test. Tracings from the paper playbacks of the magnetic-tape records from the static test are presented in Figures A.1 through A.7. The variation of the output from each functioning transducer with time is displayed. These same data are plotted versus the applied water pressure loading in Figures A.8 through A.19. The axes on these plots are reversed from the more common practice of plotting loads on the vertical axis. The effects of the 1.1-psi loading from the dead weight of the slab and water above it are not included in the records.

The records presented in Appendix A which are not shown for the

entire test period were terminated either when the transducer failed or was overranged, or when the gage output was obviously no longer valid.

All 29 pressure, deflection, and column load transducers produced usable records, and 53 of the 61 strain gage channels were operative.

The deflection transducers measuring the slab surface deflections did not produce the desired resolution of small deflections even with the increased amplification used during the first part of the test. A spurious signal of similar pattern appeared on all deflection records during the period of the small deflections. The resolution of these gages was decreased by a factor of 10 at an applied pressure level of 13-1/2 psi. The deflection records above this point appear to be valid until the deflections at several gage locations approached and then exceeded the 7-inch capacity of the transducer. The calibration of deflection transducer D7 appears to have been erroneous because the values from this transducer were consistently higher than others, and the trace appeared valid at values above the transducer capacity. The deflection traces in Figures A.2 and A.3 are shown as dashed lines below a loading of 13-1/2 psi.

The output from several strain gages, notably the G line of gages placed on reinforcement near the slab centerline and of gages placed on the compressive steel, was only a small fraction of the calibration value used, which affected the resolution of these records. As expected, some strains were small throughout early portions of the test and at least a factor of 10 larger when the slab deflections were large, a combination of strains which is difficult to measure accurately throughout its entire range with continuous recording equipment.

In Figures A.8 through A.19, a dashed line has been used to connect the values recorded immediately before and immediately after the sudden resistance drop from the maximum which occurred approximately 13.6 minutes after initiation of the test. The time duration of the record represented by the dashed lines is a fraction of a second. Small arrows on some of the records in this section show the direction in which a strain gage was moving when it failed. Others failed during the resistance drop, with no indication of direction of the record movement at failure.

4.2 DYNAMIC TEST I

4.2.1 Performance of Structure During Dynamic Test I. Only slight damage resulted during the first dynamic loading, which had a maximum overpressure of approximately 20 psi. This overpressure level had been selected because this load magnitude was expected to produce considerable structural damage in the model but would not be likely to lead to total collapse of the slab, thus demonstrating that the structure would survive the design 15-psi overpressure while subjecting the structure to a pressure level thought to be closer to the structure's capacity in light of the static test results.

Figure 4.15 shows the slab surface after this dynamic test. No structural damage is evident in this photograph. The tape holding the plastic membrane covers the first few inches of the slab surface perimeter and also covers the only cracks found on the upper slab surface.

The only other cracks found in the structure in addition to the cracks on the top surface of the slab were two small cracks on the underside of the slab along the centerline of the north row of panels.

The crack over the inner wall face along the north wall, where the maximum crack width was approximately 0.04 inch, is shown in Figure 4.16.

The structure displayed little permanent deflection after the test. The center of the north row of panels exhibited the maximum permanent deflection of approximately 0.15 inch (Figure 4.17).

4.2.2 Load Characteristics of Dynamic Test I. The records from the seven pressure transducers placed on the slab surface at the locations shown in Figure 3.35 are displayed in Figure B.1. The pressure records show many peaks during the first few milliseconds, a phenomenon usually observed with a Primacord explosive loading.

To obtain the maximum effective overpressure for the test, as opposed to the magnitude of a single-peak pressure spike, a smoothed curve was fitted to each pressure trace, as shown in Figure 4.18a. The characteristics of the fitted curves are presented in Table 4.1, and the average overpressure-time curve is shown in Figure 4.18b.

Gage P4 gave values considerably below those of the other six pressure gages and is thought to have been defective. Although some

pressure variations with location can be expected in the LBLG due to detonation pattern, Primacord placement, and the pressure reflection and baffling characteristics of the loading device, the large difference between the 6.3-psi value from P₄ at 150 msec and the 11.2- to 13.2-psi values from other gages almost certainly could not physically exist, especially considering the proximity of Gage P₄ to Gages P₂ and P₅. On this basis, the values from Gage P₄ were not included in determining the average overpressure-time characteristics.

The average peak effective overpressure acting on the slab was 19.7 psi at a rise time of 8.5 msec; this pressure decayed to 11.9 psi at 150 msec (Figure 4.18b, solid curve).

The distribution of the overpressure acting on the slab surface at an elapsed time of 50 msec is displayed in Figure 4.18c.

4.2.3 Records from Dynamic Test I. Plots of the records from the first dynamic test versus time are presented in Figures B.1 through B.14.

All data channels shown in these figures remained functional throughout the 170- to 200-msec time period shown.

Since no useful information was obtained from deflection Gages D1 through D13, the transducers measuring slab surface deflection, these records are not shown in Appendix B. The deflection records were completely dominated for approximately the first 15 msec by a fairly high-frequency signal with amplitudes corresponding to several inches of deflection. This, coupled with the deflections during the first dynamic test being much smaller than expected (and thus only a small fraction of the calibration level), resulted in the effective loss of these channels. The almost acceleration-like deflection traces measured during the first dynamic test apparently resulted from accelerations and vibrations during the first few milliseconds causing the moving probe and the stationary parts of the deflection transducer to make and break electrical contact in rapid sequence. These transducers did not display acceleration sensitivity in a previous program (Reference 8) and were electrically insulated from the aluminum supporting framework during the second dynamic test.

The acceleration records were doubly integrated to produce velocity

and deflection information for the structure. The accelerations along with the deflection- and velocity-time curves are displayed in Figures B.3 through B.7. The rigid-body motion of the entire structure is included along with the motions from the deflection of the slab or walls in these records.

Most of the steel strains measured during the first dynamic test were considerably below the strain levels that would cause yielding of the reinforcement except along the inner wall face (A-line of gages).

The validity of Record F4 (Figure B.9) is questionable, at least during the first few milliseconds.

Usable records were obtained from all accelerometers, pressure transducers, and load cells during the first dynamic test and from 36 of the 46 strain gages.

4.3 DYNAMIC TEST II

4.3.1 Performance of Structure During Dynamic Test II. Extensive structural damage resulted during the second dynamic test of the structure. An overpressure level of from one-third to one-half times greater than that of the first dynamic test was planned to insure that the failure mode of the model could be determined without further testing.

The model after exposure to the 30.1-psi maximum overpressure loading is shown in Figure 4.19. The manner of failure was similar to that of the statically loaded model except that it was more complete and symmetrical, as would be expected with a dynamic loading above the dynamic resistance of the slab. The column areas exhibited extensive distress, and the bar splices in the zone near the other faces of the column failed allowing the center panel to pull all four columns toward the center of the structure. Well-developed flexural yield lines formed near the centerlines of the outer panels, in the corners of the slab, and around the slab perimeter over the inner wall surface. The walls were extensively cracked and pulled or pushed in toward the center of the structure. Figure 4.20 shows closer views of the slab surface and better conveys the magnitude of the slab deflections.

The deflections of the slab surface measured after the structure

had been removed from the testing fixture are shown in Figure 4.21. Except for the areas near the columns, the deflected shape of the slab inside the yield lines near the outside panel centerlines was fairly flat. This part of the slab surface dropped approximately 12-1/2 inches (23 percent of the column spacing), with a maximum value of 15-1/4 inches (28 percent of the column spacing). Each of the four trapezoidal shapes bounded by a side of the structure remained nearly plane.

Considerable debris was visible inside the structure (Figure 4.22), and the columns were tilted in approximately 30 degrees from the vertical.

The slab surface with all debris removed is shown in Figure 4.23. The bow in the east wall can be seen by comparing this edge to the straight edge. Sizable areas of the top surface remained relatively uncracked, and extensive spalling of the concrete occurred near the corners of the slab (Figure 4.24). This spalling did not appear to have been caused by the shock wave, but seemed to have been caused by the general breakup of the concrete as it failed in compression. The rotation of the slab and the shear forces were both large enough that the slab around its perimeter appeared to be largely supported by a suspension of the slab from the reinforcement crossing the section (note the reinforcement along the wall to the right of the corner in Figure 4.24b). The top and bottom slab surfaces between two columns are shown in Figure 4.25.

Each of the four columns of the structure displayed compression-induced spalling of the concrete shell at the top of the column on the side closest to a slab corner (Figure 4.26). Tensile cracks were found near the tops of some columns on the inner side. Splitting and some spalling of the cover of the spiral columns are also shown in Figure 4.26. The drop panels were completely torn from the slab, but the column capitals were not damaged.

Figure 4.27 shows the crack patterns on the east, north, and west walls of the structure. Large horizontal cracks can be seen at about midheight of the walls extending over the center three-fourths of the slab, especially for the east and north walls. The portion of the wall

above this crack and its vertical extension to the slab surface near the slab corners rotated into the slab. Figure 4.28 shows the four walls with all cracks marked. The crack patterns were similar to those of the statically tested specimen, but more extensive. The cracking reflected the near fixity of the slab on three sides and followed the horizontal reinforcements over the middle two-thirds of the wall.

The greater than 2-inch inward movement of the east wall and the concentration of the rotation of this wall at the horizontal crack near the wall midheight are best shown in Figure 4.29. An L-shaped portion of the slab and wall located approximately 1 foot from the corner of the slab appeared to have failed primarily in torsion (Figure 4.30), thus allowing the wall inside this location to rotate.

4.3.2 Load Characteristics of Dynamic Test II. Figures B.15 and B.16 present the records of the seven pressure transducers placed on the surface of the slab, designated P1 through P7.

The characteristics of the dynamic loading were determined in the same way as outlined in Section 4.2.2 for Dynamic Test I and are shown in Table 4.2. Again, the readings from Transducer P4 were much lower than those from the other pressure transducers and were not considered in determining the average applied overpressure characteristics. Some pressure transducers failed to produce records as slab failure progressed, possibly because of damage to the lead wires inside the structure. The pressure values from P2 and P3 differ considerably after the first 75 msec, and the hostile conditions inside the structure are thought to be the cause.

The effective overpressure-time history for Dynamic Test II is shown in Figure 4.31. The average peak overpressure acting on the slab was 30.1 psi at a rise time of 4.5 msec. The overpressure decayed much more rapidly during Dynamic Test II than during Dynamic Test I, starting at about 35 to 45 msec into the test. This rapid pressure decay extended out to about 100 msec time (Figure 4.31) and resulted from the deflection and rupturing of the shelter roof slab, which allowed the pressurized gas above the structure to expand into the interior of the structure.

4.3.3 Records from Dynamic Test II. The transducer output versus time plots of the records from Dynamic Test II are presented in Figures B.15 through B.17.

The records from many gages are not shown for all of the first 170 to 200 msec of the test because the transducers were overranged or damaged during this period. Some lead wires were damaged, and the output of other gages became invalid.

The quantities measured during Dynamic Test II do not include any strains or deflections remaining at the start of this test from the Dynamic Test I loading.

The deflection transducers produced usable records after the initial 15- to 20-msec period that contained the same type of spurious signal as was experienced in Dynamic Test I. The records in Figures B.17 and B.18 have been filtered to reduce the magnitude of this signal.

The output from the four load cells placed under the columns are valid only until the columns tilted enough to bring the two steel plates on either side of bearing spheres into contact, an action which caused highly eccentric loads to be placed on the load cells. Further tilting of the column caused the top plate to be pried from the gaged portion of the load cell to which it was press fitted. The dashed portions of the load cell traces (Figure B.16) are not thought to be accurate.

The maximum strains measured by each strain gage channel during the Dynamic Test I are noted in Figures B.23 through B.27. As previously mentioned, the strains displayed in these figures are only the increment added by Dynamic Test II. The small arrow on many of the strain gage traces marks the approximate time of gage failure and the direction of trace movement at failure.

TABLE 4.1 CHARACTERISTICS OF APPLIED PRESSURE, DYNAMIC TEST I

Pressure Gage	Peak Pressure	Time to Maximum Peak	Pressure at 50 msec	Pressure at 100 msec	Pressure at 150 msec
	psi	msec	psi	psi	psi
P1	17.6	5.0	15.8	13.8	11.2
P2	19.6	8.4	17.2	14.4	12.0
P3	21.6	11.3	19.0	15.6	13.2
P4 ^a	13.8	8.0	11.1	8.3	6.3
P5	19.4	2.8	16.8	13.8	11.3
P6	20.1	12.0	16.4	14.9	12.1
P7	20.0	11.3	15.5	12.1	11.3
Average ^b	19.7	8.5	16.8	14.1	11.9

^a Considered to be questionable data (see Section 4.2.2).

^b Values from Gage P4 not used in computing averages.

TABLE 4.2 CHARACTERISTICS OF APPLIED PRESSURE, DYNAMIC TEST II

Pressure Gage	Peak Pressure	Time to Maximum Peak	Pressure at 25 msec	Pressure at 50 msec	Pressure at 100 msec	Pressure at 150 msec
	psi	msec	psi	psi	psi	psi
P1	29.0	4.2	27.5	23.1	a	a
P2	25.6	4.7	25.6	22.3	8.6	6.0
P3	32.9	4.9	32.2	28.3	19.6	16.6
P4	16.0	4.8	15.9	11.0	a	a
P5	31.1	4.3	29.8	26.2	14.3	12.0
P6	29.6	4.0	28.8	24.5	12.0	11.2
P7	32.6	(9.8)	30.0	23.3	a	a
Average ^b	30.1	4.5 ^c	29.0	24.6	13.6	11.4

^a Indicates that no record was obtained at the listed time.

^b Values from Gage P4 not used in computing averages.

^c Average without P7.

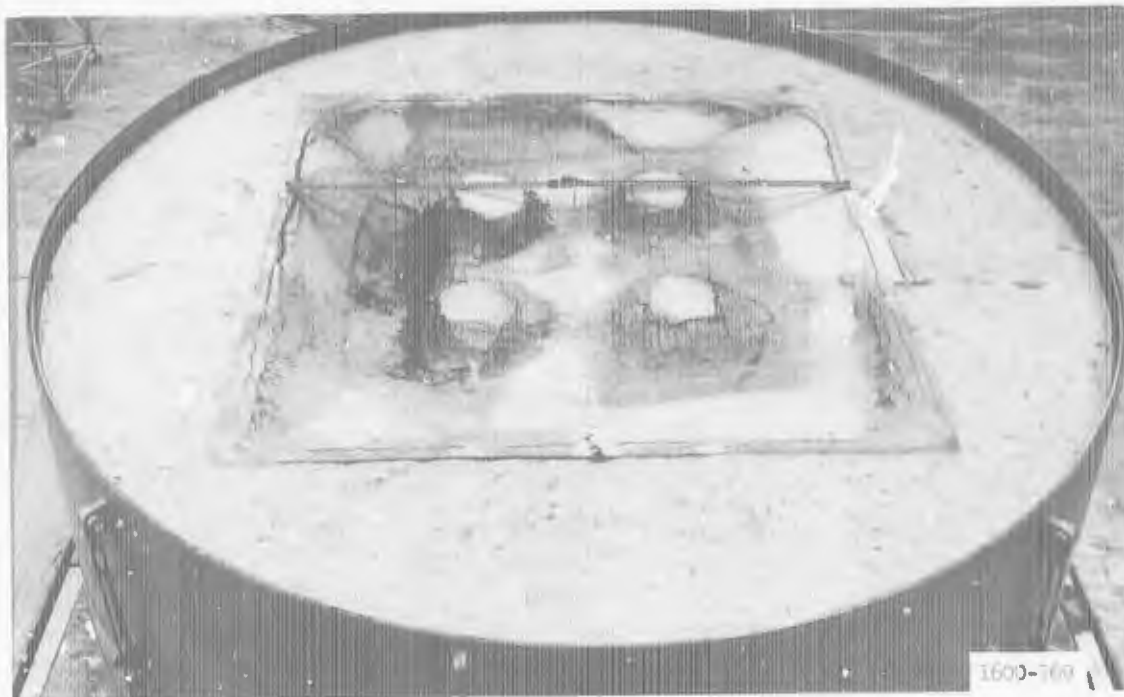


Figure 4.1 Statically tested model structure in testing chamber.

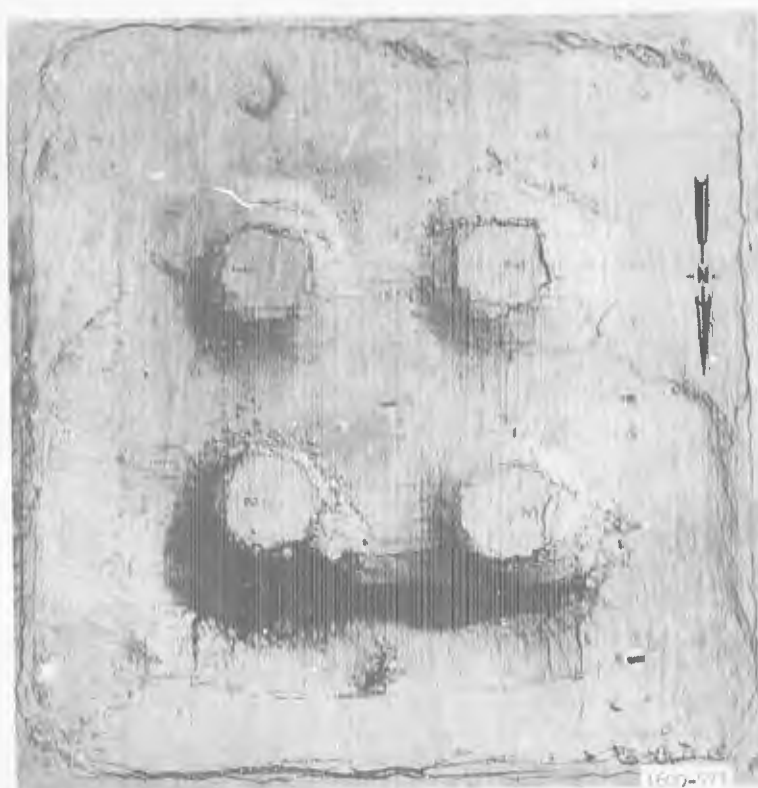


Figure 4.2 Top view of statically tested slab.

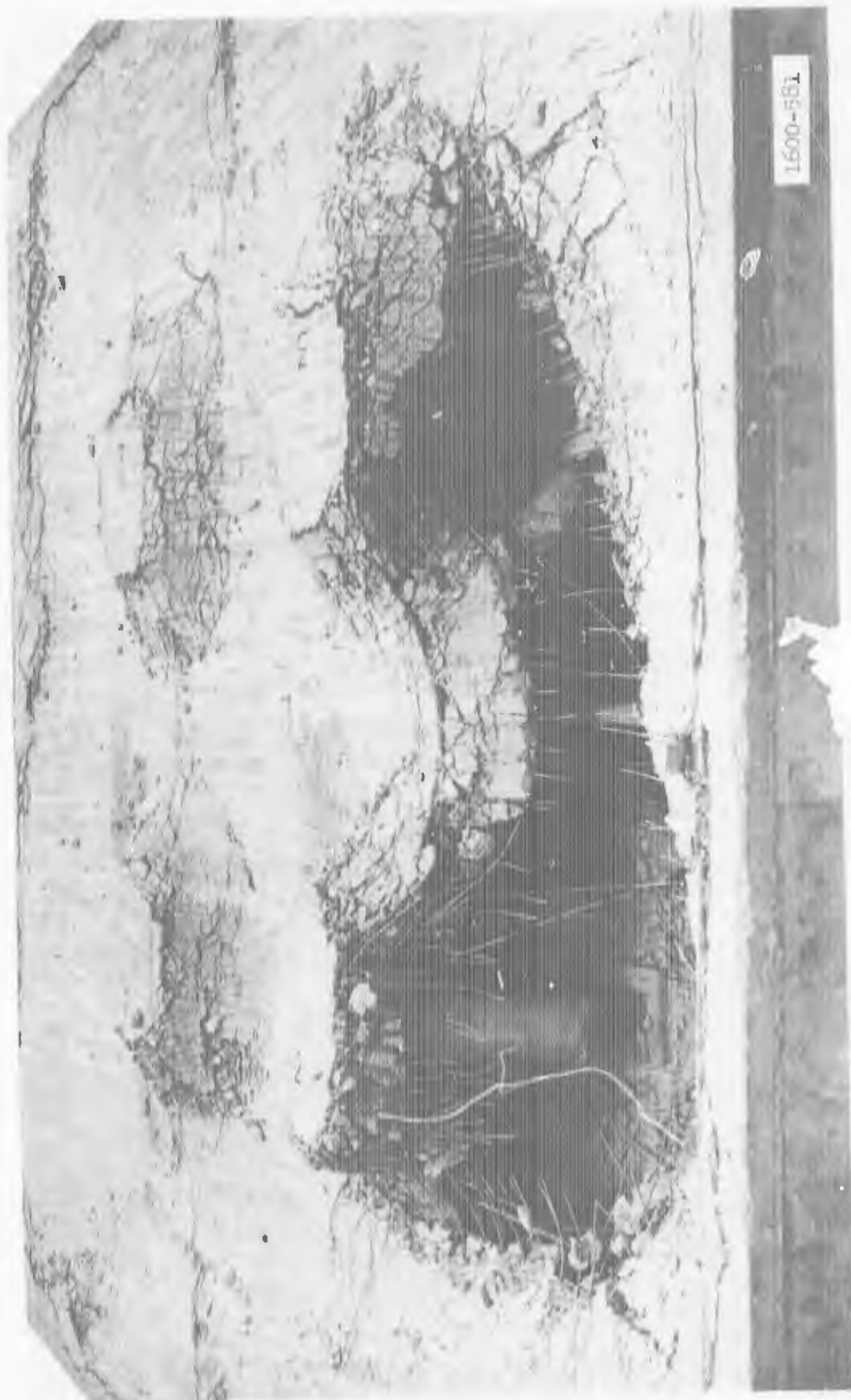


Figure 4.3 Rupture zone and deflected shape of slab after static test (looking south).



a. Center of slab from west side



b. Northeast slab corner.

Figure 4.4 Details of slab surface after static test.

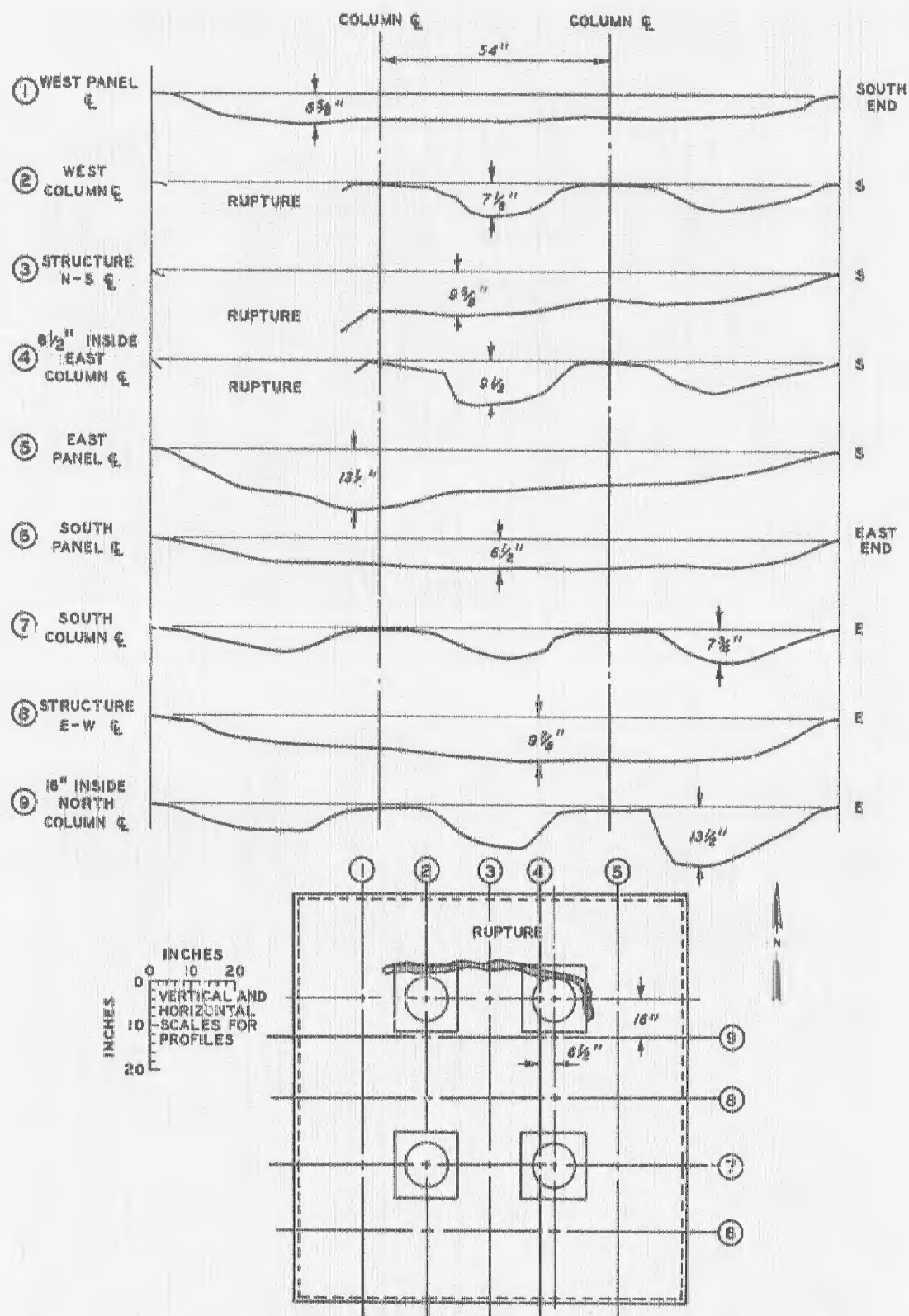


Figure 4.5 Slab profiles for statically tested model.



Figure 4.6 Interior of statically tested structure as viewed from south wall opening.

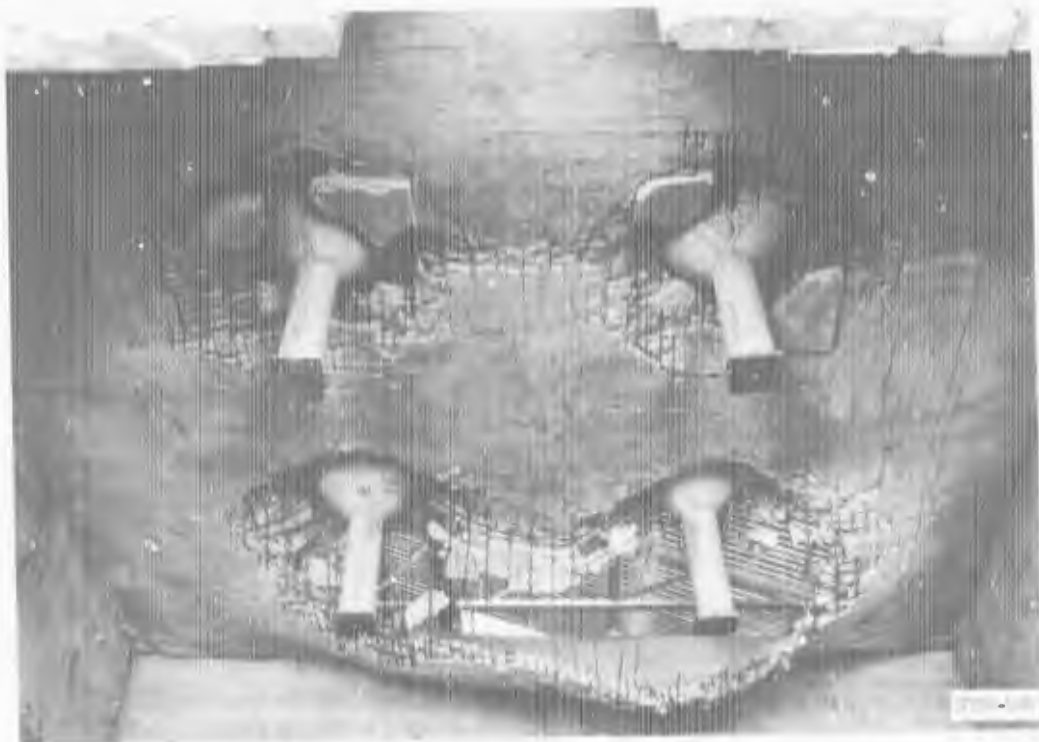


Figure 4.7 Bottom slab surface as viewed from south after static test.

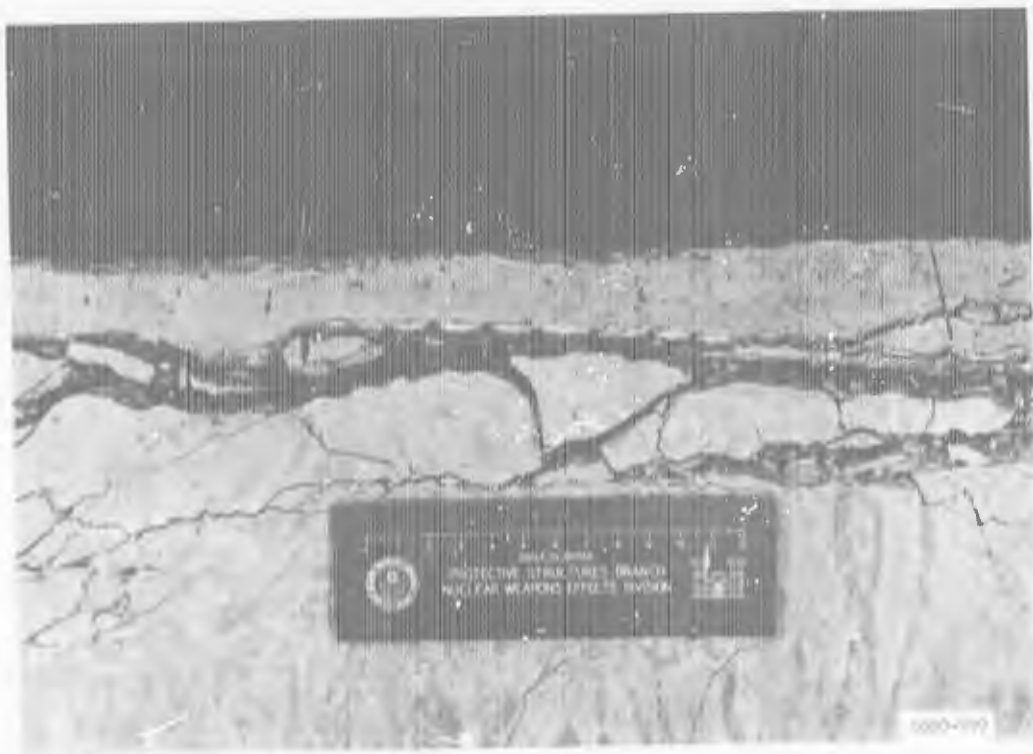


Figure 4.8 Detail of crack in slab along north wall after static test.

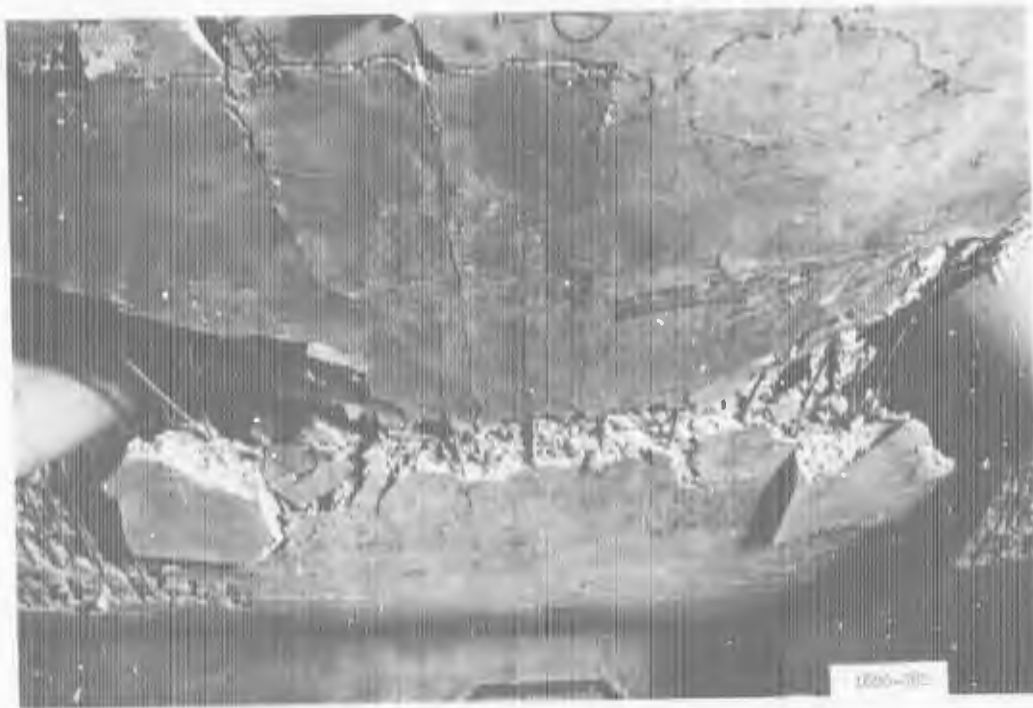
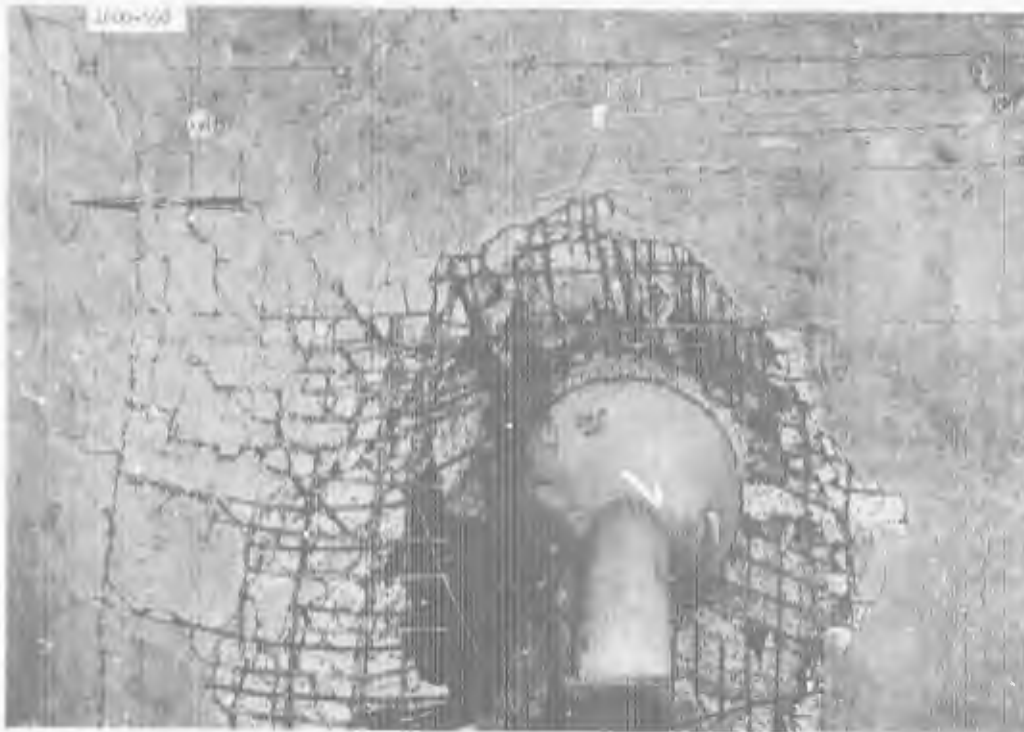


Figure 4.9 South half of center panel bottom slab surface and beyond after static test.



a. Northwest column area.



b. Northeast column area.

Figure 4.10 Bottom slab surface near the north columns after static test.

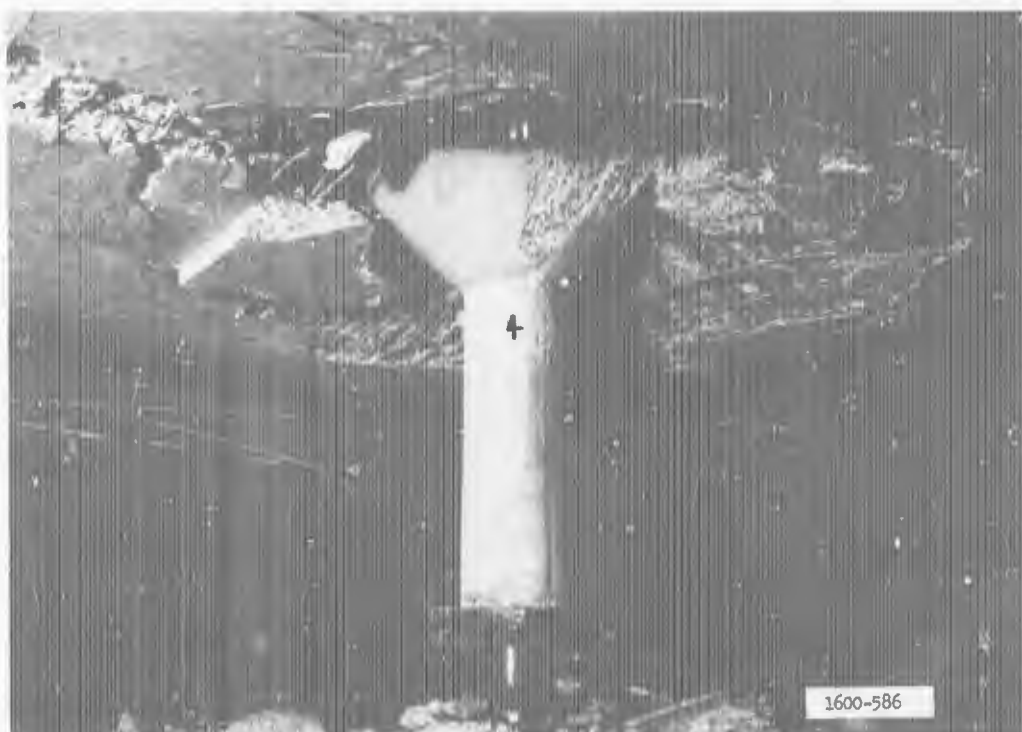


a. Northwest column.



b. Northeast column.

Figure 4.11 Columns and adjacent area after static test (sheet 1 of 2).



c. Southwest column.



d. Southeast column.

Figure 4.11 (sheet 2 of 2).

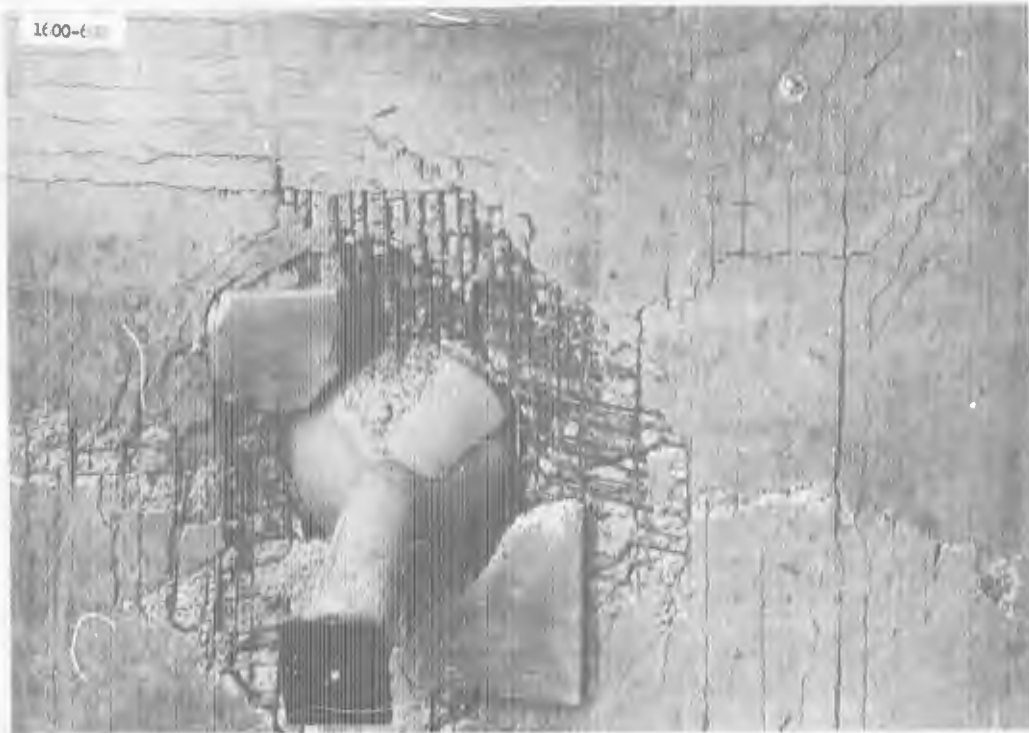


Figure 4.12 Column capital and drop panel of southeast column after static test.

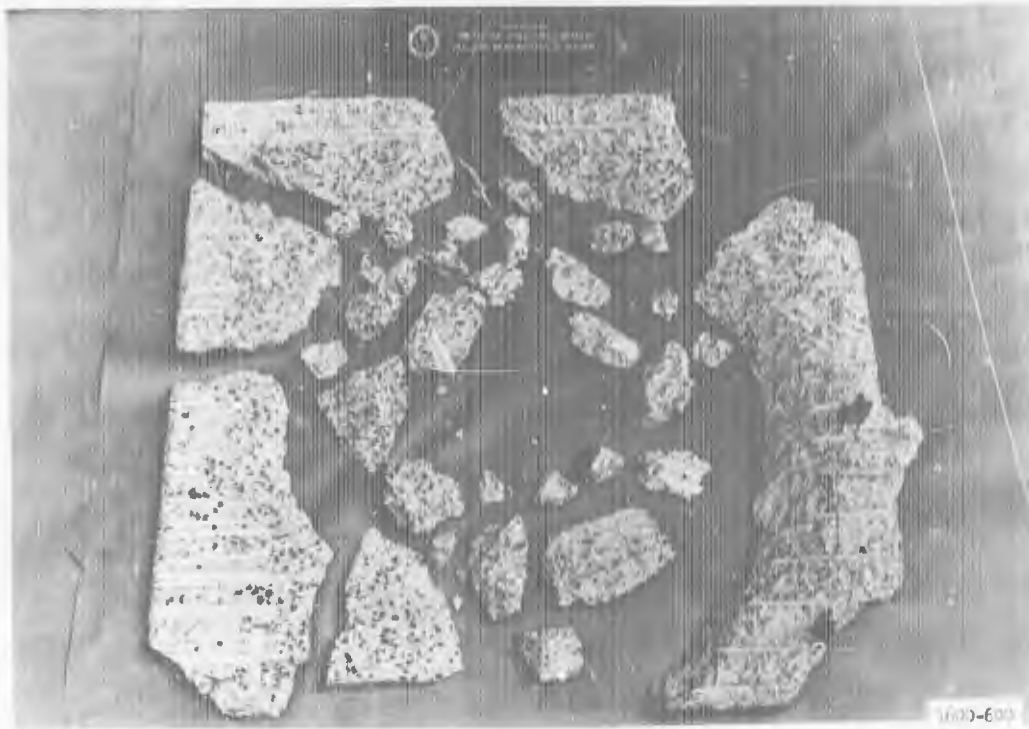
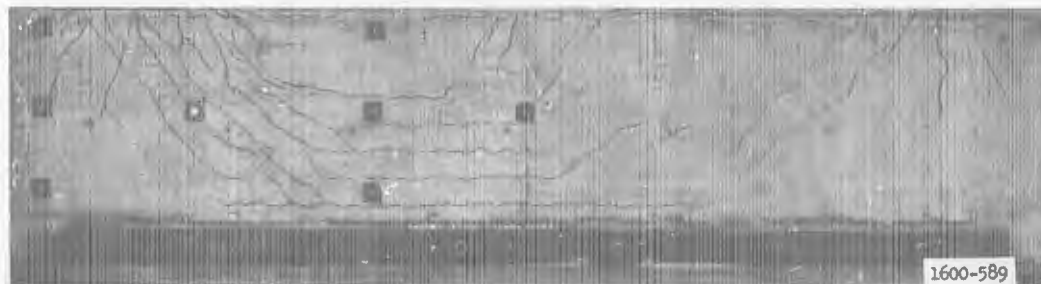


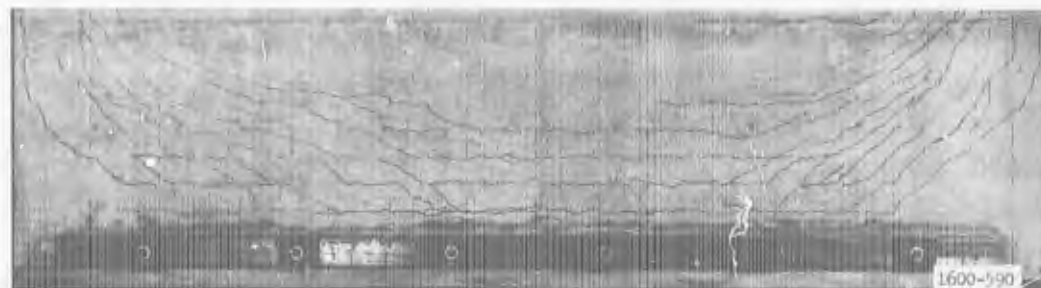
Figure 4.13 Debris from drop panel around northeast column after static test.



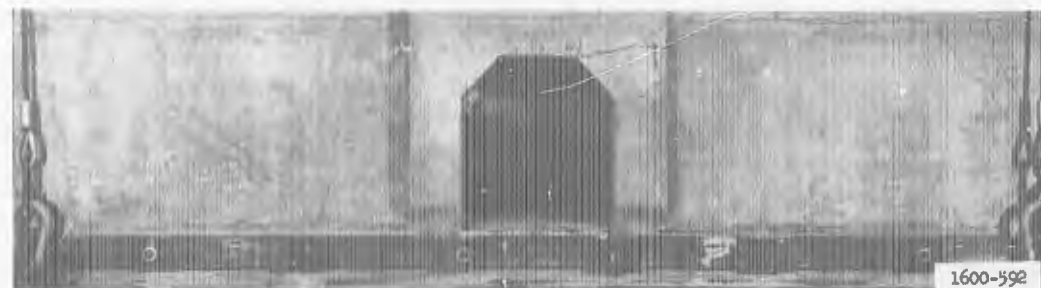
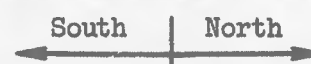
a. North wall.



b. West wall.



c. East wall.



d. South wall.



Figure 4.14 Walls of the statically tested model with cracks marked.



Figure 4.15 Slab surface after Dynamic Test I.



Figure 4.16 Crack over inner wall face, north side of specimen after Dynamic Test I.

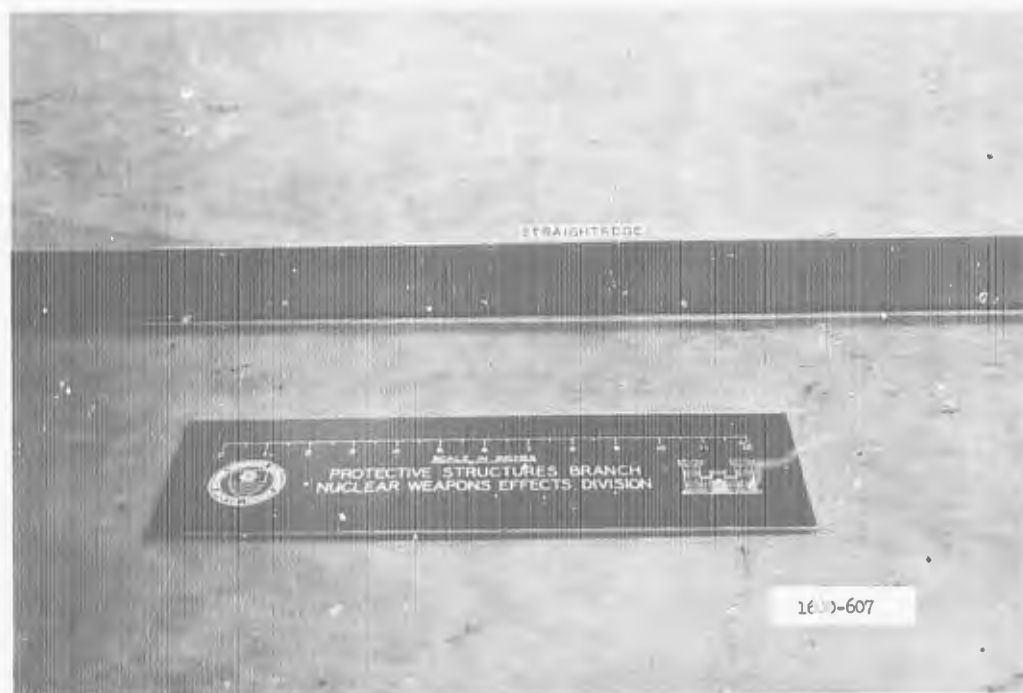
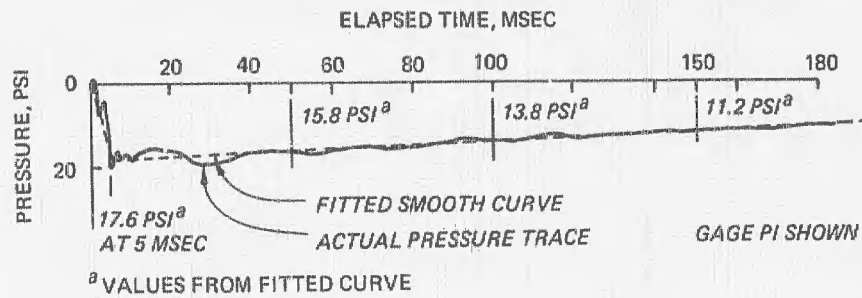
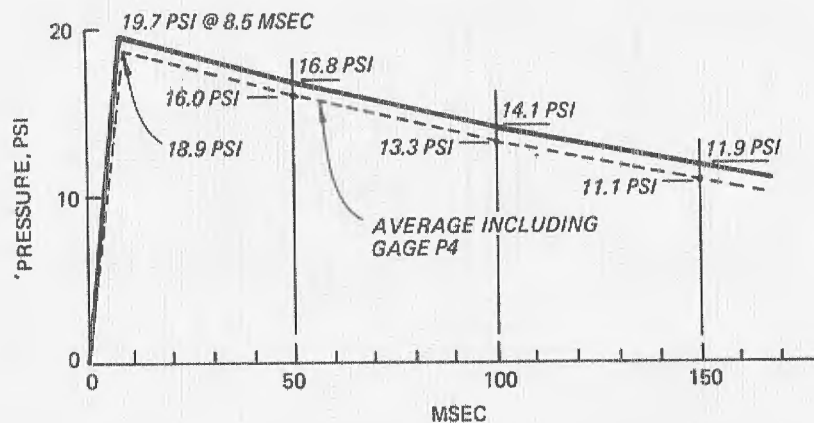


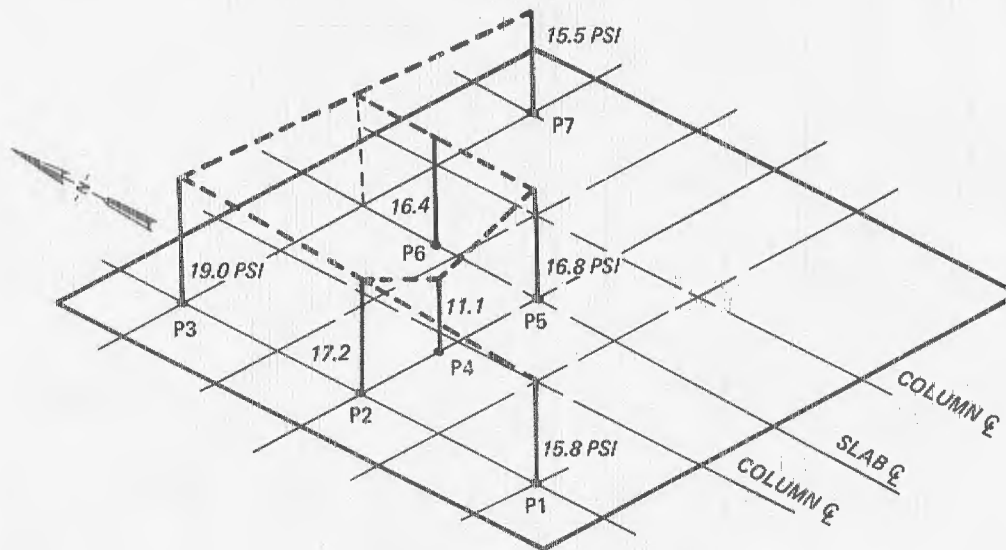
Figure 4.17 Permanent deflection of the north row of panels after Dynamic Test I.



a. EXAMPLE OF FITTED CURVE TO ACTUAL PRESSURE TRACE



b. AVERAGE OVERPRESSURE-TIME CURVE, DYNAMIC TEST I



c. VARIATION OF SURFACE OVERPRESSURE ON THE SLAB STRUCTURE AT 50 MSEC

Figure 4.18 Characteristics of applied load (Dynamic Test I).

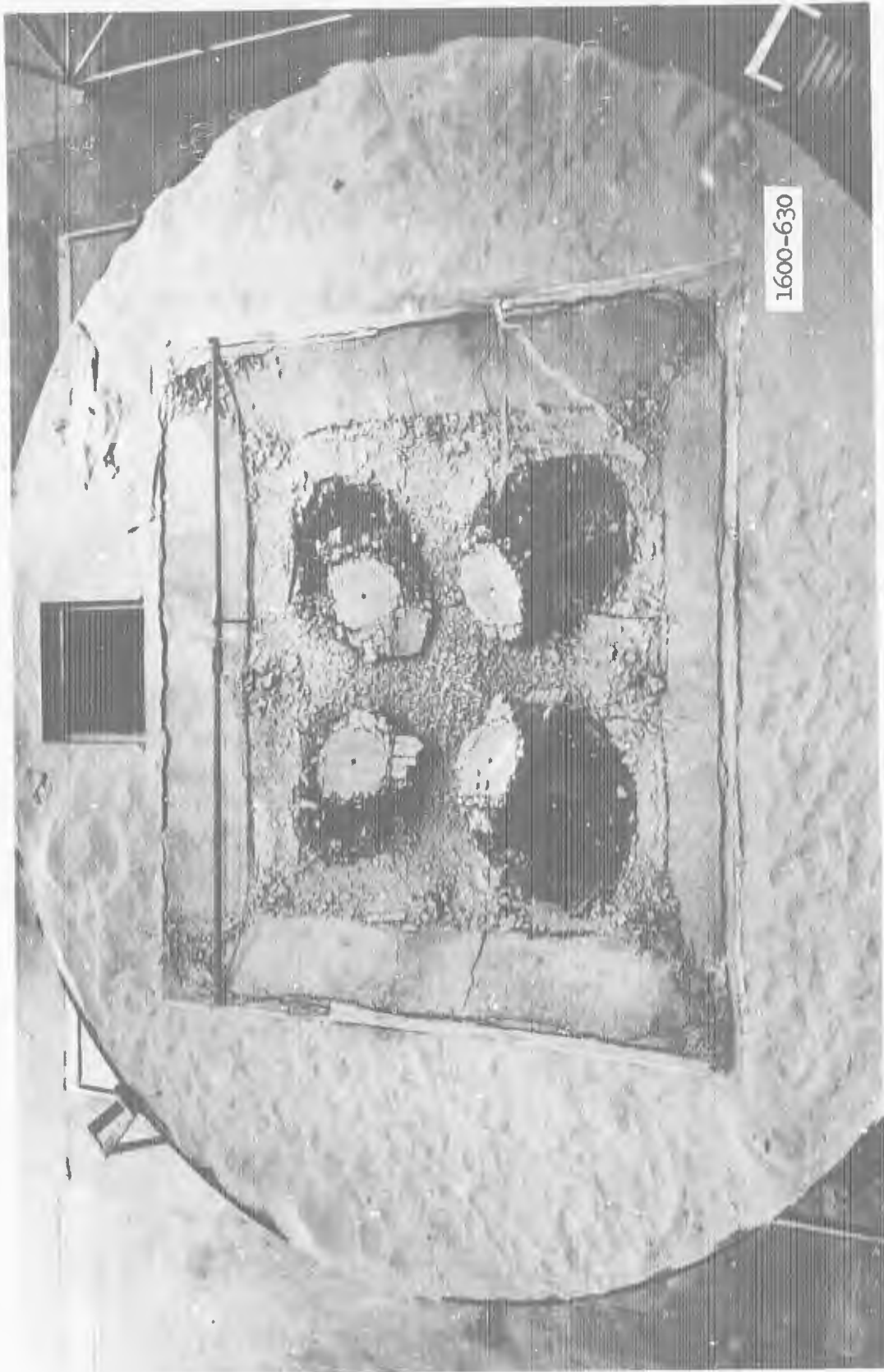
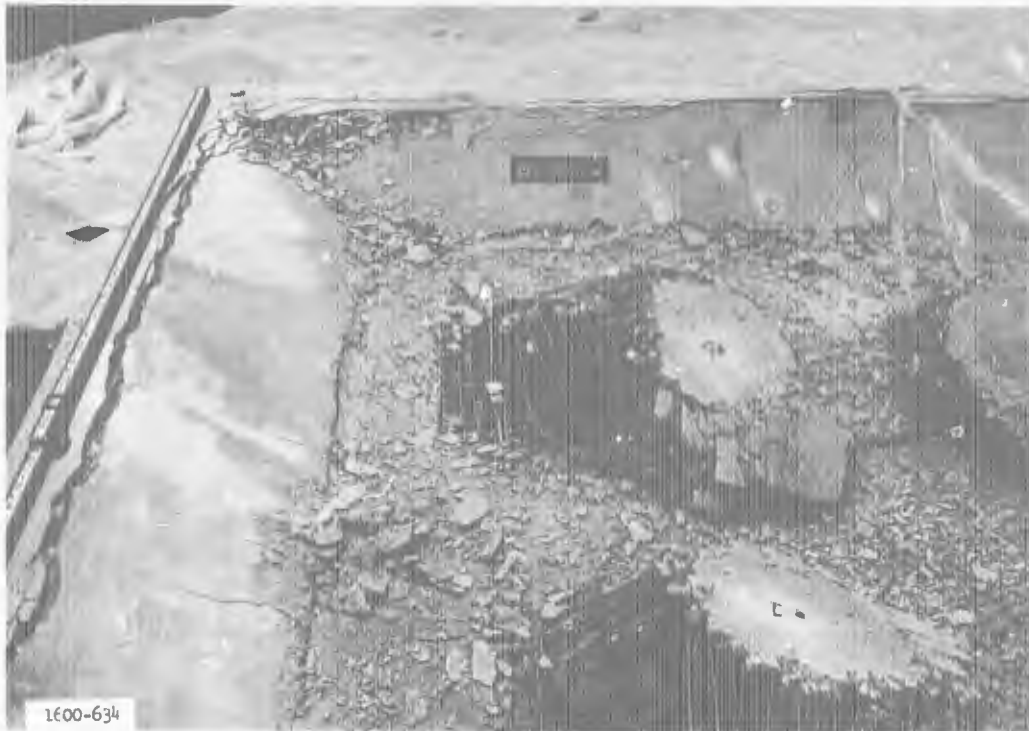


Figure 4.19 Slab surface after Dynamic Test II.



a. Southwest corner of the slab.



b. Northwest corner of the slab.

Figure 4.20 Details of the slab surface after Dynamic Test II.

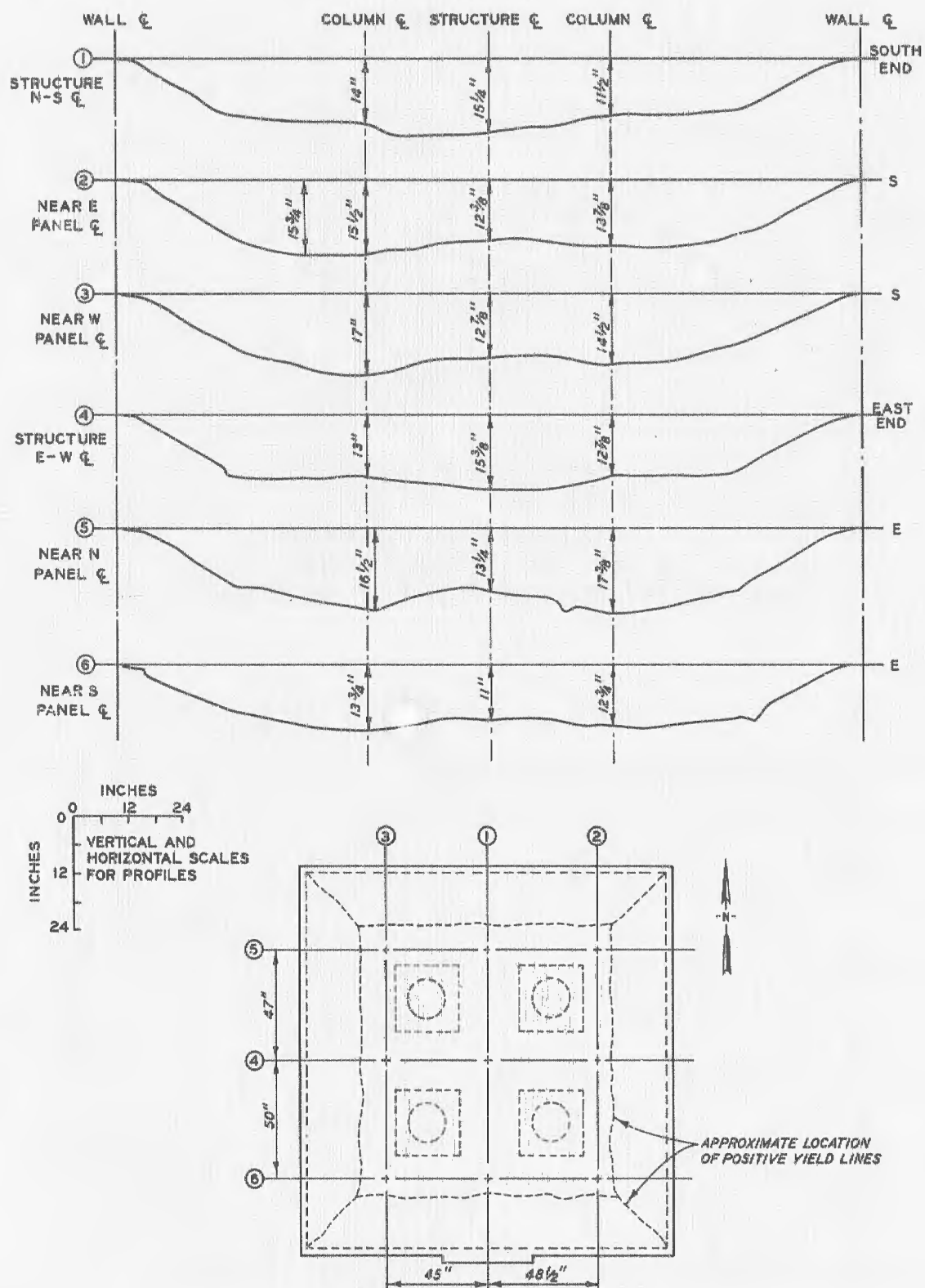


Figure 4.21 Slab profiles after Dynamic Test II.



Figure 4.22 Interior of the model after Dynamic Test II.

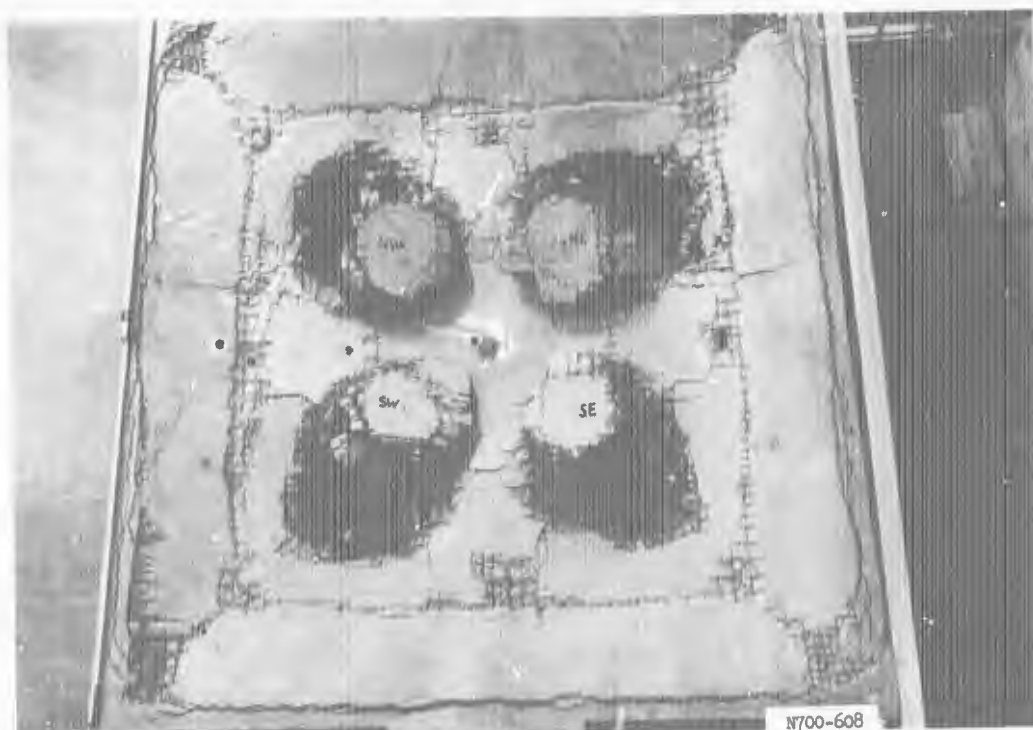
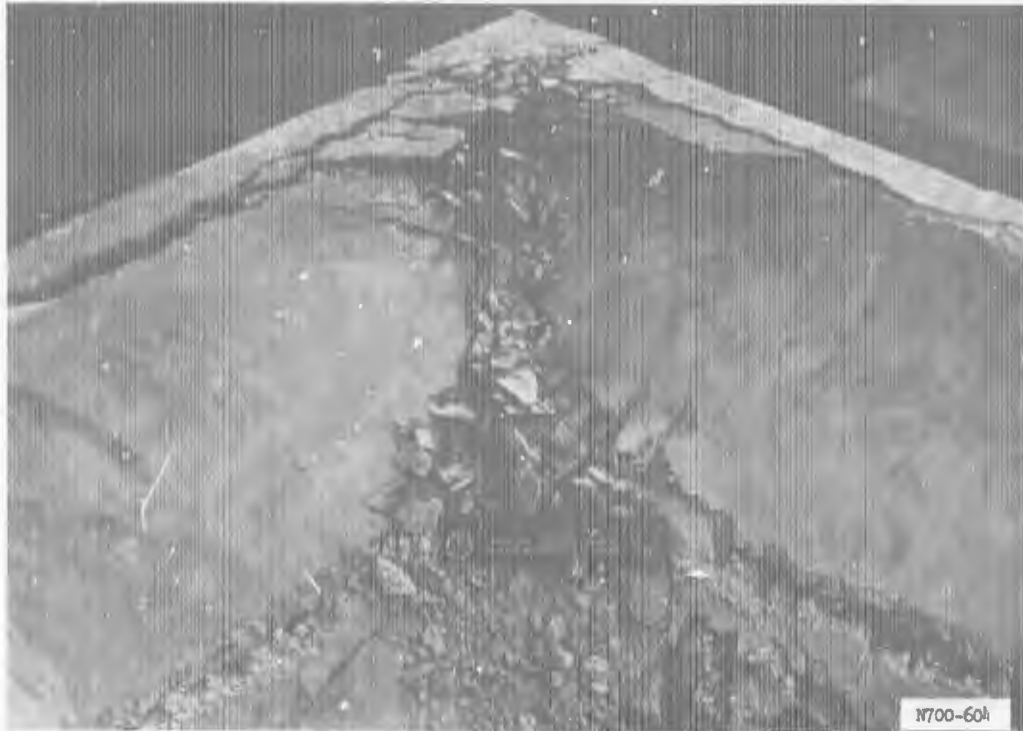
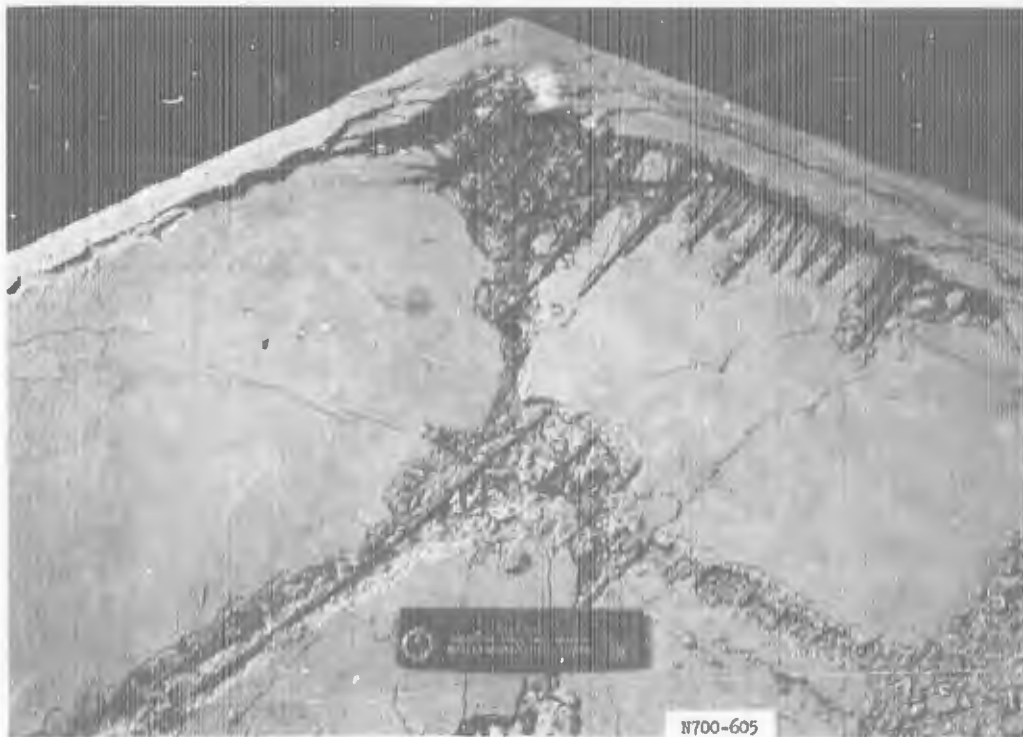


Figure 4.23 Slab top surface with debris removed after Dynamic Test II.

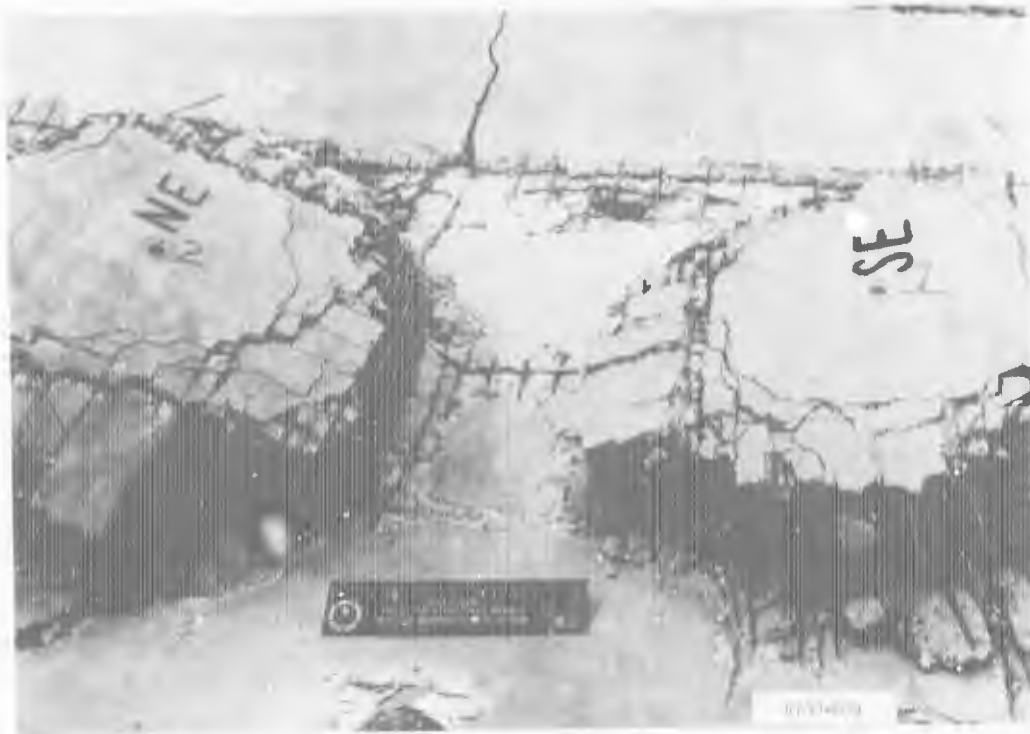


a. Northeast slab corner.

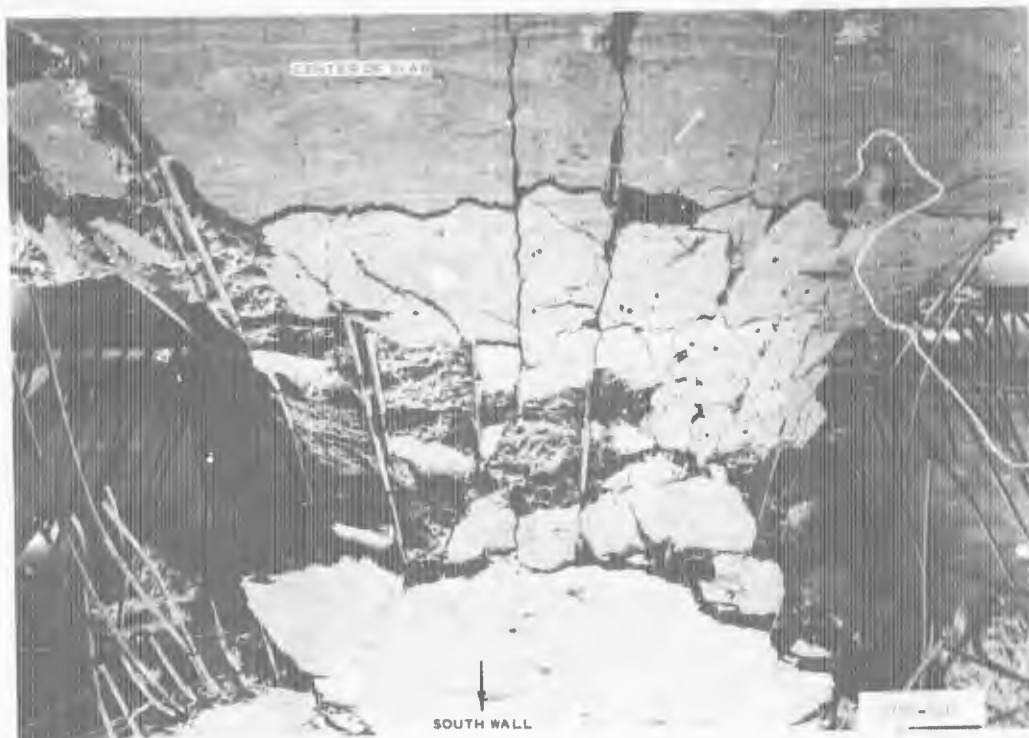


b. Southwest slab corner, debris removed.

Figure 4.24 Cracking and yield-line formation at the slab corners after Dynamic Test II.

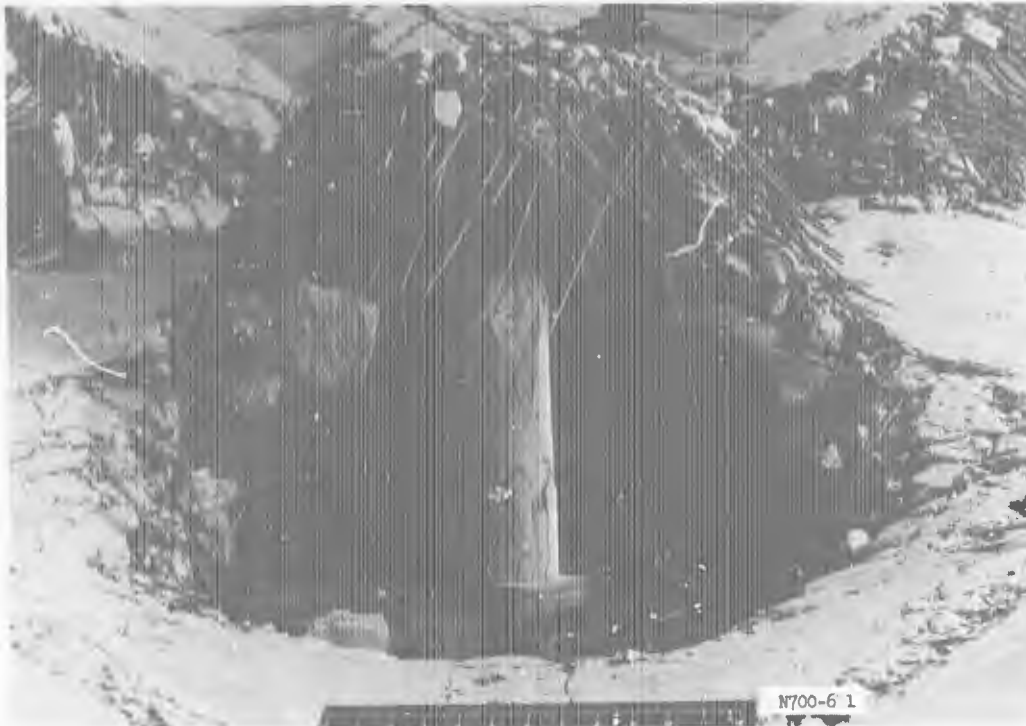


a. Top of slab between east columns.

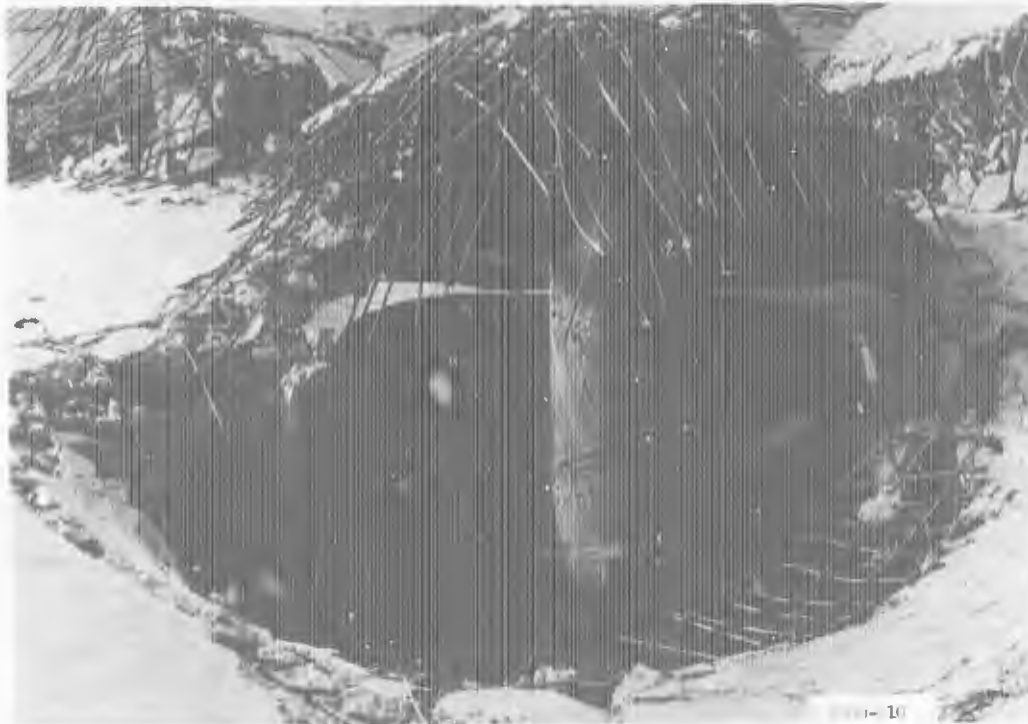


b. Bottom of slab between south columns.

Figure 4.25 Cracking and concrete spalling between columns after Dynamic Test II.



a. Northwest column.

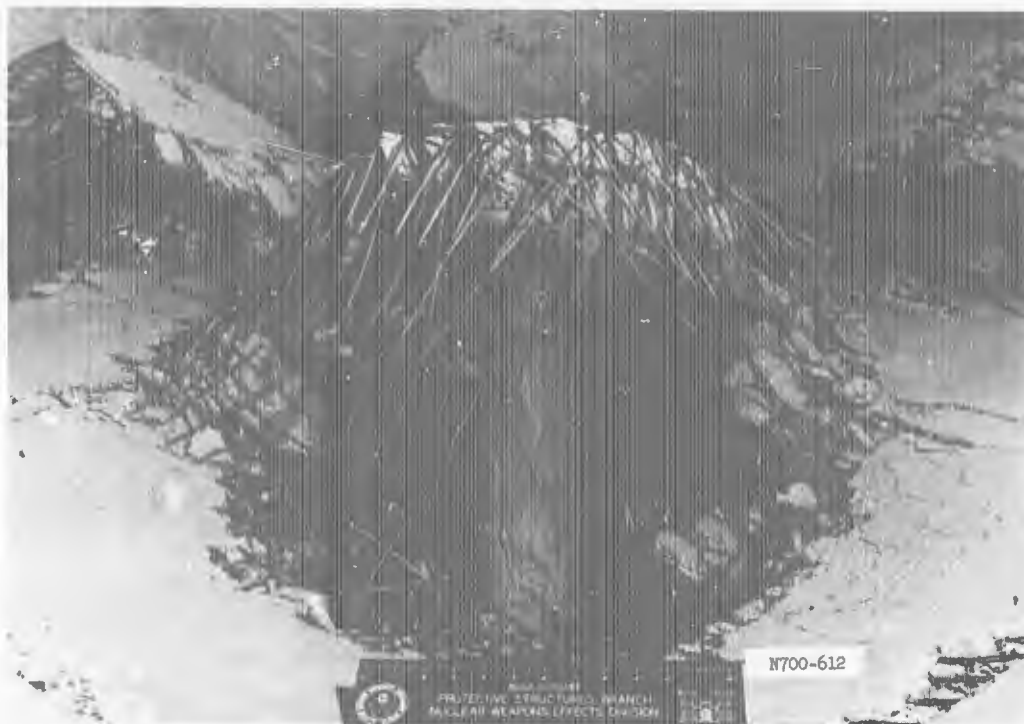


b. Northeast column.

Figure 4.26 Columns and adjacent areas after Dynamic Test II
(sheet 1 of 2).

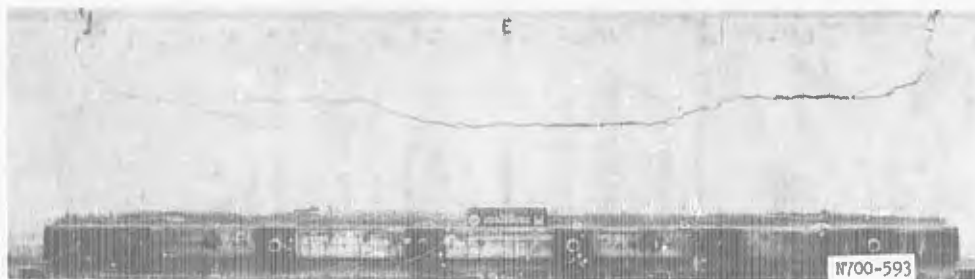


c. Southwest column.

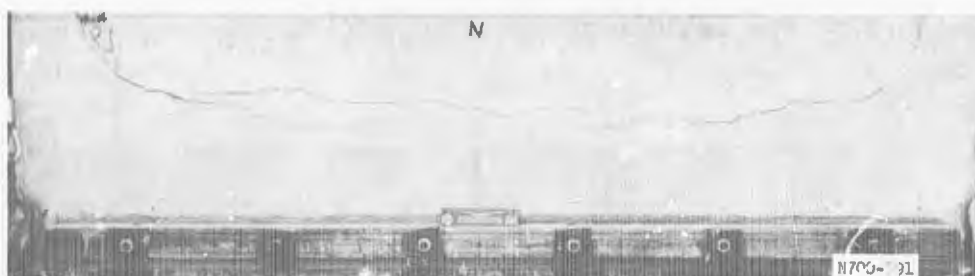


d. Southeast column.

Figure 4.26 (sheet 2 of 2).



a. East wall.

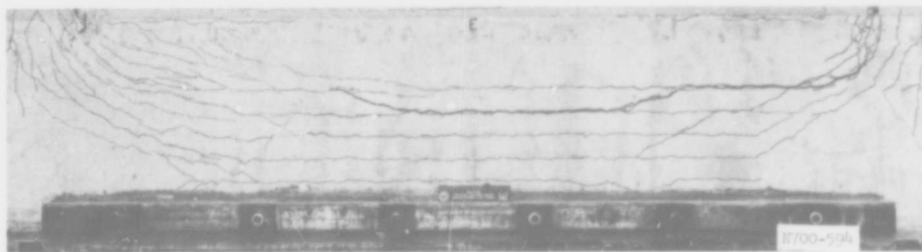


b. North wall.

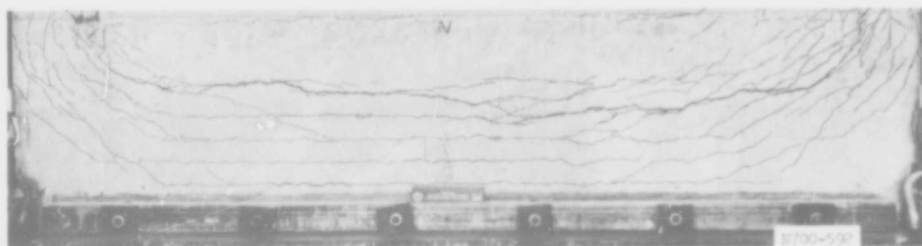


c. West wall.

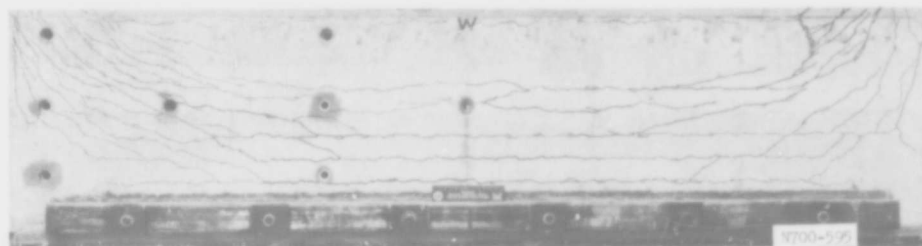
Figure 4.27 Model walls after Dynamic Test II.



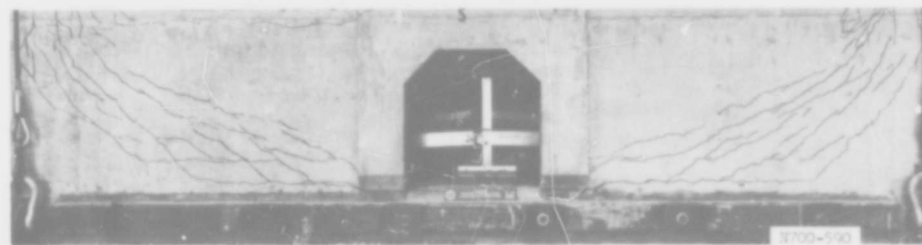
a. East wall.



b. North wall.



c. West wall.



d. South wall.

Figure 4.28 Model walls after Dynamic Test II with cracks marked.



Figure 4.29 Rotation at east wall midheight after Dynamic Test II.

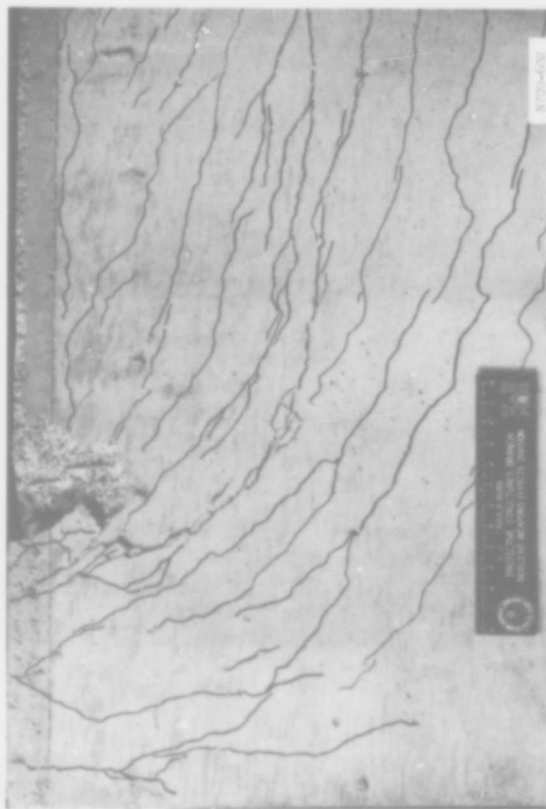


Figure 4.30 Cracking and rotation at top of the south end of the east wall after Dynamic Test II.

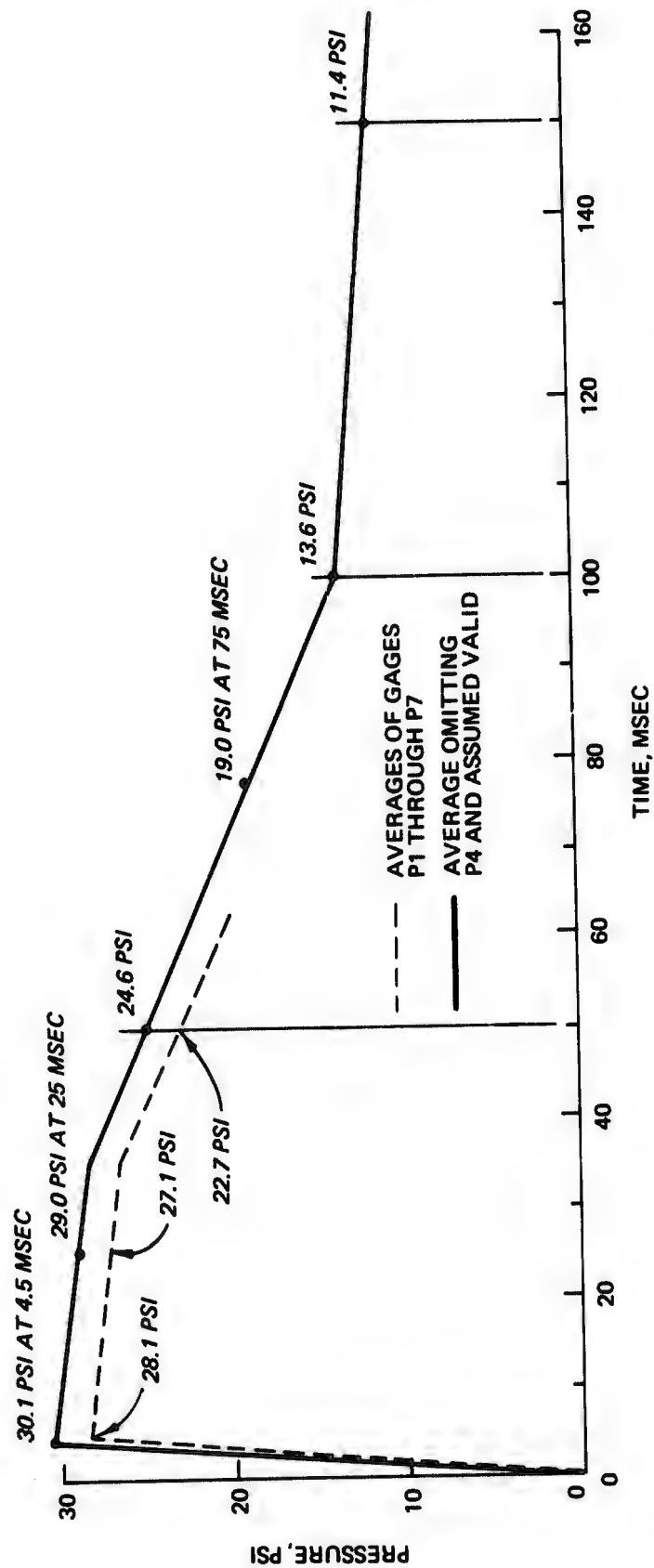


Figure 4.31 Average overpressure-time curve for Dynamic Test II.

CHAPTER 5

DISCUSSION AND ANALYSIS OF STATIC TEST RESULTS

5.1 BEHAVIOR OF THE SHELTER MODEL

5.1.1 Behavior Prior to Maximum Slab Resistance. The slab system displayed increasing resistance with deflection throughout this portion of the loading history. The stiffness of the slab at several deflection transducer locations decreased appreciably during the application of the last few increments of load (see especially the deflection-load plots for D1, D7, D8, and D10, Figures A.9 and A.10), but at no observed location did the slab display the leveling off or decrease in resistance with increasing deflection associated with the attainment of flexural capacity and yielding behavior.

The deflected slab shapes at approximately 55 and 100 percent of the maximum load are shown in Figure 5.1. At the 25.5-psi applied pressure level, the outer panels each deflected in a trough shape with little difference in deflection between the panel center points and the adjacent midspans of the column-to-wall spans. The deflection of the interior panel (0.67 inch) was considerably less than for the other panels. Most exterior panel deflections at maximum loading were about 43 percent (ranging from 31 to 48 percent) of the slab thickness of 2-3/4 inch. The deflections appear reasonably symmetrical considering that the deflections of any appreciably weaker panel of a multiple-panel slab model would differ considerably from the others under static loading conditions.

The compressive in-plane forces acting on the slab resulting from both the soil loadings on the walls and the restraint of the slab movement by the walls were very important in increasing the flexural capacity during this stage of response. This effect is discussed in Section 5.3.

The slab deflections were not large enough during this initial portion of the loading to allow the formation of a tensile membrane action. A flexural yield-line pattern was probably in the process of forming

when the maximum load was reached, but no mechanism resulted because the pattern remained incomplete. The deflected shape shown in Figure 5.1b suggests that the yield-line pattern assumed in the design (Figure 2.8a), which includes only the exterior panels, was being formed.

The necessity of testing the model in a closed chamber and the use of instrumentation chosen to measure large deflections prevented much potentially interesting and useful information from being obtained during this portion of the static test. These data, such as observations of the sequence of crack formation and a more precise definition of the slab deflected shape at low load levels, would have been most useful in examining the performance of the slab system at working load levels for conventional uses of the slab. This was considered to be a secondary goal of this program.

5.1.2 Behavior at and Immediately After Maximum Slab Resistance.

The load-carrying capacity of the slab was limited by a punching shear distress that developed almost simultaneously at all four interior columns at an applied load of 25.5 psi. This inclined crack formation and associated separation between the slab and the column capital will be referred to as shear distress rather than shear failure because the term shear failure would suggest a total loss of load-carrying capacity to many readers. This distress developed very rapidly at all four column locations because the distress at the initiating column and the associated slab motion resulted in the transferring of additional vertical forces, and, probably more importantly, unbalanced moment loadings to the remaining column connections which were already loaded nearly to their ultimate strengths. Distress at the remaining three columns quickly followed.

Some increase in slab deflection at this point resulted from the separation of slab and column along the inclined cracks surrounding the column as the suspension action of the reinforcement crossing this crack was mobilized (see Figure 2.9b). Column area damage, which influenced the flexural strength of the slab, and the presence of compressed air in the loading chamber (see Section 4.1.2) also affected the deflection changes at this time (Figures A.9 and A.10).

Because of the separation along the shear crack, the concrete stresses in the bottom section of the slab at this location could not remain in the compressive state that previously existed. Therefore, the negative moment capacity of sections near the perimeter of the column capital was greatly reduced. A small moment capacity would remain because the resultant horizontal tensile force from the reinforcement crossing the shear crack was eccentric and was above the middepth of the slab. This decrease in flexural capacity in the negative column strip section, the section assigned 56 percent of the static moment (Figure 2.7a), was the primary cause of the local drop to the 14- to 16-psi level after the shear distress. Approximations of the expected decrease in capacity caused by this moment capacity loss can be made by assuming that the moment capacity was destroyed over the 16-1/2-inch width of the column capital. The column strip width is 27 inches. Thus, this computed loss is $16.5/27$ of 56 percent, or 34 percent, of the slab moment capacity. The 25.5-psi maximum load times 0.66 is 16.8 psi, approximately the observed value.

The reinforcement crossing the shear crack around the column was adequate to transfer the vertical forces between the slab and column in the manner expected (Section 2.3.4 and Figure 2.9).

The state of the in-plane forces acting in the slab was changed in a complex way with the development of the column shear distress. Prior to this event, the section at the shear crack location around the column was part of the slab still in a compression membrane state. After the shear distress occurred, all reinforcement crossing the inclined crack was at or near its yield stress in tension, resulting in a tension membrane zone around the column which pulled on the rest of the slab and caused compressive forces to increase in other areas of the slab in order to satisfy the requirements of equilibrium. At this stage, the additional depth of concrete in the drop panels around the column capital was of no help in supporting the load. Very possibly, portions of the drop panels fell from the slab surface with the occurrence of the shear distress.

In summary, the sudden drop in resistance displayed by the slab

resulted from the development of a punching shear failure at the columns which, while not fatal to the slab system because of the many bars crossing the inclined crack, did lead to a sizable reduction in the moment capacity of the slab in the vicinity of the columns. The slab system was effectively transformed from a solid slab supported on four columns to that of a slab containing four holes and hung at these locations by inclined cables placed over the columns extending through these holes in the slab.

5.1.3 Behavior at Large Deflection. The slab system continued to support load after the shear distress had occurred and very sizable deflections had formed. Figure 5.2 shows the limited deflection information obtained at two times after the drop in resistance. The deflections plotted in Figure 5.2a are for a time 21 seconds after the load drop and indicate that the deflections at this time varied considerably from a symmetrical pattern. It is possible that the shear distress started at one of the north columns. The deflections along the north row of panels exceeded the 7-inch gage capacity at the 16-minute elapsed time plotted in Figure 5.2b. Deflections at all locations except D1 (5.1-inch maximum), D3 (5.7-inch maximum), and possibly D7 exceeded the 7-inch transducer capacity; 7 inches represents over 2.5 slab thicknesses and $1/8$ of the column to column centerline spacing. Both the deflection data and the posttest photographs (Figure 4.1) show that the exterior panels in the north and east rows deflected more than the other panels.

At the large deflections reached, some tensile membrane response developed throughout the slab. The limited ability of the wall to anchor the slab at the top of the wall would act to limit the tensile membrane forces. However, portions of the slab adjacent to the wall could act as a deep horizontal beam anchoring tensile forces, an action more visible in the dynamic tests and described in Section 6.1.2 and Figure 6.2.

The soil pressure plots (Figure A.11) and wall strains (Figure A.18) indicate that the walls of the structure were pulled inward by the highly deflected slab after the load decrease. The wall crack patterns visible after the test also reflected this inward pulling of the walls.

The center of the west wall, which originally moved outward (see deflection trace D14 in Figure A.1), began to move inward as the slab deflections increased. The cracks observed at the bases of the east and north walls after the test indicated yielding of the welded wire mesh at this level. A horizontal loading at the level of the slab (i.e., neglecting soil forces) of approximately 60 pounds per inch of slab would cause this steel to reach its yield stress, assuming a vertical beam strip.

During the time of increasing deflections due to the development of the column shear distress, any benefits from tensile membrane action were small and were probably partially offset by increasing deterioration of the moment capacity of the slab in the column areas. The resistance remained nearly constant throughout this period.

5.1.4 Rupture of the Slab. The static test was terminated when the north row of panels ruptured (Figures 4.1 through 4.4) when the reinforcement at this location pulled out of the concrete slab. These bars did not break, but failed because of inadequate anchorage.

The conditions leading to anchorage failure of these bars were much different from and much more critical than those usually assumed in the analysis and design of reinforced concrete structures and used in Section 2.3.5 of this report. In retrospect, the anchorage failure mechanism appears simple. The draping action of the slab over the column after the nearly inevitable punching shear distress (see Figure 4.4a) resulted in the prying or spalling off of much concrete from the bottom slab surface, the yielding of most of the reinforcement near the column, and extensive cracking through the slab over a 7- to >10-inch-wide ring around the column where local tensile membrane action was greatest. Most of the spliced portions of bars located on the exterior panel side of the column were within this zone, as shown in Figure 5.3. Many of the top bars (Figure 5.3a) reaching yielding were spliced totally or almost entirely within extensively cracked concrete, a very severe condition. Most of the spliced portions of the bottom bars (Figure 5.3b) over a 20- to 25-inch width were completely exposed by the spalling occurring on the slab intrados (see Figures 4.10a and 4.12) and were thus totally ineffective. When these facts are considered, it is not at all surprising

that the slab ruptured where it did. It is believed that rupture would have occurred at greater slab deflections had deformed bars having better bond characteristic been used. Whether this increase in deflections would have been significant is not known. Because of the limited lateral resistance of the wall, the carrying capacity of the slab would not have increased appreciably had the slab held together longer.

The test slab response and the discussion above illustrate that the design of slabs for which large deflection alone does not constitute failure must consider types of behavior that are unknown or are of only academic interest for construction intended for conventional use only. The importance of the so-called "details" of the structure can hardly be overemphasized; nature excels in finding the weakest link.

After the rupture of the slab commenced, but before the sealing membrane ruptured and the load was entirely released, the two north columns were laterally loaded principally on one side by the still-intact center row of panels. The columns, being unable to resist this loading, tilted toward the center of the slab. This aided in the pulling of the reinforcement from the slab and the completion of the failure of the north row of panels.

The two zones of cracking extending from the center of the east and west sides of the slab to the nearest of the two south columns (Figures 4.2 and 4.4a) are thought to have been formed simultaneously with the southward movement of the two north columns at the end of the test.

The breaking of the few bars at the north wall negative moment section of the slab (Figure 4.8) probably occurred only after the slab had opened and the slab portions attached to the wall had begun to fold into the shelter space.

5.2 SOIL LOADING ON THE WALLS

The soil loading on the walls of a basement shelter will vary greatly with the soil type, compaction, and ground water conditions near the structure. Consequently, the soil-structure pressures measured in this test must be considered as indicative only of possible soil pressure magnitudes and pressure distribution patterns. The pressures

measured at several times throughout the static test are shown in Figure 5.4.

The maximum measured pressures from the fine sand backfill before the column shear distress developed were near the top of the wall and exceeded the applied pressure by nearly 40 percent. The pressure distribution decreased rapidly with depth during this loading stage and pressures of only a few pounds per square inch were measured near the level of the model floor except at the corners.

After the drop in slab resistance, the pressure distribution on the wall changed and reflected the inward motion of the wall. The pressures measured near the top of the wall and away from the corner decreased to almost zero, while the soil pressure in some areas near the slab corner increased. Near the end of the test, the five gages away from the corner of the wall indicated nearly zero or negative pressures, while the pressures at the bottom two gage locations near the northwest corner were at a maximum. The negative pressures could indicate a release of soil pressure acting on the structure prior to the test and resulting from the backfilling and consolidation operations. The -4-psi value shown in Figure 5.4d is of questionably high magnitude.

5.3 IN-PLANE FORCES ACTING ON THE SLAB

Both the soil forces on the wall and the geometry of the slab and wall model contributed to the in-plane forces acting on the slab prior to the inward movement of the wall.

The compressive force on the slab from the wall forces can be approximated over the center half of the slab by assuming the corresponding part of the wall to be a wide vertical beam loaded with the soil pressures measured at the column line (Figure 5.4b) and at the top with the slab yield moment. The pressure was assumed to vary linearly between the transducer locations and from the 25.5-psi applied load at the top of the wall to the 33-psi value measured at the 6-inch depth. These assumptions led to an estimate of 400-lb/in compressive loading on the slab.

Near the corners, the increasing soil loading on the wall is

carried by the wall perpendicular to the loaded wall, thus leaving less force acting on the slab. However, the slab in the vicinity of the wall is restrained by the parallel wall. Some compressive forces on the slab would result from this restraint.

5.4 PERFORMANCE OF THE SLAB IN SHEAR

5.4.1 Column Areas. The maximum loads measured by the four column load cells were 89 kips (NW column), 91 kips (SW column), 89 kips (SE column), and 88 kips (NE column). These column loads, which differ only slightly, total 357 kips. This load is 53 percent of the 668-kip load applied to the slab (area inside wall centerlines times 25.5-psi applied load), a slightly larger percentage than the 50.2 percent computed in Section 2.3.4. The loads supported on the columns exceed by 20 percent the load which would be computed by assuming that each column supports only the load from a contributing area one column spacing on a side.

The maximum unit shearing stress supported by the columns at a section $d/2$ from the column capital outline was 377 psi, or $5.83\sqrt{f'_c}$, as shown by the computations below:

$$\begin{array}{c}
 \text{total load with dead load} \\
 \downarrow \\
 91 \text{ kips} \left(\frac{26.6}{25.5} \right) - \underbrace{26.6 \text{ psi} (20.13 \text{ in})^2 \frac{\pi}{4}}_{\substack{\text{load on circular area} \\ \text{with diameter } (c + d)}} = 86.5 \text{ kips} = V_u \quad (5.1) \\
 \uparrow \\
 \text{load applied during test} \\
 \uparrow \\
 \text{maximum column load}
 \end{array}$$

$$v_u = \frac{V_u}{\pi(c + d)_d} = \frac{86.5 \text{ kips}}{\pi(20.13 \text{ in})3.63 \text{ in}} = 377 \text{ psi} \quad (5.2)$$

for

$$f'_c = 4,190 \text{ psi}, \quad \sqrt{f'_c} = 64.7 \quad \therefore \quad \frac{v_u}{\sqrt{f'_c}} = 5.83 \quad (5.3)$$

Although this analysis neglects the effects of the unknown moment also transferred to the columns, the value above is considerably greater than both the conservative $v_u = 3\sqrt{f'_c}$ value used in the design

(Section 2.3.1) and the $v = 4\phi\sqrt{f'_c}$ given in Paragraph 1701(d) of the 1963 ACI code (Reference 1) for allowable shear stress at this location.

The maximum unit shearing stress at the column capital face is used in several prediction formulas. The maximum value of the computed unit shear stress at this location was 459 psi, or $7.10\sqrt{f'_c}$. The ratio of column diameter to slab thickness, r/d , computed as one-fourth the column capital perimeter divided by the effective depth, d , is 3.58.

The formula presented in Reference 26 is accepted as being quite representative of the test results from isolated column specimens with r/d ratios up to 4, although recent tests (Reference 8) have indicated this formula may be high for $r/d \approx 4$. For a ratio of shear strength to flexural strength of 1.00, the formula in Reference 26 becomes:

$$v_u = \frac{V_u}{bd} = (9.75 - 1.125r/d)\sqrt{f'_c} \quad (5.4)$$

Where: v_u = ultimate unit shear stress

V_u = maximum shear supported by column

b = perimeter of a section located $d/2$ away from the column outline for a slab

For $r/d = 3.58$, this formula predicts a maximum $v = 5.75\sqrt{f'_c}$. The $7.10\sqrt{f'_c}$ value observed is 23 percent greater than this predicted strength.

The observed excess in shear strength over the quantities predicted above can be explained to some extent by considering two conditions present in the slab system: the compressive in-plane forces and the circular column shape.

At an r/d ratio of 3.58, results of recent tests (Reference 15) using slabs containing 1.0 percent flexural reinforcement showed that a round column may be about 30 percent stronger than a rectangular column or capital having the same perimeter.

The effect of the in-plane force can be estimated by using results of the few experiments on the shear strength of prestressed slabs and some assumptions concerning the in-plane forces acting within the slab system. For this calculation, the in-plane loading at approximately

maximum loading will be assumed to be 400 lb/lin in (see Section 5.3), although the stiffer area within the drop panel would attract in-plane forces from other sections.

The equation below was presented in 1958 (Reference 27) as a fit to data from prestressed slabs:

$$\frac{V_u}{bdf'_c} = 0.175 - 0.0000242f'_c + 0.000020 \frac{F_e}{S} \quad (5.5)$$

Where: $\frac{F_e}{S} = \frac{\text{effective cable force}}{\text{cable spacing}} = \text{effective concrete prestress}$

Using $f'_c = 4,190$ psi and $F_e/S = 400$ lb/lin in in this equation results in a predicted strength 11 percent above that for the case of $F_e/S = 0$.

Another equation representing experimental results from prestressed slabs is that given in Reference 28:

$$V_u = (0.360 + 0.30f_{ce})bd' \quad (5.6)$$

Where: f_{ce} = effective concrete prestress, ksi

d' = effective slab depth less collar recess, if any

For an f_{ce} value of 0.1 ksi (400 lb/in $\div$ 4-inch slab depth), a strength increase of 8.3 percent is predicted.

Thus, the in-plane forces can reasonably be credited with adding 10 percent to the slab shear strength. Together with the 30 percent increase possible because of the round column shape used, a strength of about 40 percent above that given by the ACI code and the other usual formulas applicable for nonprestressed slabs with square loaded areas could be expected. However, any moment acting on the column would act to decrease the column area shear strength.

An effect which would exist for the test model and not the prototype is the possible effect of model scale on shear strength. It is reported in Reference 29 that the apparent strength of concrete in models scaled by a factor of 2.5 was 7 to 10 percent above that indicated by the results of tests on 6- by 12-inch cylinders, and it is suggested

that model cylinders be tested to obtain the concrete strengths.

The shear distress around the columns took place as the flexural capacity of the slab was approached and the slab deflections were sizable (up to 1.30 inches). The behavior of the column areas in shear under these conditions, including the interaction between shear and moment and the sensitivity of shear strength to slab rotations in the column area, is not yet well understood. The approaching of the slab flexural strength very possibly directly contributed to the shear distress in the column areas.

The mechanism utilized to hold the structure together and transfer vertical forces from the slab to the column after distress of the area of the slab near the column has been discussed previously (Section 5.1.2). The maximum column loads measured after the drop in the structural resistance were 43 kips (NE column), 45-1/2 kips (NW), 56-1/2 kips (SW), and 60-1/2 kips (SE), from 47 to 66 percent of the maximum column load. These loads correspond to loads causing shearing stresses of from $2.76\sqrt{f'_c}$ to $3.88\sqrt{f'_c}$ on the critical section $d/2$ from the column face used in Equations 5.1 through 5.3.

Because the 1-1/4-inch additional concrete depth of the drop panel contributed little if any to the ability of the column areas to support shear after the shear crack completely formed, a comparison of the column loads above with the computed shear capacity of the slab, had the 2-3/4-inch depth been used throughout, may be of interest. The basic 1963 ACI code equation for shearing strength of $v_u = 4\sqrt{f'_c}$ predicts a capacity of such a column connection of 36.4 kips plus the load coming directly onto the column for a total column load of nearly 40 kips. Thus, the bottom reinforcement extended into the column area was successful in supporting a load at least as large as that predicted by the 1963 ACI code for an undamaged slab having an effective depth equal to the depth of concrete remaining around the column capital after the shear distress.

The total load measured on the four columns was 191 kips at 14 minutes elapsed time (21 seconds after load drop, applied load = 14.2 psi) and 182 kips at 16 minutes elapsed time (applied load = 15.8 psi).

These values are 51 percent (for 14 minutes) and 44 percent of the total load applied inside the wall centerlines. Thus, a smaller proportion of the load was supported by the columns after the resistance drop of the slab.

5.4.2 Shear at Wall-Slab Junction. The average shearing stress at the critical location near the wall (a section located one effective depth, d , inside the inner wall face) was 186 psi, or $2.88\sqrt{f'_c}$, at the time of the maximum slab resistance. Even though the shearing forces were not uniformly distributed and the average shearing stress was greater than the values given by the 1963 ACI code (Reference 1), no evidence was found of distress due to the shearing forces at the wall prior to the change in mode of shear transfer at the columns. The slab in-plane loading acted to enhance the shear strength along the walls.

Some evidence can be seen in the photographs presented in Chapter 4 that the rotations at the wall-slab junction were great enough prior to the termination of the test to lead to the crushing of the concrete compression zones at this location. This caused the slab section to lose its ability to transmit shear in the normal manner. The bottom reinforcement, which was extended into the wall of the model, was sufficient to prevent the slab from being torn from the wall. This reinforcement successfully supported the shearing forces by a suspension action identical with that around the columns.

5.5 SLAB REINFORCEMENT AND CONCRETE STRAINS

5.5.1 Tensile Reinforcement Strains Prior to Maximum Slab Resistance. The distribution of the strains measured on the tensile reinforcement at four values of the applied load (7.0, 13.8, 20.8, and 25.5 psi) are shown in Figure 5.5. Notable in these graphs are the high strains in the reinforcement at the wall and the low strains measured at the centerline of the structure.

The strains along the wall (A row of gages) increased suddenly from fairly low values at an applied load of 4-1/2 to 5 psi (Figure A.4). Cracking along the maximum moment location at the wall caused this jump. At the applied load of 13.8 psi (54 percent of the maximum load), the

A row strains were all at or above the yield strain of $1,300 \times 10^{-6}$ in/in.¹ These gages then indicated rapidly increasing strain and failed at an applied load of from 15 to 17 psi after reaching strains of nearly $5,000 \times 10^{-6}$ in/in.

Yielding was approached by all strain gages placed along the B line and was exceeded at locations in the half of the outer panel closer to the column line as the 15- to 17-psi applied load was approached.

The strains measured on the top reinforcement crossing the column line (C line) were largest in the drop panel area and in the center panel of the slab. Yield strains were reached or exceeded by gages placed in the drop panel and by those in the middle strip between the two columns. The strains measured closer to the wall were low (maximum at Gage C8 = 175×10^{-6} in/in) or negative (Gage C9). Reasons for these low strains include the in-plane compressive forces in the slab, which appear to have been sufficient to preclude flexural cracking at those locations close to the wall, and the small curvature of the slab in the direction of the instrumented reinforcement located along the axis of the trough-shaped deflection pattern, which as indicated by the deflection data (Figure 5.1), developed in the outer panels.

The strains measured on the bottom reinforcement crossing the centerline of the structure were much smaller than expected. Maximum values approaching 300×10^{-6} in/in were measured near the column line, with the strains gradually decreasing to compressive values closer to the wall. The same factors contributed to these low strains as to the low strains near the wall at the C line of strain gages. The strain distribution along the centerline of the structure changed considerably during the application of the last few pounds per square inch of load. The gages located in the middle panel and the adjacent column strip (Gages G1 through G5) displayed a sizable decrease in strain.

This decrease probably was caused by a redistribution of slab moments following the yielding of the reinforcement and the start of the yield-line formation near the B row of gages at an applied load of 18 to

¹ Tensile strain is considered to be positive.

20 psi. Because of the inability of this latter location to offer more moment capacity, moments from additional pressure resulted in a more rapid rate of increase of the negative moment sections over the column line than did previous loads. Some increase in the slope of the strain versus pressure curve for Gages C1 through C3 is evident in Figure A.14. This more rapid increase in the negative moments along the column lines would reduce the positive moment at the center of the middle panel faster than the increasing loads on this panel would increase these positive moments. This effect should be less for the outer row of panels because of instrumented reinforcement being parallel to the forming positive yield line. The leveling off and decrease of strains in the G row (Figures 5.5 and A.16) correlate well with the yielding in the B row, an expected relationship if the explanation above is valid.

Gages B10 and G10, placed on the bottom reinforcement at the inner and outer edges of the drop panel, indicated steadily increasing compressive strains reaching maximum values of nearly 500 in/in. This shows that the line of contraflexure around the column and inclosing the negative moment area extended beyond the drop panel outline.

The lack of adequate information concerning the compressive forces in the plane of the slab precluded meaningful transformation of the reinforcement strain distribution to experimental moment distributions. However, some comments on the moment distribution used in the slab design can be made. The moments developed in the slab close to and parallel to the walls were lower than assumed in the 1963 ACI code procedure, and the later yielding of the reinforcement in the column strip along the wall (Figure 5.5a) shows that the design moments along the wall were more evenly distributed than assumed. The design procedures in the 1971 ACI code (see Figure 2.14) is more realistic in the amount of moment assigned to edge strips containing beams and in the moment distribution along supporting walls. Both the 1963 and the 1971 code design methods underestimate the negative moment carried along the edge of the slab at the wall. The wall stiffness and soil loadings acting on it did result in much better fixity and moment development at this location than could be expected for most reinforced concrete floors, however.

5.5.2 Concrete and Compressive Reinforcement Strains. Maximum concrete strains of $-1,600$ to $-1,700 \times 10^{-6}$ in/in (compression) were recorded at the instrumented locations in the exterior spans except at Gage M6, where the strain was lower. No decrease or leveling off of the magnitude of the concrete strains with increasing applied loads that might indicate tensile membrane action was detected. Crushing strains were not reached at the gaged locations, which is consistent with the lack of visual crushing noted upon visual examination of these locations (Figure 4.4a).

In the column location, the strains adjacent and perpendicular to the column capital, Gages M1 and M8, both increased at a slowly but steadily increasing rate with load up to the maximum applied load (Figure A.19). The strains at these two locations were nearly equal and reached approximately $1,200 \times 10^{-6}$ in/in at the time of maximum load. The strains remained nearly constant (M1) or decreased slightly (M8) during the approximately 5-second period after maximum load and prior to the development of shear distress, a behavior hard to see with the time scale used in Figure A.3. The strain gage parallel to and 1/2 inch from the column capital displayed a similar increasing pattern up to a maximum load, with the strains being approximately one-third smaller. Unlike the strain perpendicular to the column outlines, the strain parallel to the column (Gage M2) increased very rapidly during the last few seconds preceding the large resistance drop and after the maximum resistance had been reached.

Strain gages were located at five positions on the concrete surface away from the column (Figure 3.43) and on both the top and bottom reinforcement very near the same point (Figure 3.40). The strains recorded at these locations were used to plot the strain profiles presented in Figure 5.6. The neutral axis location for bending calculated assuming a transformed section is shown in this figure. Although all experimental distributions were not linear as would be expected, the general low location of the neutral axis, especially at the center of the structure, is evidence of the existence of compressive in-plane forces.

The in-plane forces computed from these experimental strain distributions varied considerably from tensile values to sizable compressive

values. The results of these calculations are very sensitive to any small inaccuracies of the strains at any of the three levels of the section. Using the experimentally determined modulus of elasticity, E , for concrete of 3.3×10^6 psi and assuming the concrete cracking strength of $0.1 f'_c$, the computed in-plane forces at an applied load of 20.8 psi for locations a through e were (see Figure 5.6), respectively, 500-, 250-, 775-, 225- (assuming linear strain distribution from concrete surface to tensile reinforcement), and 375-lb/in compressive forces, an average of approximately 425 lb/in. Assuming this in-place force increased in proportion to the load up to the maximum applied load of 25.5 psi, a crude estimate of 520-lb/in in-plane compressive force results. This value is about 30 percent above the 400-lb/in value computed for the wall loading alone in Section 5.3. Both computations neglect in-plane forces resulting from the backfilling operation or from the dead load at the 2-foot depth of water above the slab.

5.5.3 Strains in the Slab Following Shear Distress near the Columns. The different methods employed by the slab to carry load following shear distress in the column areas are reflected by the different strain values and patterns recorded after this event.

The gages on the tensile reinforcement at the structure centerline (G series of gages) recorded very large increases in tensile strain immediately after the load drop, and all gages along this line except G1 and G9 failed at this time. Gage G1 jumped to above the yield strain, and Gage G9 reached this level in less than 2 minutes after the load drop. It should be remembered that Gages G8 and G9, close to the wall, indicated small compressive strains before the load drop. The moment carried by this section increased considerably with the change in slab geometry resulting from the damage to the column areas even though the applied load dropped approximately 45 percent.

The strains registered by gages along the B line of gages, located along the center of the outer panel span, recorded little change at the time of the load drop. The decrease in applied load must have almost matched the percentage increase in the moment that this section had to carry under the new slab geometry. As the slab deflections increased

and the soil loadings on the walls decreased as the walls were pulled in, tensile membrane behavior became more dominant and resulted in an increase in the tensile strains along the B line. All of the B gages had failed in tension by 3-1/2 minutes after shear distress occurred.

The C line of strain gages on the top reinforcement in the drop panel area or close to it (Gages C2 and C6 located on opposite sides within 5 inches of the drop panel) all failed in tension at or very soon after (Gage C3 only) the load drop, as would be expected because of the sudden transformation of this area to a tensile membrane zone (see Section 5.1.2). Two of the other three C line gages (C1 and C9) first jumped in a compressive direction, then showed increasing tensile strains until gage failure. Gage C8 showed tensile strains exceeding yield at the time of slab rupture and was the only strain gage operational at this time.

All of the gages on the top reinforcement at the wall (A line) had failed in tension prior to the load drop.

The gages placed on the compression reinforcement also reflected the changing conditions in the slab. Gage E1, at the slab center, failed while moving rapidly in a tensile direction immediately after shear distress. As stated earlier, most of the tensile strain gages at the slab center failed almost simultaneously with the load drop. The instrumented compression bars located at the outer panel centerline (Gages E4, E5, and E6) jumped in the tensile direction with the load drop. This was caused by the decrease in in-plane compressive stresses and the resulting rise in the neutral axis location, not by any sizable change in moments since the strains of the tensile reinforcement changed little at this time. Gages E4 and E6, located at the centers of panels, recorded rising tensile strains that stabilized, respectively, at and slightly above the yield strain in tension during the last few minutes of the test. Gage E5 was located at the center of the column-to-wall span and failed in tension near the end of the test.

The strain of the one compression bar at the wall that was instrumented (Gage E7) was affected little by the load drop, but jumped nearly $5,000 \times 10^{-6}$ in/in to past the yield strain in tension 7 seconds after

the load drop. A change in the shear-carrying mechanism of the slab at the wall from the normal means to a hanging of the slab from the wall by reinforcement crossing an open diagonal crack probably occurred at this time and would explain the strain jump. With further deflection, however, the effects of the rotation at the wall must have become more dominant, because the strain on this bar of compression reinforcement returned to compression and the strain gage eventually failed in compression.

The movement of the concrete strains measured at the three instrumented locations on the compressive face of the slab at the outer panel centerline (Gages M3, M4, and M6) varied from a slight increase in compression at Gage M6 to an 880×10^{-6} in/in decrease in compression at Gage M3 at the center of the corner panel. All three strains became more compressive as the deflections increased and reached values near the crushing strain. Maximum concrete strains near the end of the test ranged from $-2,350 \times 10^{-6}$ in/in at Gage M3 to $-3,150 \times 10^{-6}$ in/in at Gage M4.

At the center of the interior panel, the concrete strain first jumped in the compressive direction, then moved to a tensile strain of up to 80×10^{-6} in/in. At this location (Gage M7), the tensile membrane action evidently affected the concrete strain more than the increase in slab curvature accompanying the increasing slab deflection.

The three strain gages near the northwest column capital all ceased to function with the occurrence of the punching through of the column and the associated cracking around the column.

5.6 STRENGTH ANALYSIS OF THE SLAB STRUCTURE

The posttest condition of the slab structure indicated that the exterior panel slab yield-line pattern assumed in the design of the slab (Figure 2.8a) must have been developing before the occurrence of punching shear distress at the columns. The general yielding of the reinforcement in the slab adjacent to the wall and the measurement of strains near or above the yield strain all along the center of the outer panel and over the column line except near the wall indicate that the capacity of the slab as governed by this yield pattern was approached,

even though the load-deflection relationships did not flatten out before the sudden load drop.

The strength of the slab was considerably higher than that computed in the design stage, and some reconciliation of these values is necessary. In this section, the yield-line strength of the slab will be computed using the actual material properties. The positive yield line near the column line will be assumed to be located at the face of the column capital and the average effective slab depth will be used, two less conservative assumptions than were used in the design. The effect of the compressive in-plane forces will be introduced by using the yield moment of the section considering the interaction of moment and axial loading.

5.6.1 Interaction of Slab Moment Capacity and In-Plane Forces.

The moment capacity of a reinforced concrete section can be increased appreciably by the presence of in-plane compressive forces. This interaction is well appreciated for the case of columns and is equally applicable to beams or slabs. In most cases, axially loaded flexural members fall in the tension-controlled portion of the moment-load interaction curve, i.e., the moment loading predominates over the axial load to the extent that some reinforcement will yield in tension before any concrete will reach a limiting compressive strain, which is usually assumed to be $3,000 \times 10^6$ in/in.

This enhancement of flexural strength becomes greater when the reinforcement ratio decreases, the ratio of reinforcement strength to concrete strength is reduced, and the compressive loads are quite small relative to the balanced load, P_b , causing simultaneous reaching of reinforcement yield in tension and crushing of the concrete. An obvious extreme is an unreinforced beam that can carry sizable moments if prestressed or otherwise restrained.

The reinforcement ratios used in the slab of the shelter model tested were low (from 0.46 percent to 1.16 percent based on the average effective depth) and were often controlled by minimum reinforcement requirements. The reinforcement used had a low yield strength ($f_y = 36.9$ ksi). Due to these conditions, the strength of the model

structure was quite sensitive to the presence of in-plane forces.

The interaction diagram up to the balance point is shown in Figure 5.7 for sections at the wall and end span positive moment of the middle strip. The reinforcement ratio of this section ($p = 0.698$ percent) is approximately the same as the average used in the slab. The computed moment capacity of this section at the balanced condition, $M_b = 4,410$ in-lb/in of slab width, is 3.06 times the moment capacity with no axial load, $M_u = 1,440$ in-lb/in. The axial load corresponding to the balanced condition for this section is 4,660 lb/in of slab width, which is much higher than the in-plane loadings which could have been mobilized in the slab system.

Figure 5.8 presents interaction diagrams for all sections of the slab plotted up to an axial load of 1,000 lb/in. All reinforcement has been considered to be located at the average effective depth, and the effects of the "compression" steel have been included. For the range shown, the diagrams are nearly linear. The low reinforcement ratio and bar placement resulted in the computed strain of the compression steel being tensile or below the yield strain in compression throughout almost all of the range shown. Because the bottom mat of reinforcement in the drop panels was placed at the same level as the bottom mat throughout the rest of the slab, this reinforcement acted as tensile reinforcement. The interaction diagrams for moments causing the compression steel to act as the primary tensile reinforcement are shown for the two sections for which this information was needed for the yield-line analysis.

Some characteristics of the interaction diagrams are presented in Table 5.1. The values for M_b and P_b , the moment and load at the balanced point, varied little for the sections other than at the drop panel. Moments increased slightly at most sections when the effects of the compression reinforcement were included. The calculated enhancement of the flexural strength with the addition of 400-lb/in in-plane compression force varied from 13.6 percent (over the column) to 52.5 percent (negative moment in positive middle strip section).

5.6.2 Yield-Line Analysis. The assumed yield-line pattern is shown in Figure 5.9. The yield-line capacity was determined from

equations formulated by equating the external work done by the applied loads when the positive yield line was moved down a unit distance (sum for all segments of the forces acting on a slab segment times the downward movement of the center of gravity of this area) to the internal work done by the resisting moments acting along the yield lines (sum over all yield lines of resisting moment times rotation at the yield line). Because of symmetry, the yield-line strength can be computed by considering one-eighth of the slab. The location of the positive yield line was varied to minimize the predicted strength. The resisting moments were determined from the interaction curves (see Figure 5.8) for an assumed in-plane force.

The computed capacity of the slab system varied from 17.91 psi for no in-plane load to 28.53 psi for an in-plane loading of 1,000 lb/lin in of slab. This variation of slab resistance with in-plane loading is shown in Figure 5.10. The computed slab strength increases nearly linearly with in-plane loading at the rate of 1.06 psi per 100 lb/in of in-plane load.

The magnitude of the in-plane forces acting on the slab model at maximum load can only be estimated. Estimates of the increase of these forces during the static test, based on experimental results from that test, of 400 lb/in and 520 lb/in are presented earlier in this chapter (Sections 5.3 and 5.5.2).

The predicted slab strength at these magnitudes of in-plane forces are 22.3 and 23.5 psi. Although these values credit a strength increase of 24 and 31 percent to the effects of in-plane forces, they fall 16 and 12 percent short of the 26.6-psi strength observed. The observed strength of 26.6 psi corresponds to that predicted (Figure 5.10) for an in-plane loading of 800 lb/in.

Three other items must also be considered in examining the computed versus the observed flexural strength of the slab. First, small in-plane forces of an unknown magnitude existed in the slab prior to the test. These forces resulted from the backfilling operation, including the compaction of the backfill material, and from the active soil pressure resulting from the 2 feet of water placed in the test chamber prior

to the test. Second, even though the flexural capacity was approached, the yield-line pattern was not completely formed and the flexural strength was not completely mobilized. And finally, the yield-line analysis neglects several beneficial effects including that of strain hardening of the reinforcement, which may have occurred in this test at least along the yield line located at the inner face of the wall.

5.6.3 Comparison of the Design and Analysis Slab Strengths. The difference between the 14.60-psi design flexural strength of the slab, which was based on a yield-line analysis with in-plane forces credited with increasing the flexural strength 15 percent (Section 2.3), and the yield-line flexural strength of 17.91 psi resulting from the analysis presented in Section 5.6.2 and with no in-plane forces considered demands an explanation. The basic design flexural strength without in-plane effects of 12.70 psi ($14.60 \text{ psi}/1.15$) is 71 percent of the 17.91-psi analysis value.

The two yield-line flexural capacities differ because of the more conservative assumptions used in the design and because the model was constructed including material strengths that would be close to the expected values in actual construction for the given design strengths. More specific causes of the increase in the capacity given by the analysis of this section are listed below and demonstrate how the flexural strength of a slab can be increased considerably by the compounded effects of conservative assumptions.

The analysis was based on the reinforcement contained in the model which was proportioned to give the effect of bars placed at one-fourth the prototype spacings and having a yield strength of 44,000 psi, a value close to the average strength for Grade 40 bars. This strength is 110 percent of the minimum specified yield strength of 40,000 psi used in the design. For the lightly reinforced slab sections used, the resulting moment capacity was also increased approximately 10 percent.

The effective depth to the center of the layer closer to the compression face of 9.125 inches (for 11-inch-thick portions of the slab) was used in the design. The average effective depth of 9.50 inches (prototype) used in the analysis is 2.8 percent greater.

The distance from the positive yield line near the column to the yield line at the inner wall face in the analysis is 93.7 percent of that used in the design. This decrease resulted from moving the assumed yield-line location from the centroid of one-half the column capital area out to the face of the column capitals. Because moments are proportional to the square of the span, capacities of the slab given by the analysis should increase by nearly 14 percent ($1/(0.937)^2 = 1.14$).

Sections were considered to be singly reinforced in the design. The compression reinforcement was considered in the analysis. This reinforcement was placed far enough from the compression face to act as tensile reinforcement at ultimate flexural conditions (using $\epsilon_{cu} = 0.003$ in/in) and increased the section moment by an average of 5 percent (see Table 5.1).

Minimum reinforcement requirements of the 1963 ACI code used in the design governed reinforcement percentage in several locations of the slab (Figure 2.10). These increased reinforcement percentages were considered only in the analysis and acted to increase the slab strength by 4 percent.

The two slab strengths can be reconciled by multiplying the design value without consideration of in-plane forces by the magnitude of the five effects listed above:

$$12.70 \text{ psi} (1.10)(1.028)(1.14)(1.05)(1.04) = 17.9 \text{ psi}$$

Reinforcement yield ————↑

Effective depth ————↑

Yield-line placement ————↑

Compression reinforcement ————↑

Minimum reinforcement requirements ————↑

5.7 PERFORMANCE OF THE COLUMNS OF THE MODEL

The columns of the model were loaded to near their capacity during the static test and easily could have been the weakest parts of the structure.

The interaction diagram (axial load and moment) for the columns is

shown in Figure 5.11 along with the maximum measured column load, including dead load. A ϕ or understrength factor of 1.0 rather than the 0.75 used in design, a steel yield strength of 44 ksi (assumed for the Grade 40 reinforcement used), and an average concrete strength of 4,190 psi were used in obtaining this diagram. The intersection of the observed maximum column load and the eccentricity of the column load computed in Section 2.3.6 plot very close to the boundary of the interaction diagram line describing the column strength. The moment or eccentricity acting on the column at failure could not be determined from the test data.

The spiral reinforcing of the column was designed to satisfy the 1963 ACI building code and was not heavy enough to produce any appreciable increase in the column strength after the cover or shell of the column outside the spiral spalled off, an increase which would result from the confinement of the core concrete by the spiral. The use of heavy spirals to obtain increased axial capacity after spalling of the shell was rejected because the undesirable effects of debris creation, the psychological impact of the shell spalling, and the accompanying decrease in the moment-carrying capacity of the column would more than offset any small savings of materials.

The splitting or vertical cracks visible in the bottom portions of the columns (see Figures 4.6, 4.10, and 4.11) attest to the high column loads, but may also be due in part to the reaction load being concentrated near the center of the column in spite of the 3/4-inch-thick steel bearing plate placed below the column.

The test results emphatically point out the fact that slab flexural strength, and to a lesser extent, the shear strength, can be greatly increased by the presence of slab in-plane forces, but that these forces have no beneficial effects on the column strength. Unless the columns supporting the slabs are designed conservatively and the increased loads resulting from the effects of in-plane forces on the slab strength are considered, the strength of the slab may never be developed. Rather, the structural strength and performance of the slab can be limited by premature failure of the supporting columns.

5.8 BEHAVIOR OF THE WALLS OF THE MODEL

The reinforcement strains measured at the midheight of the wall, both at a column centerline and at the structure centerline, were small prior to the punching distress around the columns. Strains were generally below 200×10^{-6} in/in.

Because the perimeter of the slab rested on the wall, the wall carried an appreciable vertical load. The excess in applied slab load above the sum of the measured column reactions equalled 520 lb/lin in of wall, assuming the wall to be uniformly loaded.

An outward motion of the center of the west wall, which increased roughly in proportion to the applied load and reached a maximum of approximately 0.10 inch, was measured during the loading period prior to the slab resistance drop (Figure A.10, Gage D14). The moment from the slab acting at the top of the wall was sufficient to deflect the wall into the backfill around the structure at midheight in spite of the soil forces acting to deflect the wall into the structure.

The contraflexure line between the moments near the top of the slab causing tension on the outer face of the wall and the moments arising from the soil loading and causing the tensile face to be the inside wall surface was near the strain-gaged location and explains in part the low strains measured on the wall reinforcement. The recorded wall strains (Figures A.2 and A.18) imply that the midheight location at the structure centerline was slightly above this contraflexure line while the location at the column centerline was slightly below.

The behavior of the wall began to change gradually after the load drop because of the inward forces acting at the top of the wall accompanying the large slab deflections. The deflection gage measuring horizontal motion of the wall center began a rapid inward motion about 3 minutes after this load drop (Figure A.1, trace D14) and reached an inward deflection of 0.14 inch at the end of the test. The outer wall layer reinforcement strains also reflected the inward pulling of the top of the wall and increased in a tensile direction approximately to the yield strain of the welded wire fabric reinforcement.

The wall adequately served its intended function as a wall per se

and as a method for allowing the negative slab moment capacity to be reached around the perimeter of the slab. The limited ability of the wall to provide horizontal restraint to the slab during the tensile membrane stage accompanying large deflection was discussed in Section 5.1.3. A wall with sufficient strength to serve this anchoring function, even if designed as a retaining wall, appears to be impractical unless massive counterforts or interior walls perpendicular to the basement wall can be placed at fairly close spacing along the wall.

5.9 COMPARISON OF THE STATIC TEST RESULTS WITH THE RESULTS OF A 1/24-SCALE STATIC TEST

A 1/24-scale model of the prototype described in Chapter 2 was constructed and tested at the University of Illinois as a part of a National Science Foundation Undergraduate Research Participation Program (Reference 30). The model was reinforced with annealed 20-gage bright basic wire having a yield strength of approximately 60 ksi and placed at 1/24th the spacings shown in Figures 2.11 and 2.12. The 20-gage wire size (0.0348-inch diameter) is slightly larger than 1/24th the prototype No. 6 bar ($0.75 \text{ inch} \div 24 = 0.03125 \text{ inch}$). Sand concrete with a design compressive strength of 4,000 psi was used.

The 27-1/2-inch-square model consisted of the slab, walls, and four interior columns cast monolithically and supported on a base slab. The procedures used to accurately form and position the two mats of reinforcement in the 0.47-inch-thick slab included spot-welding the mats and are presented in Reference 30.

The model was placed in a 4-foot-diameter pressure chamber that was backfilled to the top surface of the slab with Ottawa standard sand. Bottled nitrogen gas provided the loading and resulted in a very sudden failure.

The 1/24-scale model failed under a loading of 34.8 psi. Two adjacent columns broke close to the base of the column capital, and the six panels adjoining these columns failed entirely in the sense that slab areas either dropped to the floor or were folded down against the wall. Shear distress developed around only one of the two remaining columns. Cracks in the bottom slab surface suggested that the slab was seeking a

flexural yield-line pattern similar to that shown in Figure 5.9, but also that the flexural capacity of the slab had not been fully developed. Cracking in the remaining slab areas was not extensive, and crushing of the concrete did not occur at the potential yield-line locations. Difficulties with the five deflection transducers employed resulted in a paucity of deflection data. From this limited data, the panel deflections at failure appeared to be approximately 0.05 to 0.07 inch.

This model supported 135 percent of the maximum resistance of the 1/4-scale model used in the study reported herein. Failure was much more catastrophic for the smaller model and apparently occurred at a lower relative deflection. This would be expected for at least two reasons. First, the magnitude of the pressurized gas loading would not be appreciably reduced by deflection of the slab and the large deflection portion of resistance curve resulting from the 1/4-scale test, in which water pressure was used, was never stable in the pressurized loading arrangement. Second, the increased reinforcement strength and scaled size in the smaller model would make the slab shear and column strengths more critical than the slab flexural strength.

The larger shearing stress around the columns for the 1/24-scale model could have been the result of both a higher tensile strength for the small-aggregate concrete and the favorable effects of the higher flexural strength of the slab. The columns of the 1/4-scale model were slightly larger than the 1/24-scale (1.00-inch diameter versus $22 \text{ inch}/24 = 0.917\text{-inch}$ diameter) but contained a smaller reinforcement ratio. The columns must have been loaded very close to their ultimate capacity, although no experimental evidence was found to suggest that distress of the columns initiated the failure of the slab system.

The yield-line flexural capacity of the 1/24-scale model ignoring in-plane loading effects is reported in Reference 30 as 27.0 psi, as compared with 17.9 psi for the 1/4-scale model. The smaller model failed at 129 percent of the computed flexural yield-line strength not considering in-plane forces, whereas the 1/4-scale model reached 148 percent of its computed capacity of 17.9 psi.

Reasons for the larger increase above the conventional yield-line

flexural strength for the larger model include the potential flexural strength being more closely approached in the model with the lower reinforcement strength. Also, the percentage increase in the moment capacities resulting from the interaction of moment capacity and the in-plane compressive forces resulting from the soil loading on the walls would be larger for the 1/4-scale model because of the lower reinforcement strength and moment capacity (see Section 5.6.1).

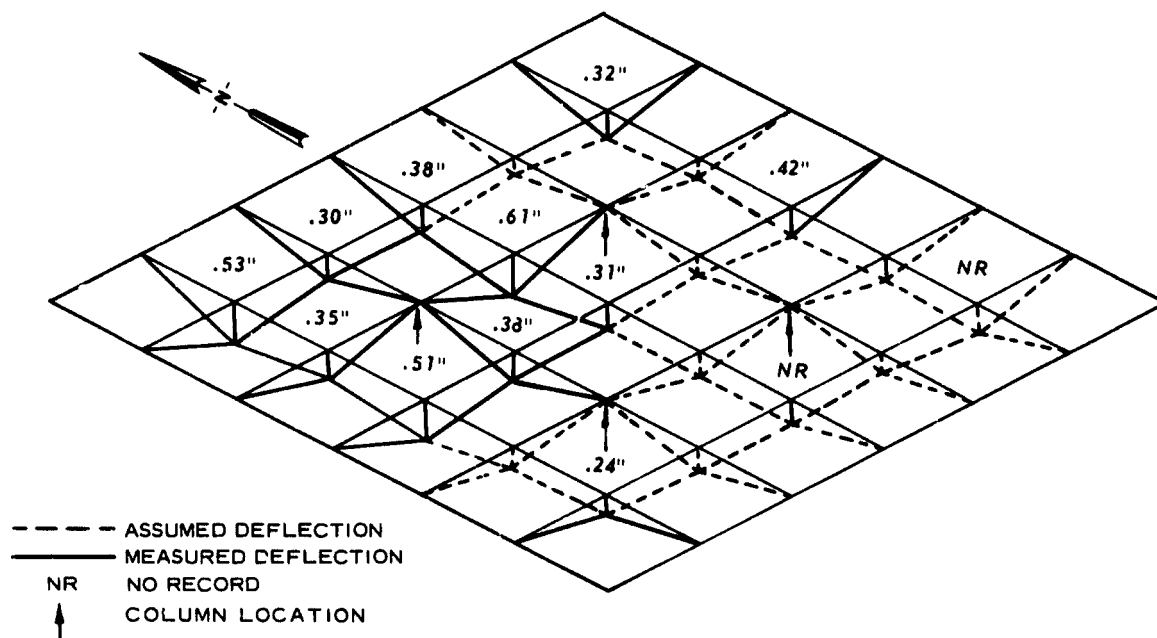
The favorable interaction of moments and axial loads for tension-controlled reinforced concrete sections can be used to explain most of the increased capacity of the slab observed in both model tests. The observed strength of 34.8 psi for the smaller model corresponds to the yield-line strength, including the slab in-plane effects, resulting from a uniform 40-psi loading on the wall surfaces. This average pressure is slightly above that which would be expected considering the pressure distributions measured on the wall for the 1/4-scale test. Figure 5.10 shows that the flexural strength of the 1/4-scale model was also slightly higher than that predicted from a yield-line analysis using the probable in-plane slab forces.

TABLE 5.1 DATA FROM MOMENT-AXIAL LOAD INTERACTION DIAGRAMS FOR SLAB SECTIONS

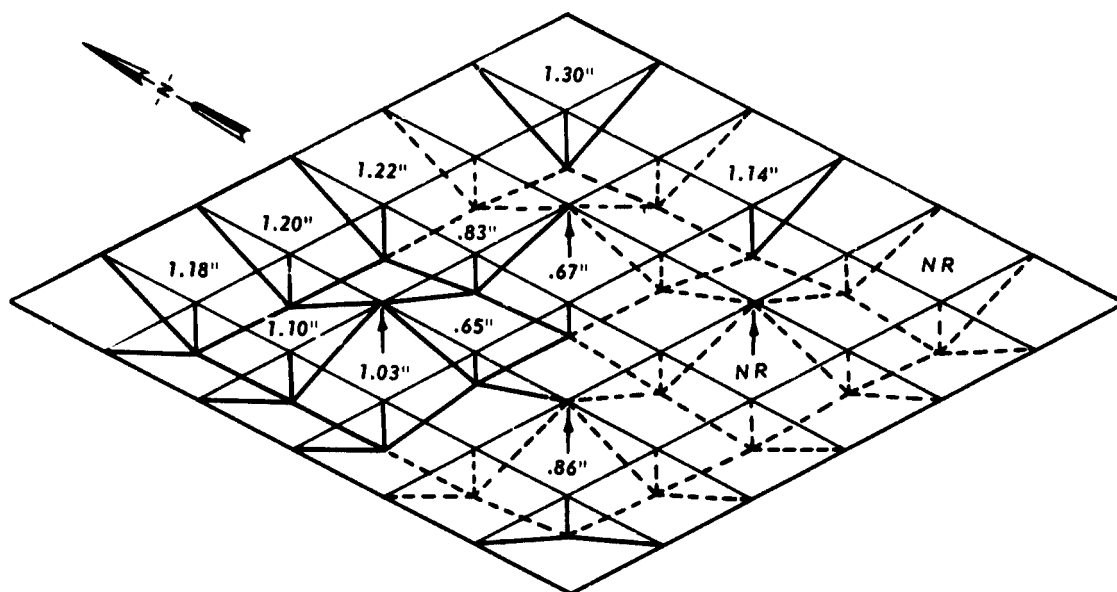
Data given are for 1-inch width of slab section locations and reinforcement percentages shown in Figure 5.8.

(1) Section ^a	(2) M_u when $P = 0$ and p' Is Neglected	(3) M_u when $P = 0$ and p' Is Included	(4) Ratio of M_u with p' to M_u without p' , (3)/(2)	(5) M_b	(6) P_b	(7) M_u when $P = 200$ lb/in	(8) Percent Increase Over M_u when $P = 0$ and p' Is Included	(9) M_u when $P = 400$ lb/in	(10) Percent Increase Over M_u when $P = 0$ and p' Is Included	(11) M_u when $P = 1,000$ lb/in	(12) Percent Increase Over M_u when $P = 0$ and p' Is Included
	in-lb/in	in-lb/in		in-lb/in	lb/in	in-lb/in		in-lb/in		in-lb/in	
1	944	1,005	1.066	4,165	4,850	1,220	21.4	1,450	44.2	2,095	108.3
2	1,138	1,185	1.041	4,230	4,740	1,415	19.5	1,625	27.1	2,265	91.2
3	1,248	1,300	1.041	4,300	4,710	1,525	17.2	1,735	33.4	2,310	81.5
4	1,842	1,865	1.013	4,710	4,570	2,075	11.2	2,295	23.0	2,910	56.0
5	1,171	1,235	1.054	4,300	4,780	1,455	17.9	1,665	34.9	2,300	86.2
6	3,770	4,460	1.181	8,380	7,240	4,775	7.1	5,065	13.6	5,905	32.4
7	1,106	1,160	1.049	4,230	4,770	1,380	19.0	1,600	38.0	2,235	92.8
8	1,399	1,440	1.030	4,410	4,660	1,665	15.0	1,875	30.2	2,500	73.7
9	1,842	1,865	1.012	4,810	4,670	2,080	11.7	2,295	23.1	2,920	56.6
10	1,243	1,300	1.041	4,270	4,680	1,525	17.2	1,735	33.4	2,360	81.5
11	2,270	2,280	1.004	4,900	4,370	2,490	9.2	2,700	18.4	3,300	44.9
8a	--	820	--	--	--	1,040	26.8	1,250	52.5	1,890	130.5
10a	--	1,190	--	--	--	1,390	16.9	1,605	34.9	2,225	78.5

^a M_u computed using Equation 16-1 of Reference 1.

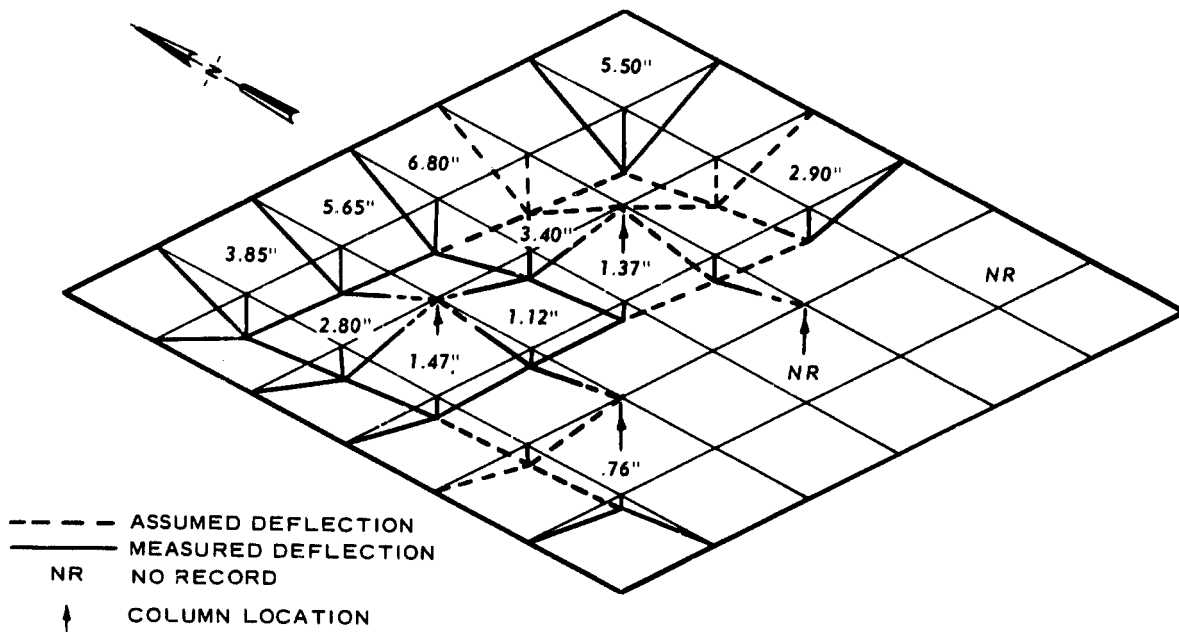


a. SLAB DEFLECTIONS AT $t = 8$ MINUTES, APPLIED LOAD = 13.8 PSI

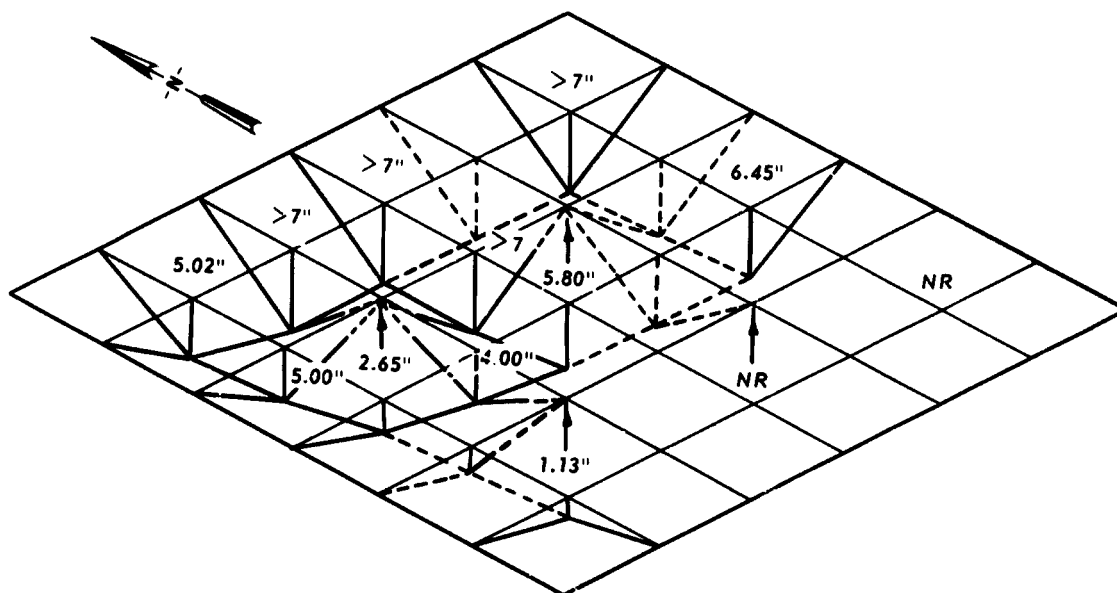


b. SLAB DEFLECTIONS AT $t = 13.6$ MINUTES, APPLIED LOAD = 25.5 PSI

Figure 5.1 Slab deflections during ascending resistance branch.

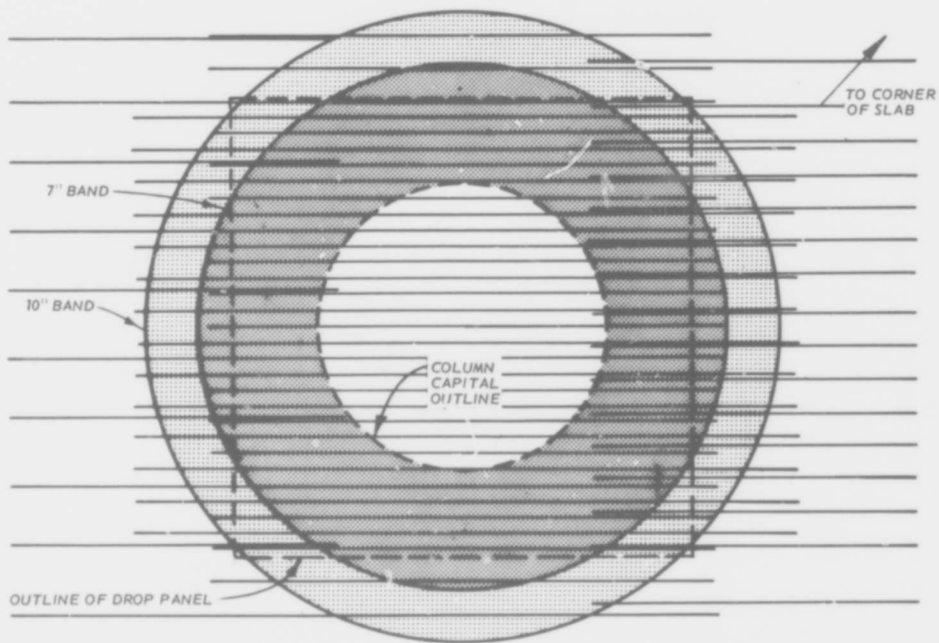


a. SLAB DEFLECTIONS AT $t = 14$ MINUTES, APPLIED LOAD = 14.2 PSI

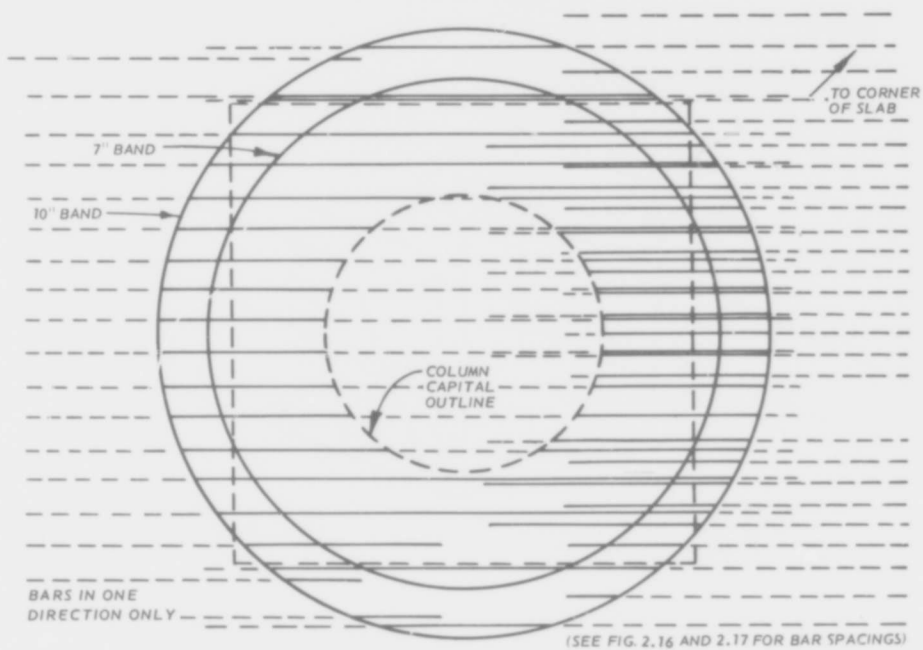


b. SLAB DEFLECTIONS AT $t = 16$ MINUTES, APPLIED LOAD = 15.8 PSI

Figure 5.2 Slab deflections after shear distress.



a. TOP REINFORCEMENT IN 7- OR 10-IN.-WIDE TENSILE ZONES



b. BOTTOM BARS EXPOSED BY 7- OR 10-IN.-WIDE BANDS AROUND COLUMN CAPITAL

Figure 5.3 Effect of column tensile membrane area on bar anchorage.

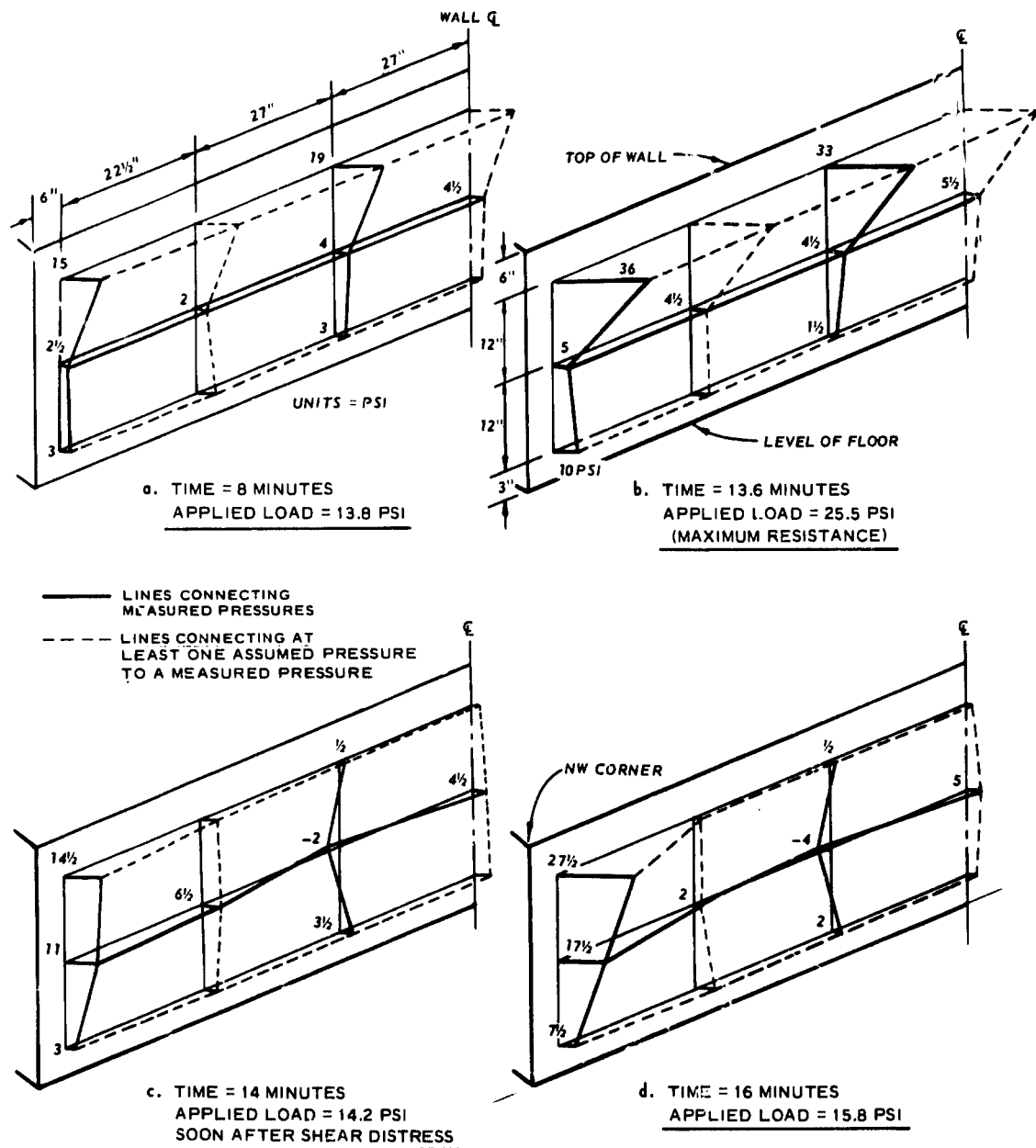


Figure 5.4 Soil pressure action on west model wall.

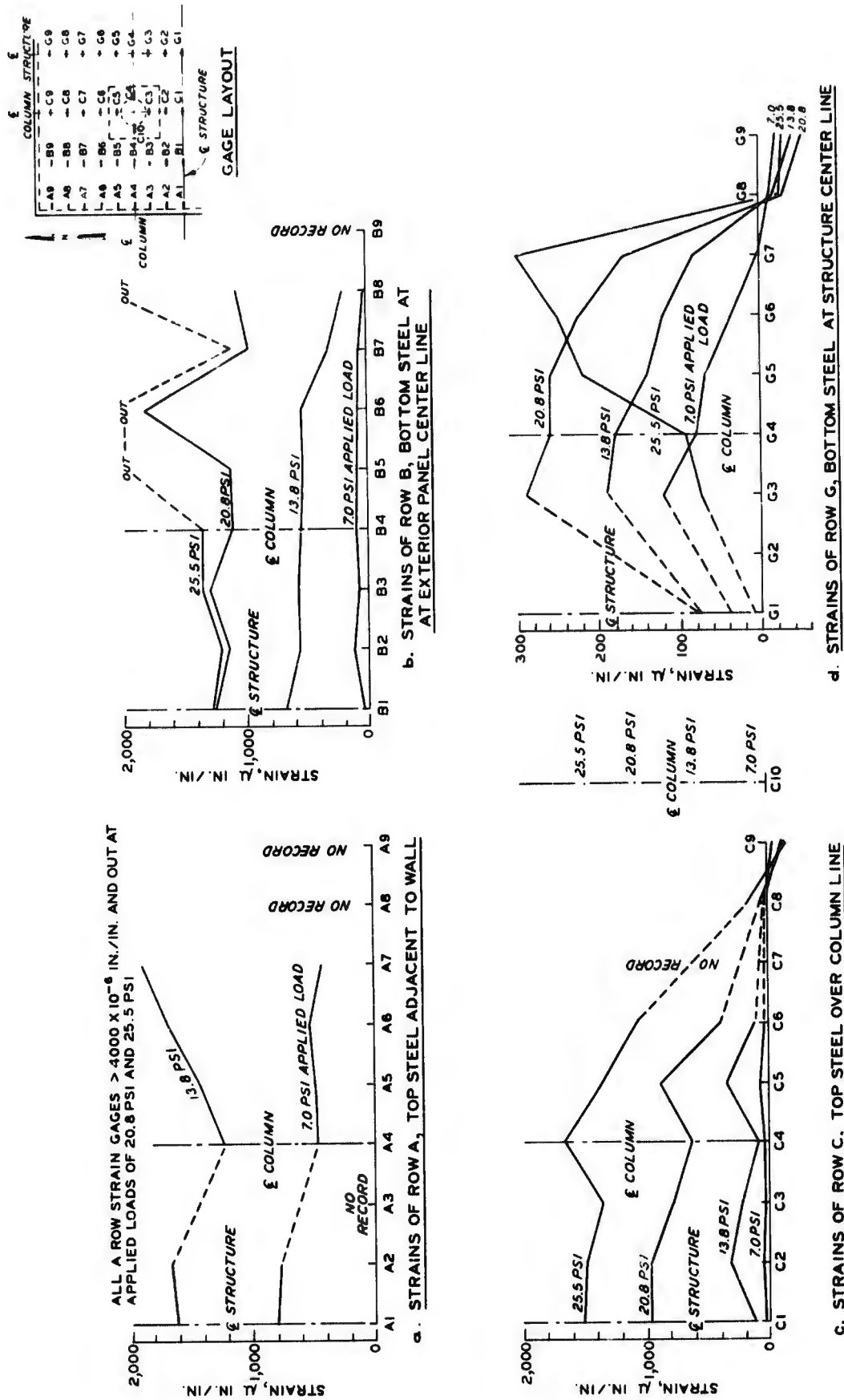


Figure 5.5 Reinforcement strains prior to resistance drop.

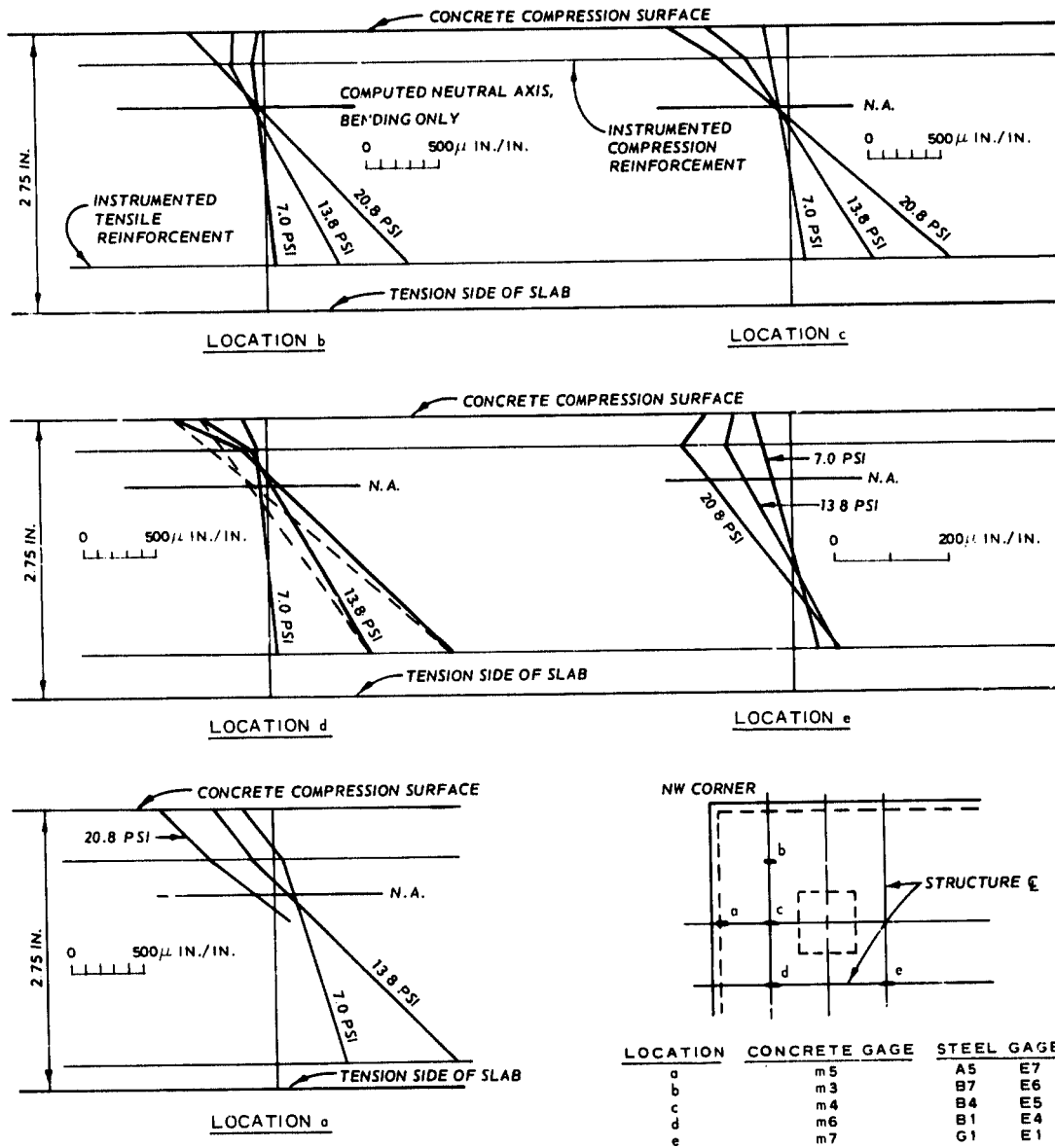


Figure 5.6 Experimental strain profile plots.

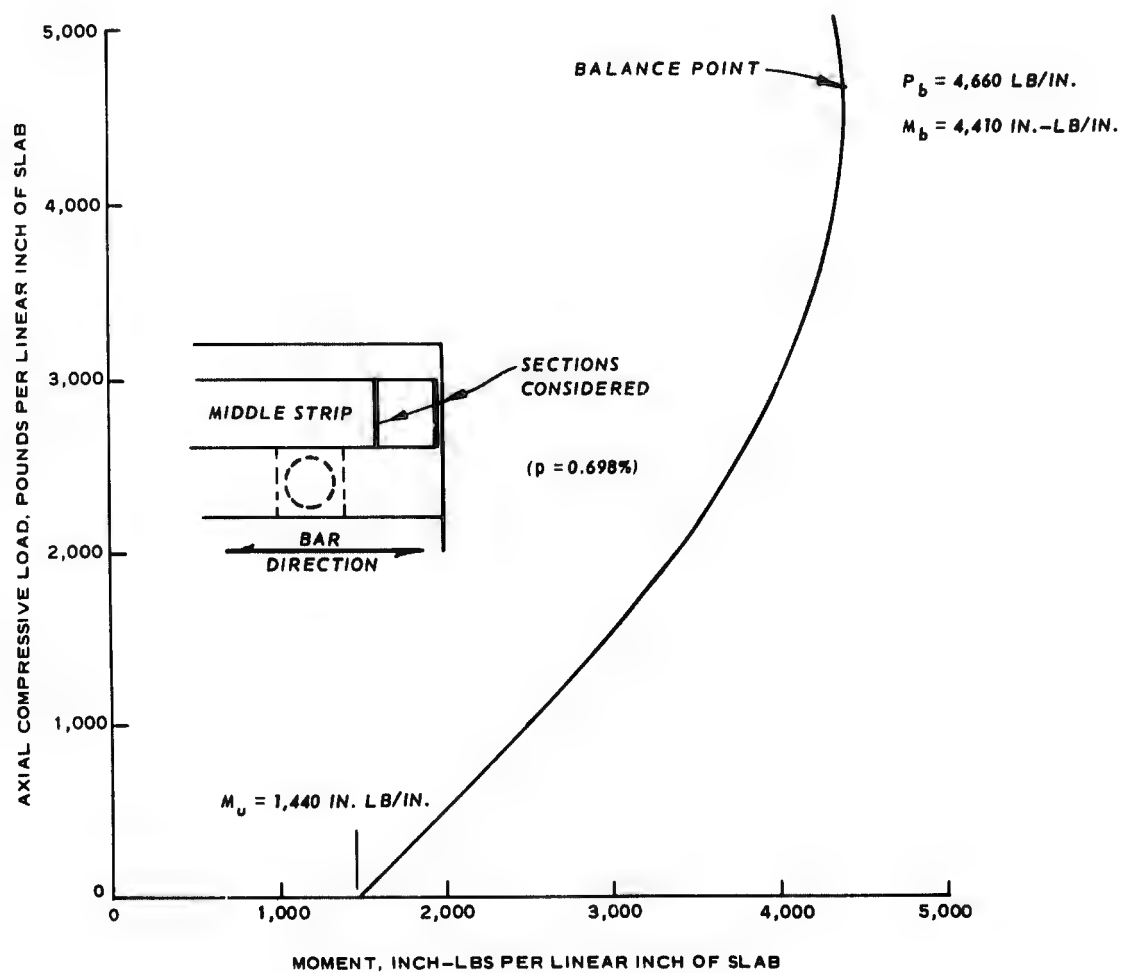


Figure 5.7 Typical axial load-moment interaction diagram.

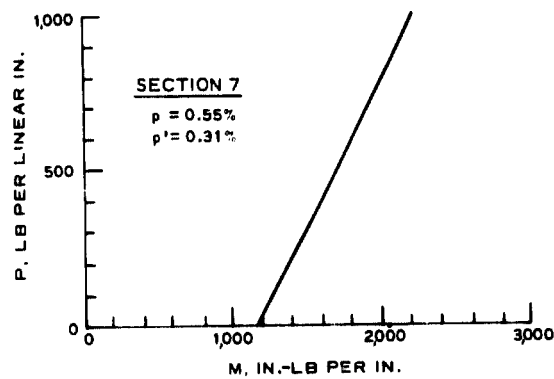
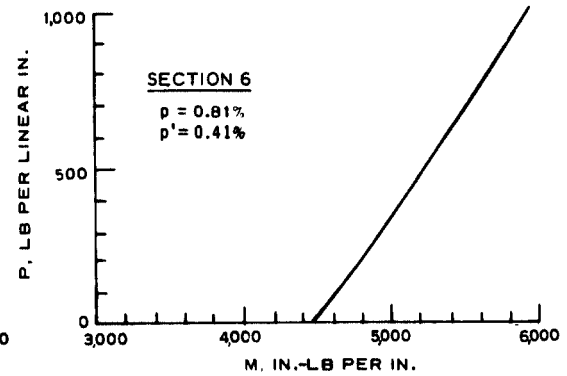
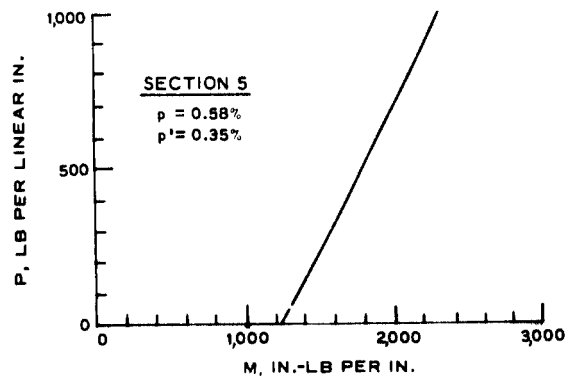
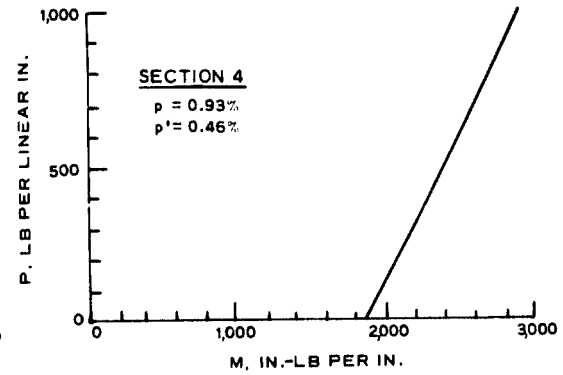
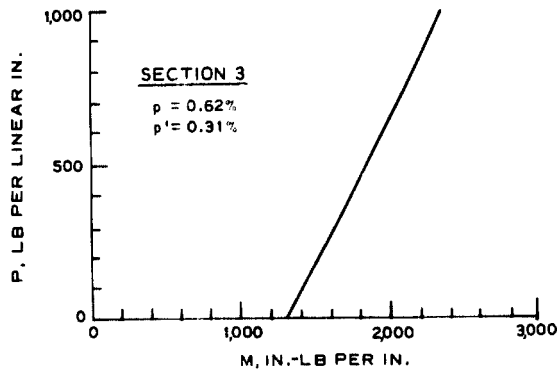
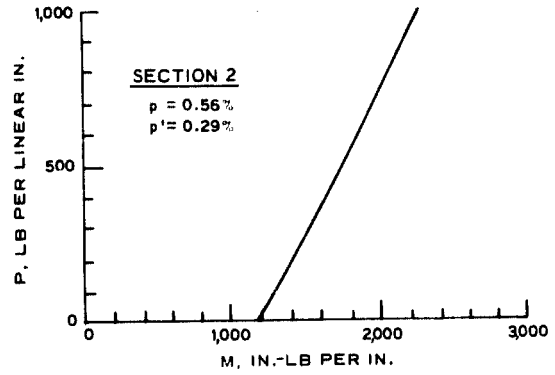
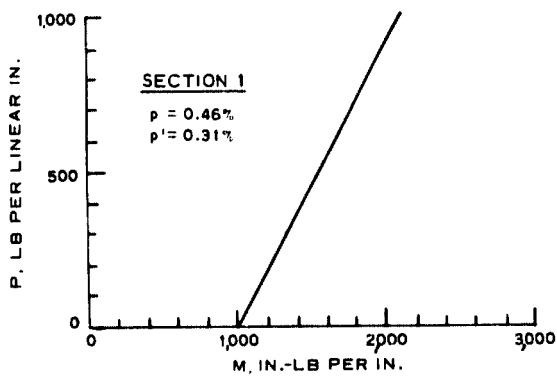


Figure 5.8 Partial interaction diagrams for slab sections (sheet 1 of 2).

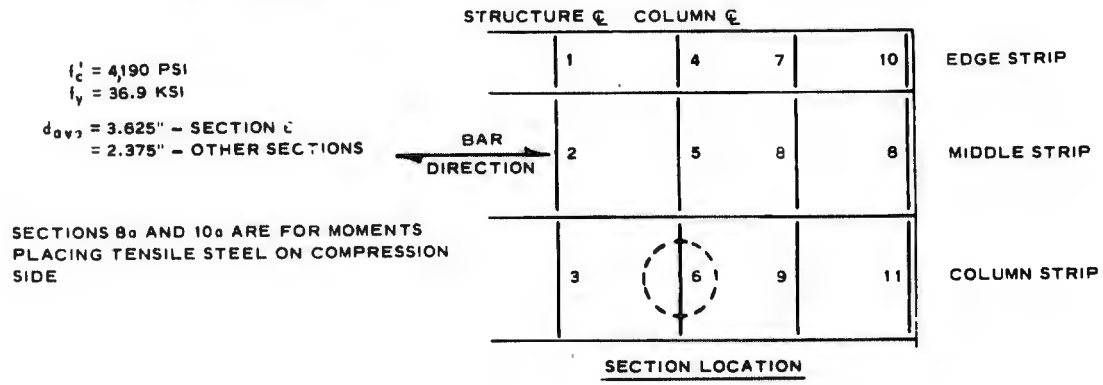
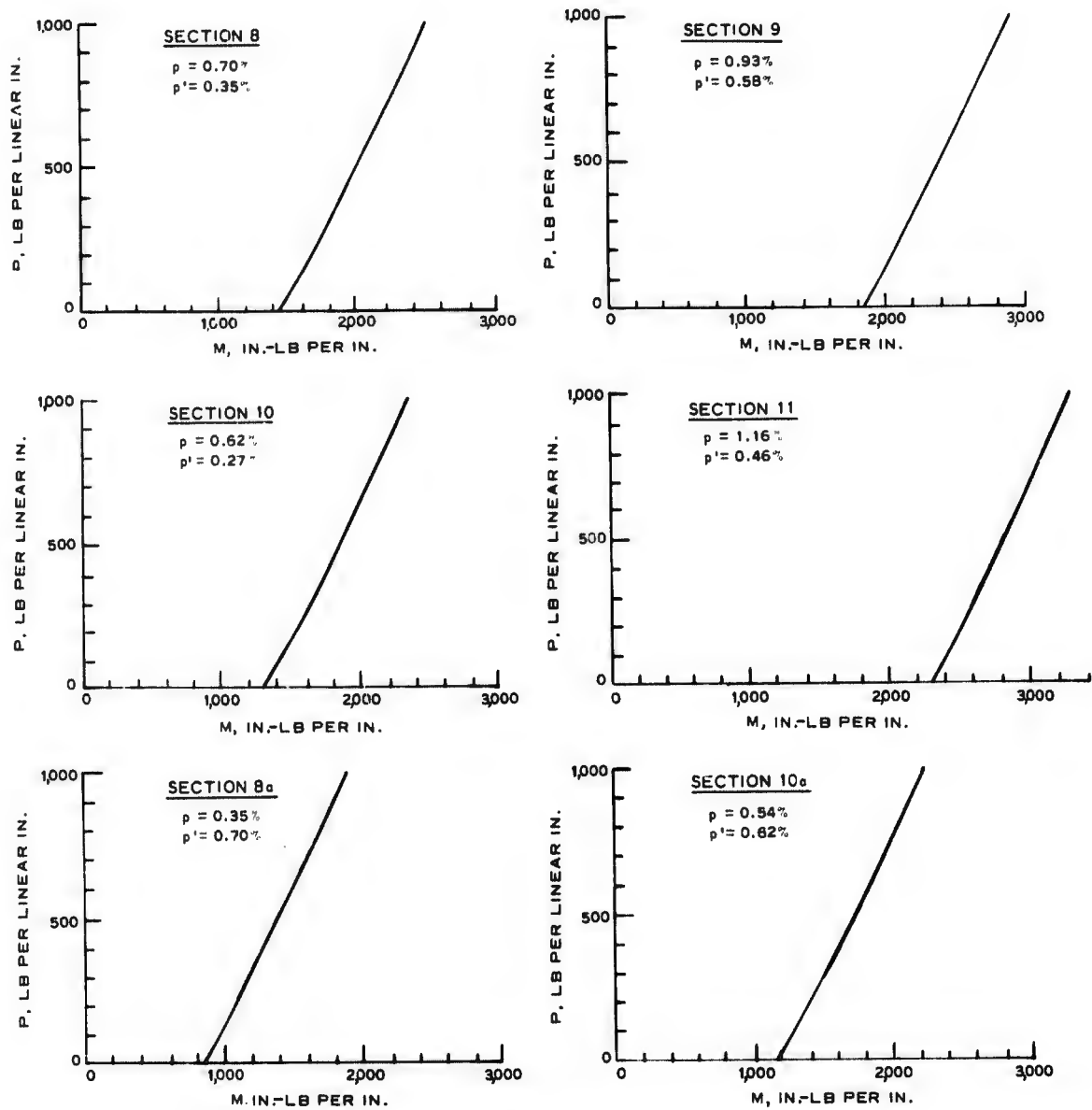
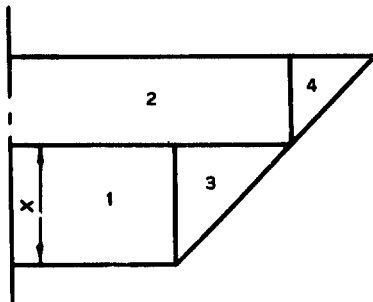
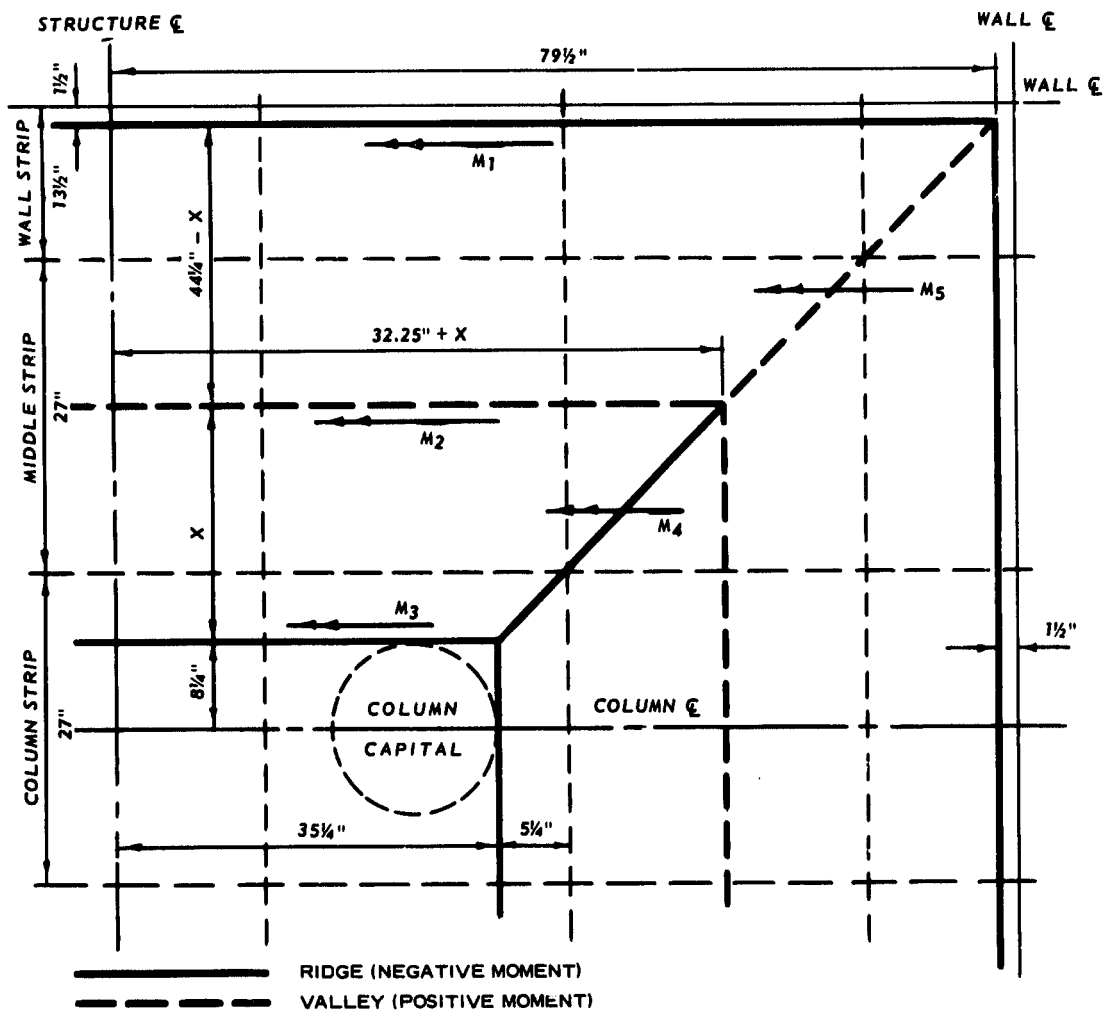


Figure 5.8 (sheet 2 of 2).



AREAS USED TO DETERMINE
EXTERNAL WORK

$$M_1 = 27(M_{11}) + 40.5(M_8) + 12(M_{10})$$

$$M_2 = 27(M_9) + (8.25 + X)M_8$$

$$M_3 = 13.5(M_5) + 21.75(M_6)$$

$$M_4 = 5.25(M_6) + (X - 5.25)M_{8a}$$

$$M_5 = (32.25 - X)M_8 + 12(M_{10a})$$

WHERE M_i = MOMENT PER UNIT WIDTH AT
SECTION I (FIGURE 5.8)

Figure 5.9 Yield-line pattern.

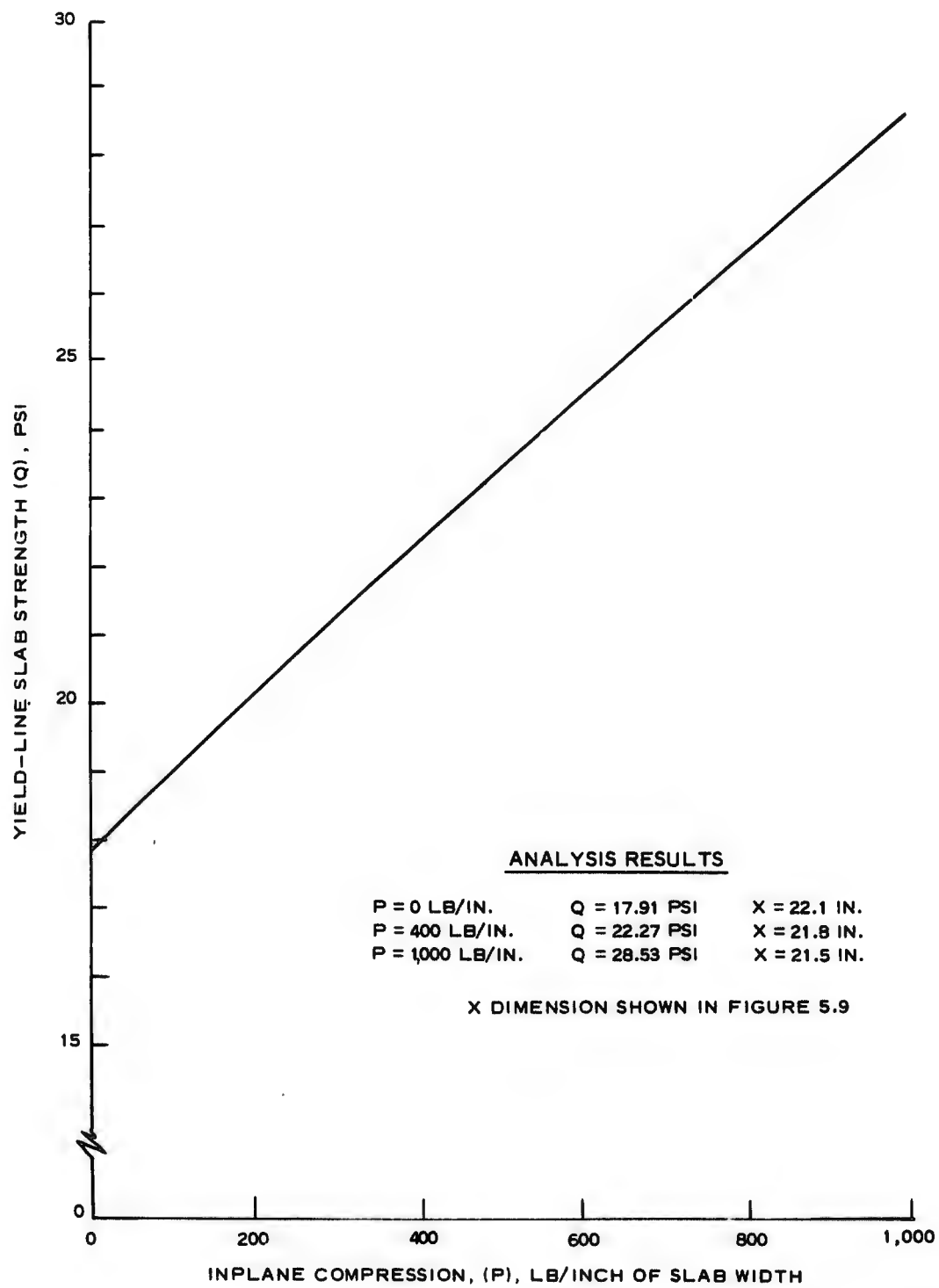


Figure 5.10 Effect of in-plane forces on slab strength.

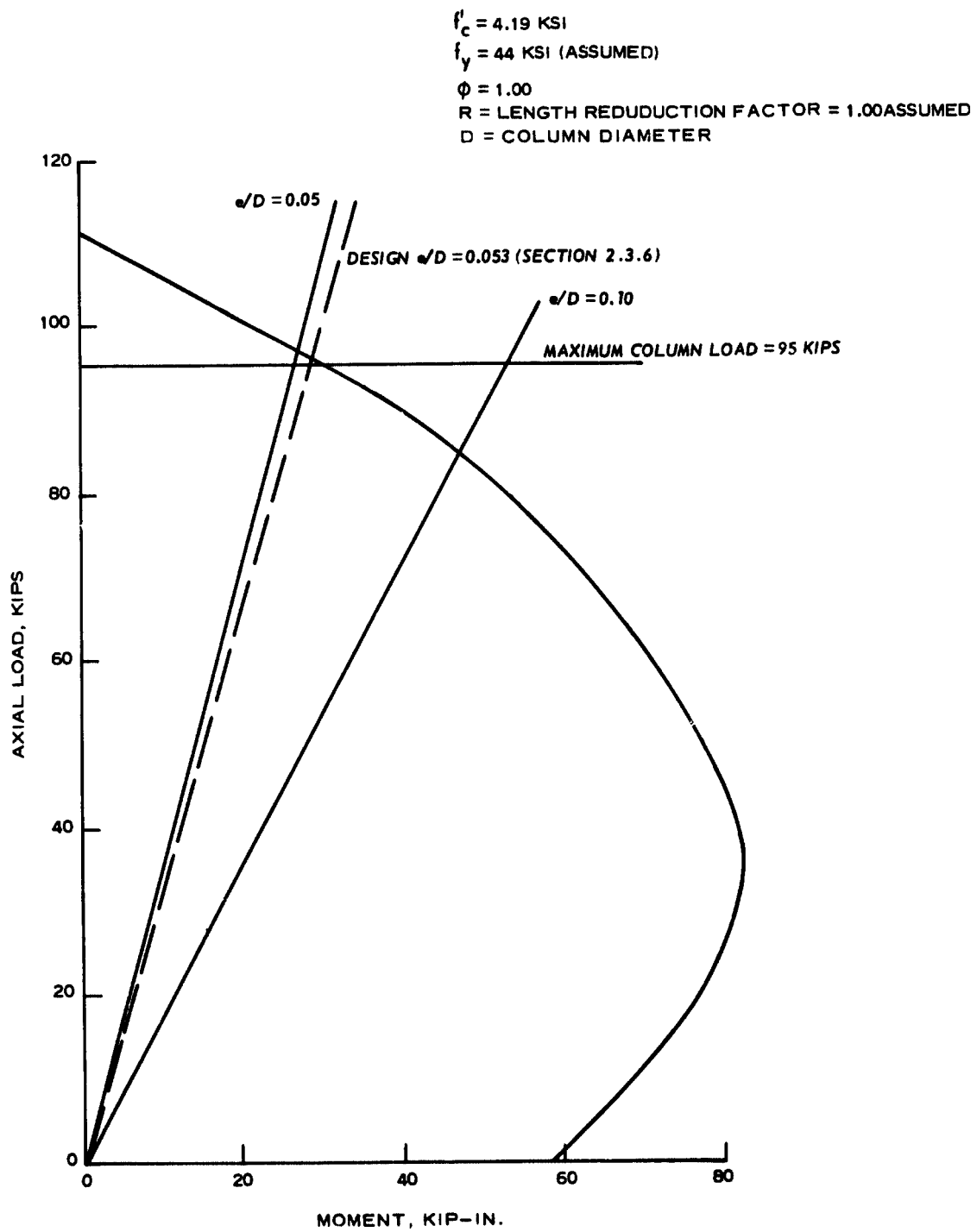


Figure 5.1.1 Load and interaction diagram for the columns of the model.

CHAPTER 6

DISCUSSION AND ANALYSIS OF DYNAMIC TEST RESULTS

6.1 RESPONSE, STRENGTH, AND FAILURE MODE OF THE SHELTER MODEL

6.1.1 Dynamic Test I. Only slight damage to the shelter model resulted from Dynamic Test I, which had a 19.7-psi maximum average overpressure level and a rise time of 8.5 msec (Figure 4.18). The small permanent deflection of approximately 0.15 inch maximum and the measurements from the strain gages placed on the tensile reinforcement both indicated that the response of the model was largely but not entirely within the elastic range. Yielding of the reinforcement in the slab was detected only along the wall and at one location along the centerline of the structure (Gages A1 through A6 and G3, Figures B.10 and B.12).

The majority of the measured slab strains and the column loads reached their maximum values at 20 ± 4 msec after the detonation. The time period from 20 to 30 msec was the most critical for the slab system; during this time, the slab was successful in stopping its downward movement and passed the test of surviving the dynamic loading. The velocity records, obtained by integration of the acceleration records, also show that the rate of downward motion decreased from a maximum at about 13 msec to nearly zero at approximately 20 msec. The soil pressures acting on the instrumented wall of the shelter model reached maximums at approximately 26 msec, slightly later than the slab instrumentation. After these peak values were reached, the output of the strain and load transducers was composed of a usually dominant and slowly decreasing quasistatic response to the decreasing overpressure with a complex and usually smaller dynamic component superimposed. In many cases, one frequency was predominant. This dynamic component resulted from the vibrations of the structure and testing chamber excited by the blast loading and had decreased only slightly at 200-msec elapsed time.

Some information about the vibration characteristics of the structure can be determined from these periodic components of the output from several transducers during the first dynamic test. The periodic motion

was especially noticeable in the records from the column load cells, tensile reinforcement strains, and soil pressure gages. The period of this cyclic component was determined for the transducers for which this motion was most evident and regular by measuring the elapsed time for five to ten cycles usually starting at 40 to 45 msec into the test. The results, presented in Table 6.1, showed three obvious frequencies, rather than one, in the shelter response.

The average period for the applied loads and column loads was approximately 24 msec (ranging from 22.2 to 25.2 msec), while the dominant period detected by the tensile reinforcement strain gages except those located along the column centerline (C line of gages), one location near the wall on the B line of gages, and the gage located at the center of the structure (G1) was slightly less than 12 msec (ranging from 11.4 to 12.3 msec). The average period for the other strain gages and for the soil-structure transducers was approximately 16.7 msec. These three observed frequencies, 12, 16.7, and 24 msec, are very close to the ratios of $1:\sqrt{2}:2$. The computed period for the model, one-fourth the prototype period, was 9 msec (see Section 2.3.2).

The pressure rise inside the model was small during the first dynamic test and occurred primarily in the first 20 to 25 msec. The two pressure gages placed inside the model, P8 and P9, recorded maximum pressures of 0.45 and 0.46 psi, respectively. The average pressures inside the shelter for the period 25 to 180 msec, as determined from the shape of impulse-time curves, were nearly constant at 0.17 and 0.29 psi for Gages P8 and P9, respectively. Both pressure transducers were loaded at only a small fraction of the calibration, and the small signal may account for the difference in these two values. The small change in the shelter volume resulting from the slab deflections and from the compression of the backfill material and the air entrapped therein caused this small pressure increase.

6.1.2 Dynamic Test II. The dynamic loading with a maximum overpressure of 30.1 psi resulted in structure failure that was very similar to the failure of the statically loaded model. This failure, although total in the conventional sense, did not lead to a total collapse of the

roof slab. Areas of the failed structure, especially those near the perimeter wall, could still provide limited shelter.

The response of the slab to this blast loading of twice the design value can be roughly reconstructed with the aid of information contained on the records from this test. Events that happen very quickly can be measured and the time scale greatly spread out with the recording system used. Some events that are usually thought of as occurring instantaneously are found to occur in intervals of several milliseconds because of inertial forces.

The events in the structural response of the model during Dynamic Test II are summarized in Figure 6.1.

The failure of the slab was fairly well formed during the first 50 msec of the test. The column loads and several soil pressures reached peak values at 20 to 25 msec (Figures B.16 and B.19), nearly the same elapsed time as in Dynamic Test I. Most of the strain gages on the tensile reinforcement of the slab did not show this maximum because the instrumented reinforcement had either yielded or was influenced by the development of the yield lines that must have developed followed the yielding near the A and B rows of gages at 7 to 9 msec into the test. The deflections at 20 msec were also great enough to allow the conclusion that flexural yield-line formation development started before the column load response peak at 20 to 25 msec.

The resistance of the slab relative to the applied blast load was not sufficient to halt the downward motion of the slab at this time as it did when subjected to the first dynamic loading of lower magnitude. The effects of the continuing slab deflections and the associated increase in strains and rotations in the column area led to the same type of shear distress around the column that was observed in the statically tested model. The sudden drop in the compressive concrete strain parallel to the column capital at 30 msec (Gage M2, Figure B.27) marks the formation of the inclined or shear crack at that location, and the shear failure of the column area, in the traditional sense, quickly followed.

The bottom bars crossing the column area again were adequate to transfer the shear forces from the slab to the column by a suspension

action. No large decreases in the measured column reactions (Figure B.16) can be seen at the time of the shear distress; the decrease starting at 25 msec for the northeast column is thought to have occurred before this event. Shear distress at all four columns probably developed at nearly the same time.

The decrease in the flexural strength of the slab because of the effective loss of the negative moment area near the columns led to an increase in the downward motion of the slab, after which slab rupture was just a matter of time.

The changing configuration of the slab resulted in more positive moments along the centerline of the slab. This is reflected by the increase in strains detected by the G row of strain gages starting at 35 to 40 msec. Most of the compressive reinforcement strains (Figure B.26) began to move in a tensile direction at about this same time as a result of the changing slab in-plane forces and the rise in section neutral axis location as the section approached its ultimate moment capacity.

The strain of the column reinforcement nearest the corner of the structure (Gage CS-1, Figure B.16) started moving from a compressive strain above $2,000 \times 10^{-6}$ in/in to tension at nearly 40 msec, an event indicating that an increasing majority of the shear forces were entering the column along the perimeter closer to the center panel. This probably marks the beginning of anchorage failure in the splice zones near the column faces closest to the model walls.

The deflections of the slab were great enough to pull the walls in slightly at about 50 msec. Slab deflections at several points had exceeded 3-1/2 inches by this time. The soil pressure gages located at or above the middle of the wall and away from the corner of the model (Gages SS4, SS5, SS6, and SS8, Figure B.19) showed a very rapid release of the soil loading starting at 45 to 50 msec. A small but sufficient inward motion of the wall away from the soil would be necessary for this phenomenon to occur. This motion possibly could have resulted from a decrease in the negative slab moment acting at the top of the wall and tending to force the wall into the soil. Such a decrease could result from an inclined crack forming near the wall and the resulting downward

motion of the slab needed to mobilize the longitudinal slab reinforcement as supporting inclined tension members. This could effectively destroy the compression zone needed for large moment resistance until the slab deflected and rotated enough to jam the bottom slab surface against the wall. The two strain gages placed on the inner wall mat (Gages F1 and F3, Figure B.20) responded to this loss of lateral soil loading by moving from tension to compression during this period.

The rupture of the reinforcement splice regions near the columns had opened enough at 50 to 55 msec to result in the beginning of the sizable pressure rise inside the structure that was detected by the two internal pressure transducers, Gages P8 and P9 (Figures B.15 and B.16). At this time, the tops of the columns started to be pulled in toward the center of the slab.

By 55 to 60 msec, the model walls had developed extensive flexural cracking and yielding at and slightly below their midheight (Figures 4.27 and 4.28) and began an appreciable inward movement. This movement is shown by the records from a horizontally oriented deflection gage placed at the center of the wall (Gage D14) and is further evidenced by the very large tensile strains developing in the outside wall reinforcement mat (Gages F2 and F4, Figure B.20).

The soil pressure transducers indicated that the backfill material around the structure began to move in and bear against the walls during the 70- to 80-msec period (Figure B.19). These forces acted with the in-plane tensile forces of the highly deflected slab to pull the tops of the walls in as much as 2 inches (Figure 4.29). The intersecting walls at the corner of the structure prevented the corners of the walls from moving in and resulted in sizable curvature along the top of the wall (Figure 4.23, right-hand side). The top of the wall was supported during this period by the slab portions between the positive yield line near the center of the outer rows of panels and the wall. These slab areas functioned as deep (approximately 22 inches deep), horizontally oriented beams placed against each wall in a configuration allowing the beams to largely support one another. Parts of the top of the wall and a strip of the slab inside the positive outer-panel yield line also

acted as a part of this beam. Sizable flexural cracking of these horizontal "perimeter beams" are shown in Figures 4.19 and 4.23.

Also during the 70- to 80-msec period, the output from the column load cells began to be much more varied, and some tension loads were indicated (Figure B.16). These records are not valid after about 70 to 75 msec into the test as a result of the columns having tilted enough to jam the plate on the bottom of the column against the edge of the top load cell plates, a loading condition for which the load cell was not designed. This is evidence that the rupture zones near the columns had fairly well opened up by this time.

The behavior of the structure during the 70- to 80-msec period is summarized in Figure 6.2.

The pressure inside the structure equalized the pressure acting on the slab at an elapsed time of 90 to 95 msec and at a pressure of nearly 15 psi (Figure 6.3a). After this time, the slab was subjected to only small net transverse loadings; the structural response of the system had largely been completed in the first 1/10 second of Dynamic Test II.

The net pressure loading on the slab considering the internal pressure rise inside the structure is shown in Figure 6.3b. Unlike a field-tested structure, the laboratory model could reduce the overpressure acting over it by allowing the volume of the pressurized air above it to expand because of the slab deflection and venting into the interior of the model. Much of the pressure decrease in the 30- to 95-msec period resulted from this action. This more rapid pressure decrease was not present during Dynamic Test I, in which the slab deflections were small and no venting in the structure occurred.

The measured deflections of the slab are shown in Figure 6.4 for elapsed times of 30, 40, 45, and 50 msec. As in the static test, the predominant pattern was the formation of a trough around the center of the exterior panels, with only small differences in the deflections at the center of the exterior span and those at the column-wall midspan. This is consistent with the expected yield-line pattern. The small deflections of the center panel should be noted; the probable reason for these small values is discussed in Section 6.4.2. The deflected shapes

of the slab are quite symmetrical, more so than for the static test (Figures 5.1 and 5.2).

Deflections at several other elapsed times are listed in Table 6.2. The deflection traces were too dominated by an electrical noise (see Section 4.2.3) to be usable until about 20-msec elapsed time. All slab deflection transducers ceased to function during the period of 50- to 65-msec elapsed time. The end points of the deflection records are also listed in Table 6.2. The slab continued to deflect after this time until a position close to the permanent deflected shape shown in Figure 4.21 was reached. Final deflections were usually 12 to 15 inches for the slab region inside the positive yield line near the center of the outer panels.

6.2 RESPONSE IN SHEAR DURING DYNAMIC TESTS I AND II

6.2.1 Dynamic Test I. The column loads during Dynamic Test I were smaller and less critical than those in the static test. The maximum column loads ranged from 69 to 77 kips, with an average maximum of 73.3 kips or about 74 kips if the effects of the approximately 1/4-psi dead load of the slab and thin sand layer on the slab (Section 3.3.5) are included. This is 80 percent of the average column reaction from the static test results corrected for dead load. The ratio of the loads in the two tests is 0.75 (19.7 + 0.3 psi dead load divided by 26.6-psi total load for the static test). Thus, the experimental dynamic load factor for the column load was $0.80/0.75 = 1.07$, which is approximately the same as the dynamic load factor of 1.13 computed in Section 2.3.2.

The traces from the two north column load cells are nearly identical (Figure B.2), whereas the records from the south two column load cells followed a slightly different pattern. Loads on the two north columns first increased in a 2-msec period to a peak at 5 to 6 msec of 50 to 60 percent of the maximum load, then decreased, and then increased again to the maximum values at 19 msec into the test. The loads of the two south columns displayed the same initial increase to about 65 percent

of the maximum loads, but then tended to increase slowly to the maximum values reached at 27 msec.

The maximum column load of 77 kips, when corrected for the small dead load and the pressure supported directly on the column (see Equation 5.1), corresponds to a unit shearing stress of 313 psi on a section $d/2$ from the column capital outline, where d is the effective slab depth. This shear is $4.45 \sqrt{f'_c}$ based on the average cylinder strength of 4,970 psi for the second slab model.

The loading in shear of sections of the slab adjacent to the wall was also less critical than in the other two tests (static test and Dynamic Test II). No signs of shear cracking or distress at these locations resulted from Dynamic Test I.

6.2.2 Dynamic Test II. All four column load cells showed similar patterns of response that closely resembled those from the two south load cells during Dynamic Test I for the first 30 to 35 msec. The column loads rose quickly from small values at 3 msec to from 70 to 85 percent of the maximum column loads at 6 msec, then increased more slowly to maximum values of 106 (northeast column) to 127 kips (southeast column) at elapsed times ranging from 20 to 26 msec. The average maximum recorded load was 117 kips, corresponding to an average maximum column load of 118 kips when the dead load of approximately 0.3 psi is considered. This is 127 percent of the average column load observed in the static test. The ratio of the 30.1-psi maximum dynamic load plus dead load to the total loading on the statically tested slab is 1.14. An experimental dynamic load factor of 1.11 (1.27 divided by 1.14) is indicated.

The column loads decreased nearly linearly with time throughout the rest of the 70-msec period that the load cells are believed to have been functioning correctly. No unique characteristic of the column load trace marked the formation of the inclined or shear crack around the column capital and the associated separation of the slab and column capital. Column loads at 30 msec, the approximate time that this shear distress developed, ranged from 79 (northeast column) to 103 kips (southwest column).

The small delay in time between the maximum load and the time at which the concrete section around the column capital lost the capability to transfer shear probably resulted from a combination of at least three phenomena: First, a small but finite time was needed to complete the inclined crack formation and the destruction of the concrete compression zone below this crack. Second, the increase in material strengths due to the initial high strain rates would dissipate with time. And last, the shear and flexural behaviors of a slab are closely related. The combination of the large slab rotations in the column area and the smaller load could have been more critical than the higher loads at an earlier time when the slab deflections were smaller. Other tests (Reference 8) have shown that shear forces considerably in excess of 200 percent of the static shear capacity can be carried by a slab if these forces act while the slab deflections that aid in the shear failure formation are small.

The slab deflections measured 30 msec after the start of Dynamic Test II in the outer panels (Figure 6.4a) were about 15 percent above the corresponding deflections of the statically tested slab at its maximum loading (Figure 5.1b). The rotation at the column perimeter closest to the corner can be related to this deflection. Because the shears and moments should have been larger on this part of the column capital than at the perimeter toward the center of the slab, shear distress probably started nearer the corner.

The maximum column load of 127 kips, when modified to account for the small dead load present and the load above the column area that was supported directly on the columns, corresponds to a unit shear stress, v_u , of 516 psi on a critical location $d/2$ from the column capital outline. This shear stress is 7.34 times the square root of the 4,970-psi concrete cylinder strength of the model. As in the static test, an unknown moment was also transferred by the slab-column connection and increased the severity of the shear loading on the connection.

A comparison of the maximum shear forces in the column areas of the statically and dynamically loaded structures is best made by comparing the values of $v_u/\sqrt{f'_c}$ because of the difference in concrete strengths

for the two models. The $v_u/\sqrt{f'_c}$ value of 7.34 for Dynamic Test II is 126 percent of the value of 5.83 from the static test. This is in line with the 18 to 26 percent increase in strength with dynamic loading reported in Reference 8.

The behavior of the column areas after the occurrence of shear distress was basically identical with that in the statically loaded model. The reinforcement crossing the column capital, especially the bottom mat, successfully held the structure together and transferred loads to the column until the failure of the splice zone in the slab on the wall sides of the column resulted in termination of the loading.

Because of large shear forces and slab rotations near the model walls, the bottom reinforcing bars, which were extended from the slab and anchored in the exterior wall face, were mobilized as tension hangers to support the slab at some locations. Even though no shear reinforcement in the form of stirrups or inclined bars was used, the reinforcement at the wall section was adequate to prevent the slab from tearing along the wall.

The slab did drop a fraction of an inch along the inner wall face as was necessary to develop a vertical force component in the horizontally placed reinforcement extending across the zone of shear distress. Evidence of this downward motion is best shown in Figure 4.24b, where it can be seen that the drop was enough to cause the cover to be torn off atop several top bars.

6.3 ACCELERATIONS AND RESULTING VELOCITY AND DEFLECTION INFORMATION

6.3.1 Dynamic Test I. The acceleration records along with the velocity- and displacement-time curves obtained by numerical integration of the accelerations are presented in Figures B.3 through B.7. Peak acceleration and velocity values for both dynamic tests are presented in Table 6.3. Figures B.3 and 3.35 show the accelerometer locations.

Peak downward accelerations measured on the slab varied from 140 to nearly 240 g's except for a much lower value of 30 g's at Gage AC4 located midway between the two west columns. Peak upward accelerations at the five slab accelerometer locations averaged 70 percent of the

downward values. Maximum upward and downward peaks at all instrumented locations, except the downward acceleration at Gage AC9 on the column base, were recorded in the first 10 msec of the test. Smaller peak accelerations were measured in both vertical and horizontal directions on the wall (about 30 g's downward), on the column drop panel (near 90 g's downward), and on the load cell base (46 g's downward).

Velocity plots from vertically oriented accelerometers display peak downward velocities of 18 to 29 in/sec on the wall, base slab, and drop panel area near the column. These values typify the downward motion of the entire structure. These peak downward velocities and those resulting from instrumentation placed on the slab away from the columns and walls occurred between 10 and 13.5 msec, soon after the 8.5-msec rise time of this test, as should be expected. Peak slab velocities were 37 to 50 in/sec except at Gage AC4, which indicated a peak velocity of 4.4 in/sec, which was less than half of the overall structure velocity. Records from this accelerometer appear low by a factor of approximately four, possibly reflecting an error in the calibration of this transducer.

The peak upward velocities were smaller than the downward velocities, and most were recorded at 30- to 32-msec elapsed time. A very pronounced periodic motion can be seen in several of the velocity curves after the first 30 msec. This motion has a period of nearly 22 msec for the velocities from Gage AC8 on the drop panel adjacent to the column capital and from Gage AC9 on the load cell mounted to the base slab. The values match those obtained from the column load cell data (Table 6.1). Periods of 12.2, 16.6, and 16.8 msec are presented in the velocity records for Gages AC1, AC3, and AC4, respectively, and are most distinct after the first 50 msec. The period for Gage AC1 is the same as that for the tensile reinforcement strain gage at the same location (Gage B1, Table 6.1). Gage AC4 did not function, but an adjacent gage, Gage C2, indicated the same period of nearly 17 msec. The trace from strain Gage B5, at the location of Gage AC3, appears to contain a combination of frequencies, with $T = 12$ msec most dominant.

The displacement plots in Figures B.3 through B.7 from the vertically oriented accelerometers include movement from the downward motion

of the entire model in addition to the deflections of the slab. The displacements from Gage AC9, located on the load cell under the northwest column, and Gage AC6, placed vertically on the center of the west wall of the model, describe the overall motion of structure. The displacement traces from Gages AC6 and AC9 reached peak downward values at 26 msec, decreased over 50 percent to a minimum at 100 msec, and then reached a second (but lower) peak at 175 msec. Thus, a period of 0.15 second was observed for the rigid-body motion of the model in the testing chamber. The difference in the peak displacements for Gages AC6 (0.218 inch) and AC9 (0.307 inch) is larger than expected with the massive base slab used, although some bending of the 10-inch slab was possible. The displacement of Gage AC8, located 1 inch from the column capital on the drop panel, should be only slightly higher than the base slab and column base. The maximum value of 0.352 inch at $t = 26$ msec at this location suggests that the displacement value from Gage AC9 better indicates the motion of the entire structure than does that from Gage AC6. The shapes of the displacement curves from all accelerometers except Gage AC7, oriented horizontally on the wall, and AC3 were the same. The records from Gages AC1, AC2, and AC5 showed center span and center panel slab displacements of 0.39 to 0.41 inch. (The calibration of Gage AC4 is questionable.)

An approximate slab deflection curve can be constructed by subtracting the displacement trace resulting from the readings from Gage AC9 from the slab displacement trace, as shown in Figure 6.5a. Such curves are shown for the locations of Gages AC1, AC2, and AC5 in Figure 6.5b, c, and d. Peak deflections occurred at approximately 20 msec, the same time as the maximum for many measured strains and the maximum column loads. The values shown in Figure 6.5 are considerably lower than would be expected from the static test results (see Figure 5.1a). A partial explanation is the effect of rapid loading on the stress-strain curve and the lack of creep in the dynamic test. A corrected displacement trace from Gage AC3, located at the center of the northwest corner panel, would show that the slab moved upward by up to 1/3 inch (see Figure B.4b for displacement plot), a behavior which

should not develop and is inconsistent with the sizable tensile strain measured on the tensile reinforcement in that area (Gages B4 to B6, strains = 600 to 1,300 $\times 10^{-6}$ in/in tension).

6.3.2 Dynamic Test II. The patterns of the acceleration traces (Figures B.21 and B.22) were similar to those for Dynamic Test I for the first 25 msec. Peak downward accelerations within this time period generally increased 20 to 70 percent above the Dynamic Test I values and were recorded between 2.2- and 4.3-msec elapsed time (Table 6.3). Maximum values of 290, 256, and 230 g's were measured at the center of an exterior, corner, and interior panel, respectively. Peak upward accelerations also were recorded (except at Gage AC8) at approximately the 4.5-msec rise time of the dynamic loading. These peaks scattered about the values from Dynamic Test I, i.e. some were higher and some were lower.

After the first 25 to 30 msec, the accelerometer output reflected the downward acceleration of the slab associated with failure. Because the slab was loaded to failure, the acceleration records reflected much less of the vibrational characteristics of the model than did those for Dynamic Test I. Several accelerometers or lead wires were damaged by the failure. The resulting lack of points within the abbreviated valid record at which acceleration or velocity records could reasonably be assumed to have zero value made integration of the acceleration records to obtain deflection data questionable because of the difficulty of performing the base line corrections for the acceleration data often needed in this procedure.

The accelerations were singly and doubly integrated over the first 50-msec period once without any attempt to correct the records for base-line inaccuracies. The deflections so obtained from Gages AC1, AC2, and AC3 correlate reasonably well with measured deflection values, and those from Gages AC5 and AC8 are adequate to show deflection trends and to give approximate velocity data. These plots are shown in Figures 6.6 and 6.7.

The displacement plots include the rigid-body motions of the test structures and should be 0.40 to 0.50 inch higher than the slab

deflection measured relative to the base slab at 20 to 25 msec. This estimate is based largely on scaling up of the results of Dynamic Test I (Figure 6.5a). The displacements from Gage AC8, which reached a peak of 0.55 inch at 22 msec, should be only slightly greater than the base slab motion prior to the completion of inclined or shear cracking around the column. The upward movement of the displacement at Gage AC8 from 22 to 34 msec is largely a result of the same oscillation of the entire structure in the soil tank, as can be seen in Figure 6.5a for Dynamic Test I. This motion of the entire structure, along with the deflection of the center panel and a small base-line shift of the acceleration data in the integration process, also contributed to the upward motion in the Gage AC5 displacement from 25 to 35 msec. The small peak displacement of Gage AC5 at 8.5 msec reflects the initial deflection of the center panel also indicated by Strain Gages G1 through G4 (Figure B.25).

The peak downward slab velocities during the first 20 msec of this test are presented in Table 6.3. These velocities ranged up to 100 in/sec, roughly double those of the first test, and were recorded at 4.7- to 6.1-msec elapsed time, one-half the peak times for Dynamic Test I.

The velocity traces in Figure 6.6 show that the resistance of the slab was sufficient to slow the downward motion of the slab in the outer panels to 20 to 25 in/sec at the AC1 and AC2 gage locations at 26 to 30 msec. The velocity from Gage AC8 indicates that the base slab, walls, and columns were moving upward at 15 to 20 in/sec during this time. The subsequent increase in the slab velocities signals the beginning of the slab failure.

Some accelerations recorded during the failure and after the 50-msec period for which records are shown contained peaks that were as high as those in the initial 10 msec. These peaks included a very sharp spike of 565 g's at 79 msec for Gage AC3 and were largely the result of local and probably nonreproducible conditions near the transducer. An exception is the behavior of Accelerometer AC8 placed on the drop panel adjacent to the column capital.

The acceleration of Gage AC8 changed from the pattern of largely oscillating about 0 at the end of the 50-msec period shown in Figure 6.7

to a downward trend starting at 52 msec. The average acceleration downward from 52 to 63 msec was approximately 25 g's, and from 63 msec to the end of this downward trend at 70 msec, the average was approximately 45 g's. These accelerations and times signify a downward velocity increase of about 240 in/sec over the 18-msec period. The downward acceleration of the drop panel at this time was caused by the downward motion of the slab relative to the column capital resulting from the completion of inclined shear cracking around the column and the associated tension membrane area around the column capital.

6.4 SLAB STRAIN MEASUREMENTS

6.4.1 Dynamic Test I. Nearly all strain gages located in the slab area displayed peak values at or slightly before 20-msec elapsed time. Figure 6.8 displays the distribution of these peak strains along the four instrumented lines of tensile reinforcement. In a few cases, the greatest strain was recorded at a second peak 12 to 17 msec later.

The strain distributions shown in Figure 6.8 agree reasonably well with those observed in the static test at a load of approximately 20 psi (Figure 5.5). For both loadings, the yield strain of the reinforcement was exceeded at every gaged location on the top slab reinforcement at the inside wall face. However, in Dynamic Test I, only the strain from Gage A4 exceeded $2,200 \times 10^{-6}$ in/in, whereas in the static test all strains had exceeded $4,000 \times 10^{-6}$ and the gages had failed before a static loading of 20 psi was reached. In both tests, the strains along the B line at the center of the outer panels were close to or slightly above the static yield strain of $1,250 \times 10^{-6}$ in/in. Strains along the negative moment section along the column line (C row of gages) were generally below $1,000 \times 10^{-6}$ in/in and decreased to very small values near the wall in dynamic tests. The strains at the positive moment section at the center of the slab were generally greater in the dynamic test than at the 20-psi level of the static test, but still averaged lower than those along any other line of gages. The yield strain was exceeded at the location on the column line of this row of gages (Gage G3), whereas no pronounced piling up of strain along the column

line was evident in the static test. Strains in this dynamic test then decreased slowly as the overpressure decayed. The fluctuations in the gage output during this decrease has been noted in Section 6.1.1.

The strains on the compression reinforcement and concrete compression surface (see Figures B.13 and B.14) also displayed a maximum value at a peak between slightly before 20 msec to nearly 30 msec followed by a general slow decrease with some evidence of slab vibrations. The highest measured compression reinforcement strain was $-1,150 \times 10^{-6}$ in/in close to the wall on the bottom of the slab. The concrete strain gage at this location did not function. Maximum concrete strains varied from approximately -300×10^{-6} in/in to approximately -750×10^{-6} in/in, with observed compression reinforcement strains at the same location being slightly less. As should be expected, the traces from the strain gages on the compression reinforcement and concrete compression surface at the same general location were very similar, as can be seen by comparing the records from concrete surface strain Gages M7, M6, and M3 (Figure B.14) with those from Gages E1, E4, and E6, respectively (Figure B.13).

The rate of straining of the reinforcement varied with location. The strain rate displayed by the A and B rows of gages from strains corresponding to less than 10-ksi stress to yield were from 0.07 to 0.40 in/in/sec with most approximately 0.20 to 0.25 in/in/sec. Steel loaded at these strain rates displays a strength approximately 20 percent above its strength when slowly loaded (References 8, 14, 20, and 31). The strain rates at most locations along the C and G rows of tensile reinforcement gages were from 0.05 to 0.10 in/in/sec, with smaller values near the wall at the center of the slab.

6.4.2 Dynamic Test II. Many of the characteristics of the strain gage responses have been discussed in the description of the slab failure in Section 6.1.2. The strains recorded during Dynamic Test II do not include any permanent strains resulting from Dynamic Test I.

The gages placed over the inside wall face (A row) all failed before 20-msec elapsed time. The instrumented reinforcement at the center of the outside row of panels (B row gages) all yielded at 7 to 8 msec (Figures B.23 and B.24) except at the location of Gage B6, the gage

closest to the wall. At most locations, the strain then leveled off or decreased slightly until 40- to 70-msec elapsed time, when the tensile strain increased suddenly and the gage was lost. The positive moment yield line did not exactly follow the B line of gages, and strains greatly exceeding the yield level probably developed at the gaged locations only when the rotation of the yield line was great enough to cause local flexural bond failure and large yielding extending back to the B line of gages.

The strains measured along the centerline of the structure (G row, Figure B.25) were considerably different in Dynamic Test II from those observed during Dynamic Test I. The traces from Gages G1 and G2 in the center panel show a rapid tensile strain buildup reaching a peak at 7- to 8-msec elapsed time and then decreasing to small compressive values at approximately 30 msec. Gage G4, just outside the column line, detected a similar peak, and the signal from Gage G3, on the column line, leveled off at approximately $1,700 \times 10^{-6}$ in/in at approximately 7 msec.

This behavior during the first 10 msec was not observed during Dynamic Test I and could have resulted from the smaller rise time for the loading in Dynamic Test II (4.5 msec versus 8.5 msec for Dynamic Test I). The faster pressure rise in Dynamic Test II resulted in each panel initially acting almost independently of the rest of the slab, whereas in Dynamic Test I, the slab had enough time to respond more as a slab system and to distribute moment accordingly.

The decrease in strains along the positive moment line of the center panel (Gages G1 and G2) was probably related to the yielding near the wall (strain gage Row A) and center of the outer panels (near Row B) in the following manner: the yielding along these lines signaled the formation of a positive moment yield line around the center of the outer panels. Additional pressure and inertial loadings on the slab placed between this developing yield line and the column centerline would have to be supported by increasing negative moments along the column lines. This increase in negative moment around the perimeter of the center panel evidently decreased the positive moment at the center of the structure more than enough to offset the effects of the continuing load on this

panel. A similar decrease in strains in the positive moment areas of the center panel was observed in the statically tested model after the B row of gages indicated yielding (Section 5.5.1 and Figure 5.5d). More moments were thrown to this center panel positive moment area when the column areas developed distress in shear and the slab began to move down from the column capitals at about 35 msec. This led to the failure of several G row gages between 40 and 50 msec.

The strain rates prior to yield, indicated by the B row of gages on the positive moment reinforcement, were usually 0.3 to 0.4 in/in/sec.

6.5 SOIL FORCES AND RESPONSE OF THE WALLS

6.5.1 Dynamic Test I. The soil forces measured on the west wall of the model increased in a general trend from small values during the first few milliseconds to peak values at 26 to 27 msec at all locations except that of Gage SS7, located near the bottom of the wall along a column line. These peaks lagged those of the slab reinforcement records by about 8 msec. After these peaks, the soil pressure slowly decreased with the decaying overpressure. Some harmonic content can be seen in the soil pressure data (Figure B.8).

The distribution of the soil forces acting on the structure at the 26- to 27-msec period of the maximum values is shown in Figure 6.9b and is compared with that of the static test at near maximum load in this same figure. In both the static test and Dynamic Test I, the upper two measured soil pressures exceeded the applied load by 30 to 40 percent.

The soil pressure in Dynamic Test I decreased nearly linearly with depth. The soil pressures at midheight of the wall were much higher than those in the static test. Consequently, in-plane forces acting on the slab as a result of the soil loading on the wall were greater relative to the applied loading in Dynamic Test I than in the static test.

The response of the wall to the dynamic loading was similar to that of the statically tested model. After some variations in the first 10 msec, the strains on the inner wall mat (Gages F1 and F3, Figure B.9) assumed small (less than -75×10^{-6} in/in) tensile values, whereas compressive strains of approximately 100×10^{-6} in/in were maintained at

the location of operative Gage F2 on the outer wall mat. As in the static test, these small measured strains were not typical of the magnitude of strains throughout the wall. The small strains resulted because the midheight wall reinforcement strain gages were slightly below a line of contraflexure. The horizontal deflection of the wall, as indicated by Gage D14, reached a peak outward movement of 0.022 inch at 20 msec (Figure B.9).

6.5.2 Dynamic Test II. The soil pressures acting on the wall rose at a generally uniform rate at locations near the north end of the wall (Gages SS1, SS2, and SS3, Figure B.19) and reached maximum values at approximately 22 msec, which was 4 to 5 msec earlier than in Dynamic Test I. In contrast to the behavior seen in Dynamic Test I, the soil pressures at the midheight of the wall away from the north boundary of the wall (Gages SS4, SS6, and SS8) rose very quickly almost to maximum values during the 2- to 5-msec period, then remained at roughly constant values until approximately 50 msec.

The response of Gage SS4, located near the top of the wall at the column line, was between these two extremes. Gage SS7, located near the base of the slab below SS4, measured only small pressures during the first 40 msec of the test except during the 20- to 27-msec period.

The maximum pressures occurring at each location during the 20- to 23-msec time period are plotted in Figure 6.9c. The two pressure transducers near the top of the wall recorded pressures 50 and 63 percent above the maximum overpressure level, somewhat greater than those recorded in the two previous tests. The pressure at the midheight of the wall averaged over three times those measured in the static test (Figure 6.9a). The distribution of soil pressures was similar for Dynamic Tests I and II during the 20- to 30-msec time interval (compare Figures 6.9b and 6.9c). Calculations based on the distributions shown in Figure 6.9 suggest that the slab in-plane forces in Dynamic Tests I and II at the times plotted were 15 to 20 percent and 25 to 30 percent greater than would be predicted by multiplying the static test results by the ratio of dynamic to static loads.

The sudden release of soil pressure at the gages away from the ends

and bottom of the wall (Gages SS4, SS5, SS6, and SS8) at approximately 50 msec marks the start of an inward movement of the wall, a movement that the horizontally placed deflection transducer at the middle of the west wall (Gage D14, Figure B.20) indicated was quite small until about 58 msec into the test. The pressures at the top and midheight near the north end of the wall (Gages SS1 and SS2) decreased more slowly because the north wall adjoining the instrumented west wall reduced the wall deflections near the corner of the slab. Soil forces along the bottom of the wall (Gages SS3 and SS7) increased because more lateral support was necessary at these locations when the soil pressure above was released. The soil pressure records show that the fine sand backfill began to move against the deflected wall at about 90 msec.

The measured wall strains in the first 40 msec followed the pattern established in Dynamic Test I; i.e., tension developed on the inner face (although the strain recorded by Gage F4 was much greater than that recorded in Dynamic Test I, see Figure B.20) with small compressive stress in the outer welded wire fabric mat.

The moment acting at the instrumented height reversed when the soil loading on the wall was released at approximately 50 msec. The strain of the tensile reinforcement in the outer wall face (Gage F2 and F4) began a rapid increase in the tensile direction at 55 msec and reached the yield strain soon after 60 msec. The movement of the inner wall mat strain in a tensile direction probably reflects the rise of the neutral axis location when the curvatures at these sections increased. This rise in the neutral axis was sufficient to cause the instrumented vertical rods of the inner mat (Gages F1 and F3) to go into tension at 85 to 95 msec. Figure 4.27c shows that the widest cracking in the west wall of the model was near the midheight location of the wall reinforcement strain gages. The leveling off of the wall reinforcement strains at 115 msec suggests that the wall motion, and hence the slab motion, had largely ceased by that time.

The crack patterns of the walls shown in Figures 4.27 and 4.28 emphasize the role that perpendicular walls play in strengthening and otherwise influencing the wall and, consequently, the slab behavior.

The reinforced framing around the opening in the south wall greatly influenced the crack pattern and behavior of this wall. The wall crack patterns also show that the base of the walls approached the desired fixed condition, much more so than could reasonably be expected in a prototype structure even if the wall footings and floor slab were structurally monolithic.

Cracking in the center half of the wall closely followed the horizontal bars of the welded wire fabric mat used. The local torsion failure of the wall-slab section near the ends of the wall (Figure 4.30) accompanied the development of the wall crack pattern. The addition of practical amounts of torsional reinforcement in these areas probably would have added little to the wall performance unless the effective size of the beam at the top of the wall had been increased considerably.

6.6 FLEXURAL RESPONSE AND STRENGTH OF THE DYNAMICALLY LOADED MODEL

The flexural strength of the slab was not uniquely determined by the two dynamic loadings. The two dynamic tests did establish lower and upper bounds of approximately 20 and 30 psi on the overpressure level needed to cause considerable deflection without rupturing the slab.

The observed resistance characteristics of the model, as demonstrated in the static test, were such that the dynamically loaded slab structure should be expected either to survive a dynamic loading with relatively little damage or to be severely damaged by the loading, depending on whether the load was of large enough magnitude to produce shear distress in the column areas and the resulting loss of slab resistance. The shape of the static resistance curve for the slab system (using centerline deflection of the outer panels as a measure of slab deflection) is shown in Figure 6.10.

The dynamic loadings of interest in the shelter design are of fairly long duration (t_{50} values usually of 0.5 sec or more). Even considering that the times of the prototype would be four times those of the model, in most cases a loading large enough to drive the slab past the maximum resistance would not decrease in magnitude fast enough for the ductile portion of the resistance curve to slow and stop the slab

without rupturing and releasing the differential pressure across the slab. Even though the slab system displayed the ability to deflect at least the column spacing divided by 7.5 while maintaining a resistance of from 15 to nearly 17 psi (including dead load), this portion of the curve is enough below the maximum resistance of 26.6 psi that it would be unstable for gravity loads and all dynamic loadings except those of short duration. As the duration of a dynamic load increases, the slab behavior becomes more like the response to a gravity load where any decreasing branch of the resistance curve would be unstable and not usable.

The magnitude of dynamic overpressure loading that would have deflected the slab to a point just short of that needed for the column areas to develop shear distress can only be estimated and would be somewhat dependent on the rise time of the load, which did differ in the two dynamic tests. Based on the results of all three tests, it is thought that this critical dynamic loading would have been about 26 or 27 psi, slightly above the average of the two dynamic loadings.

One of the reasons for placing this value closer to the upper test loading is that the 19.7-psi dynamic loading produced maximum reinforcement strains that were of nearly the same magnitudes as those for the static loading at a slightly higher load level (20.8 psi + 1.1 psi slab and water weight). Because the strain rate effects would increase the yield strain for the dynamically loaded model, a dynamically loaded slab should have at least as much margin against failure as a statically loaded slab exhibiting the same strains. Also, the slab system was able to halt deflection when subjected to the lower dynamic loading in the short time of 20 msec and was able to significantly slow the downward velocity prior to the occurrence of column shear distress at 30 msec in Dynamic Test II. The maximum shearing stresses for the second dynamic test were little if any above those that would be predicted using the information in Reference 8 and the static test results.

The flexural yield-line pattern developed during the second dynamic loading (Figure 4.19) was very similar to that which was developing in the statically loaded model at maximum resistance (Figures 4.2 and 5.9).

Except for the localized response of the slab panels during the first 6 to 8 msec of Dynamic Test II (Section 6.4.2), the flexural behavior in the two dynamic tests was basically the same up to the time of occurrence of column area shear distress. The in-plane slab compression forces resulting primarily from the soil loading on the walls in the dynamic tests were at least as great as those resulting from the static test (Section 6.5.2) and increased the flexural strength of the slab by adding some axial load on the lightly reinforced slab sections. This increase in strength, which also developed in the statically loaded slab for the same reason, is explained in detail in Section 5.6.1.

6.7 BEHAVIOR OF THE COLUMNS

The columns exhibited no distress during the first dynamic loading. The one functioning column reinforcement strain gage recorded a maximum compressive strain of -980×10^{-6} in/in, which was well within the elastic range of the reinforcement. This instrumented bar was the closest bar to the northwest corner of the structure of the longitudinal reinforcement in the northwest column. This location should have resulted in this bar being more highly stressed in compression than the other longitudinal bars of this column. A visual inspection after the test showed that the column moments were not great enough to cause tensile cracking of the column side adjacent to the center panel. This evidence suggests that the behavior of the columns remained elastic during this first loading.

The columns were extensively damaged in the second dynamic loading, as shown in Figure 4.26. The one instrumented column bar showed strains of about $-2,000 \times 10^{-6}$ in/in (compression) from 20 to approximately 40 msec (Figure B.16). The column moments acting prior to the occurrence of column shear distress and possibly until the anchorage failure of the bars in the bottom reinforcement splice zone acted to increase the compression strains on the side of the column closest to the corner. The loading conditions were severe enough to cause the concrete to crush and the cover to spall on each column below the column capital at the

location of maximum column moments. Horizontal tension cracks were found near the top of the southwest column at the side closest to the center of the slab.

The column spirals were not designed to increase the column strength after spalling of the cover, and the columns were close to failure in Dynamic Test II. The importance of considering the actual slab flexural capacity, including effects of in-plane forces, in determining column loads was vividly demonstrated in this test. A slightly less conservative column design could have easily resulted in the column failing and the shelter model would have been damaged much more than it was.

Figure 5.11 shows that the columns in the statically loaded model were loaded nearly to capacity. The maximum column loads during Dynamic Test II averaged 27 percent greater than those in the static test (Section 6.2.2). This is nearly the same as the observed increase in column strength with dynamic loading reported in Reference 32.

However, the column moments in Dynamic Test II could have been significantly greater than those shown in Figure 5.11 based on a static flexural analysis. The early positive yield-line formation at the center of the exterior panels and the consequent much greater slab deflections in the outer panels relative to that of the center panel would have this effect.

The vertical cracking and splitting visible at the bases of the columns in Figure 4.26 are thought to have occurred because the bottom plates of the column were not thick enough to spread the loads evenly from the steel spheres atop the load cells to the columns.

6.8 ENVIRONMENT WITHIN THE SHELTER MODEL DURING THE DYNAMIC TESTS

The pressure rise (less than 1/2 psi, see Section 6.1.1) within the shelter model and the floor motion during Dynamic Test I were not excessive. No concrete was spalled off the slab roof of the shelter.

Conditions within the shelter model were much more severe during Dynamic Test II, which had a maximum overpressure of 30.1 psi.

During this test, the pressure buildup within the shelter at the

transducer location near the base slab was 15 psi, with the last 10 psi of this increase occurring over a 20-msec time period. Air jets of considerable velocity must have developed as the splice zones near the columns opened. The system was successful in reducing the pressures inside the shelter to 50 percent of the maximum overpressure. However, the closed configuration and limited volume of the test chamber played an important part in limiting the internal pressure to a value below that which should be expected in a full-scale, field-tested structure for many loading conditions (see Section 6.1.2). The internal pressure inside the prototype structure, when subjected to a 30-psi maximum overpressure of fairly long duration, should be close to the overpressure level at a time of approximately 0.4 sec after the pressure rise.

Considerable amounts of concrete fragments dropped from the roof slab during Dynamic Test II. Some of these fragments, along with some of the thin layer of sand placed over the slab, can be seen in Figure 4.22, which is a view of the shelter interior taken through the opening in the south wall. Figure 6.11 shows the debris from around the southwest column of the slab. The drop panel around the column capital was the source of a considerable amount of large debris. Results from Gage AC8 on the drop panel area indicate that this large debris was traveling at over 20 fps when it left the slab (see Section 6.3.2). Smaller debris from the positive flexural yield lines probably started to fall at least this fast, and debris from the splice failure zone was driven by the incoming air jets.

The large openings in the slab at these splice zones would drastically reduce the radiation protection offered by the shelter.

TABLE 6.1 OBSERVED PERIODS FOR DYNAMIC TEST I

Transducer Type and Designation	Start of Observed Portion of Record	Number of Cycles Observed	Average Observed Period
	msec		msec
Load Cells:			
LCNE	19.9 (max load)	5	25.2
LCNW	18.4 (max load)	5	25.0
LCSW	26.9 (max load)	4	22.2
LCSE	26.8 (max load)	4	22.5
Surface Pressure Gage:			
P1	28	2	25.0
Soil-Structure Pressure Gages:			
SS1	25	5	16.5
SS3	56	6	16.8
SS7	70	8	16.7
Tensile Reinforcement Strain Gages:			
A2	42	9	12.1
B1	42	5	12.3
B2	42	7	12.0
B3	42	7	11.9
B6	42	8	16.8
C2	60	7	16.8
C3	52	8	16.7
C4	52	8	16.7
C5	52	8	16.7
C6	51	8	16.8
C7	52	8	16.8
G1	51	8	16.7
G2	51	6	11.5
G3	43	7	11.4
G4	50	9	12.0
G5	50	7	11.5

TABLE 6.2 SLAB DEFLECTIONS MEASURED DURING DYNAMIC TEST II

Deflection Transducer	Deflection at Indicated Elapsed Time					Elapsed Time at End of Valid Record	Deflection at End of Valid Record
	25 msec	35 msec	45 msec	55 msec	60 msec		
	inches	inches	inches	inches	inches	msec	inches
D1	1.10	1.65	2.20	4.75	5.90	60	5.90
D2	0.65	0.95	1.45	3.00	a	59	3.75
D3	1.25	2.05	3.30	a	a	50-1/2	4.80
D4	0.95	1.35	2.00	3.75	4.80	62	5.65
D5	0.15	0.15	0.55	1.30	1.90	64	2.40
D6	1.15	1.90	3.40	a	a	53	4.80
D7	1.20	1.55	2.45	a	a	54	4.80
D8	0.90	1.20	2.25	4.80	a	56	4.95
D9	1.10	1.70	3.20	a	a	48-1/2	3.75
D10	0.95	1.45	2.25	4.65	a	55	4.65
D11	0.20	0.30	0.55	1.10	1.95	63-1/2	2.75
D12	1.05	1.80	3.15	a	a	51-1/2	4.50
D13	≈ 0.10	0.20	0.70	a	a	52	2.60

^a Transducer out.

TABLE 6.3 ACCELERATION AND VELOCITY PEAKS FROM DYNAMIC TESTS

Transducer	Peak Downward Acceleration		Peak Upward Acceleration	
	Dynamic Test I	Dynamic Test II	Dynamic Test I	Dynamic Test II
AC1	237 g's @ 2.6 msec	265 g's @ 2.2 msec	130 g's @ 6.9 msec	109 g's @ 6.8 msec
AC2	192 g's @ 3.3 msec	176 g's @ 4.2 msec	134 g's @ 2.9 msec	107 g's @ 4.9 msec
AC3	140 g's @ 9.8 msec	230 g's @ 3.5 msec	98 g's @ 6.9 msec	116 g's @ 4.1 msec
AC4 ^a	30 g's @ 4.0 msec	51 g's @ 2.4 msec	20 g's @ 5.1 msec	35 g's @ 3.0 msec
AC5	154 g's @ 2.5 msec	256 g's @ 3.2 msec	160 g's @ 4.5 msec	182 g's @ 2.8 msec
AC6	29 g's @ 6.9 msec	38 g's @ 4.3 msec	21 g's @ 9.5 msec	39 g's @ 3.9 msec
AC7	29 g's (in) @ 4.0 msec	28 g's @ 4.3 msec	29 g's (out) @ 4.1 msec	35 g's @ 5.1 msec
AC8	89 g's @ 3.1 msec	127 g's @ 2.5 msec	62 g's @ 2.5 msec	42 g's @ 21.4 msec
AC9	46 g's @ 5.8 msec	43 g's @ 3.5 msec	9 g's @ 27.4 msec	63 g's @ 3.6 msec
	Peak Downward Velocity Dynamic Test I	Peak Upward Velocity Dynamic Test I	Peak Downward Velocity During First 20 msec Dynamic Test II	
AC1	37.0 ips at 10.2 msec	12.5 ips at 33 msec	103 ips at 4.1 msec	
AC2	48.0 ips at 13.5 msec	11.0 ips at 32 msec	75 ips at 4.7 msec	
AC3	50.0 ips at 12.5 msec	37.0 ips at 27 msec	91 ips at 5.5 msec	
AC4 ^a	9.4 ips at 11.4 msec ^a	4.4 ips at 55.5 msec ^a	17 ips at 5.1 msec ^a	
AC5	48.0 ips at 11.9 msec	16.5 ips at 31.5 msec	≈91 ips at 5.2 msec ^b	
AC6	18.0 ips at 13.4 msec	4.2 ips at 81.0 msec	≈12 ips at 8.6 msec ^b	
AC7	2.0 ips (in) at 36 msec	2.9 ips (out) at 9.2 msec	--	
AC8	29.0 ips at 10.5 msec	7.6 ips at 32 msec	40 ips at 7.3 msec ^b	
AC9	23.0 ips at 11.1 msec	10.0 ips at 31 msec	≈34 ips at 7.5 msec ^b	

Gage Locations (see Figure 3.35): AC1, AC3, AC5 at center of exterior, corner, and interior panel
 AC2 at centerline of wall to column span
 AC4 at center line of column to column span
 AC6 and AC7 at center of wall, horizontal and vertical
 AC8 near column capital on drop panel
 AC9 on base of column

^a Questionable transducer calibration, both tests.

^b Velocity values given are low due to probable base-line shift of accelerometer data.

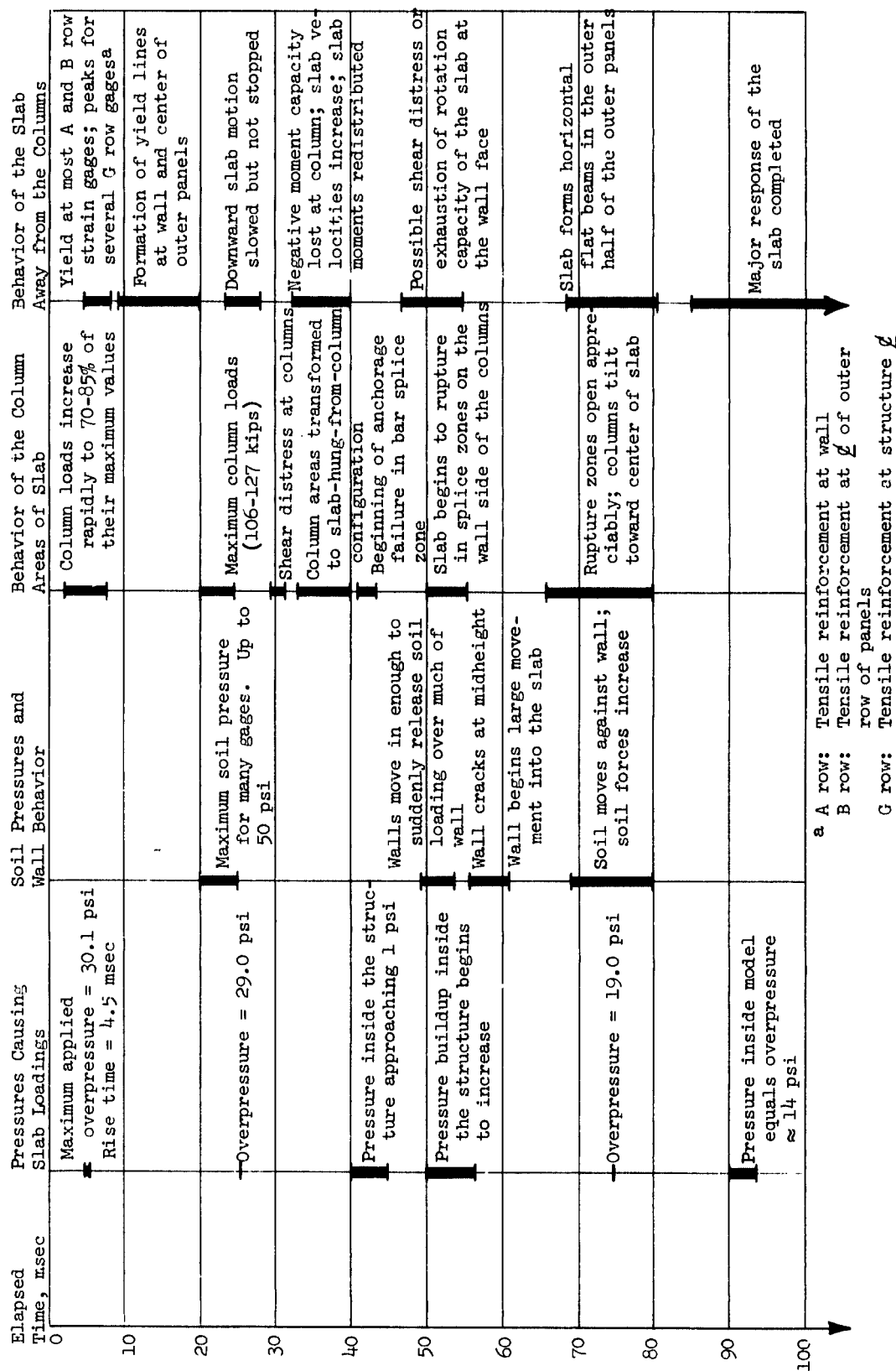


Figure 6.1 Behavior of the shelter model during the first 100 msec of Dynamic Test II.

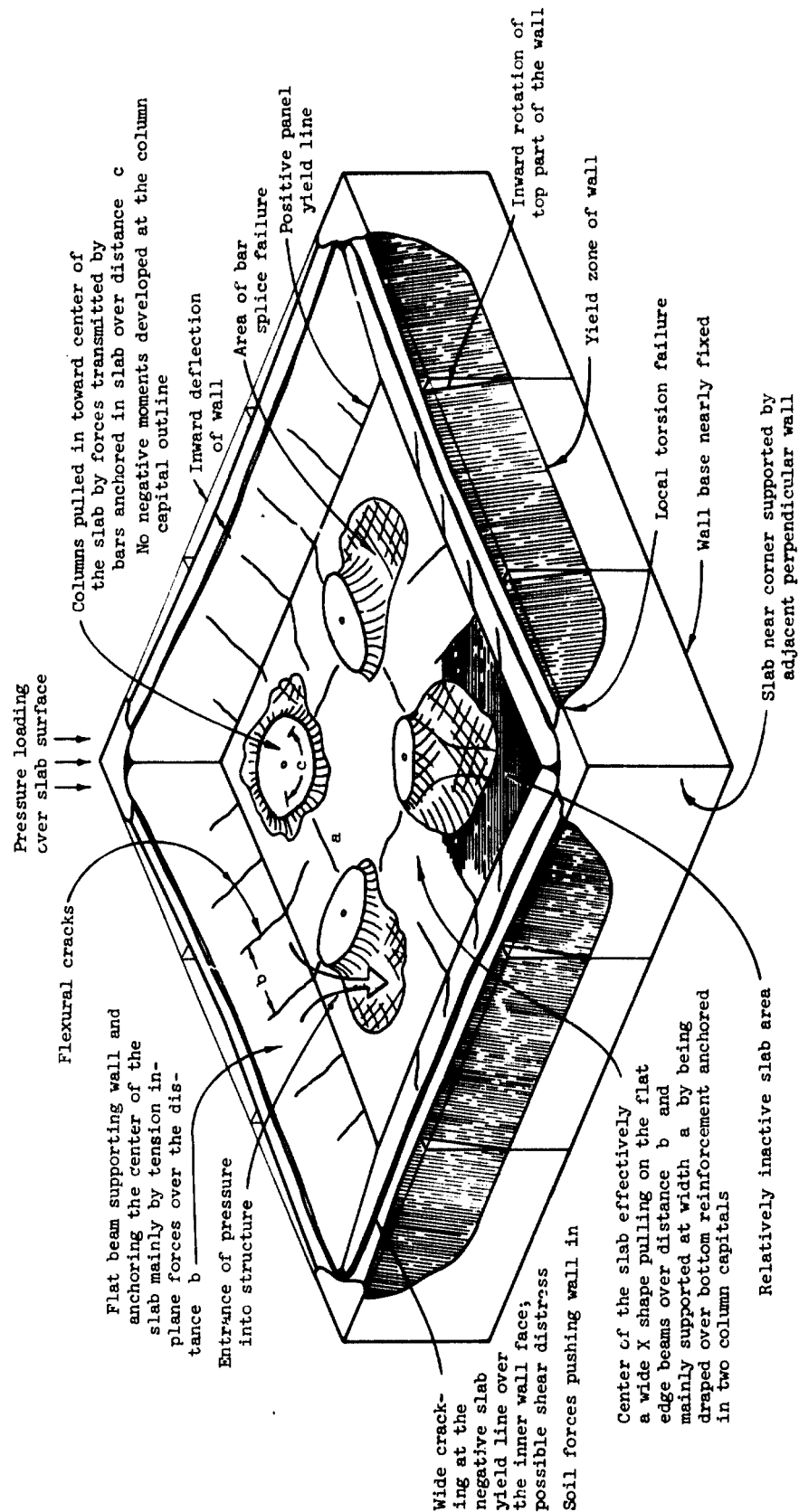
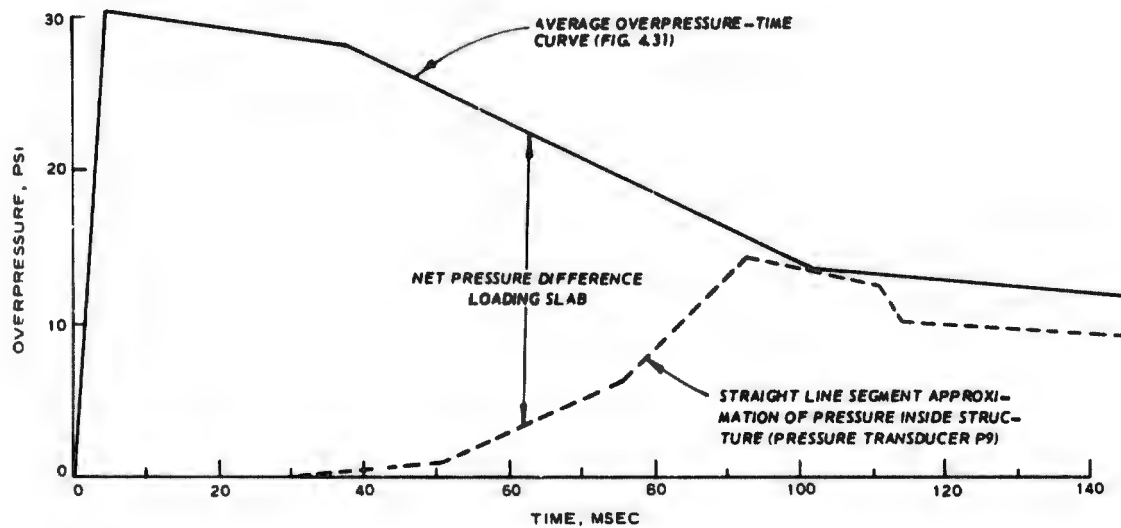
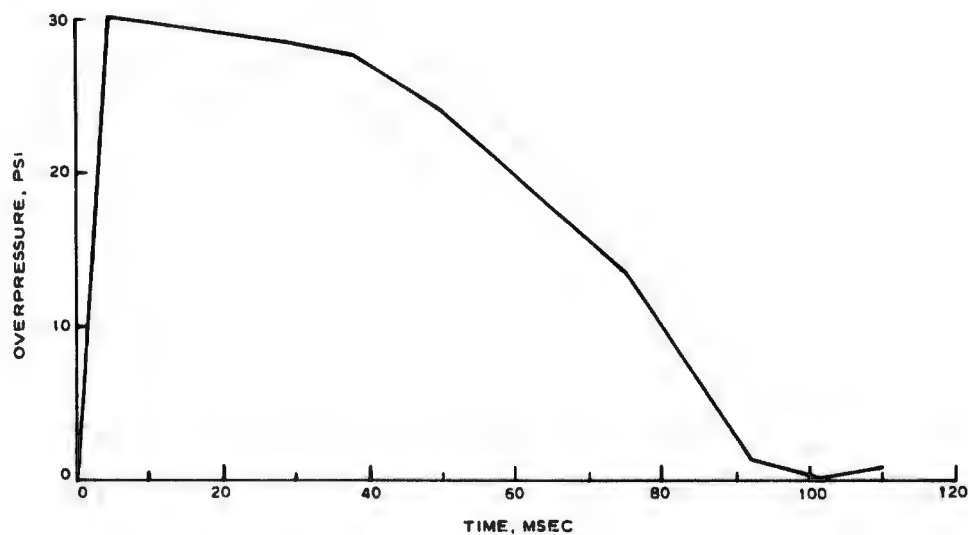


Figure 6.2 Behavior of the slab during 70- to 80-msec period.



a. PRESSURE ABOVE AND BELOW SLAB



b. AVERAGE NET PRESSURE ON THE SLAB

Figure 6.3 Pressure acting on the slab during Dynamic Test II.

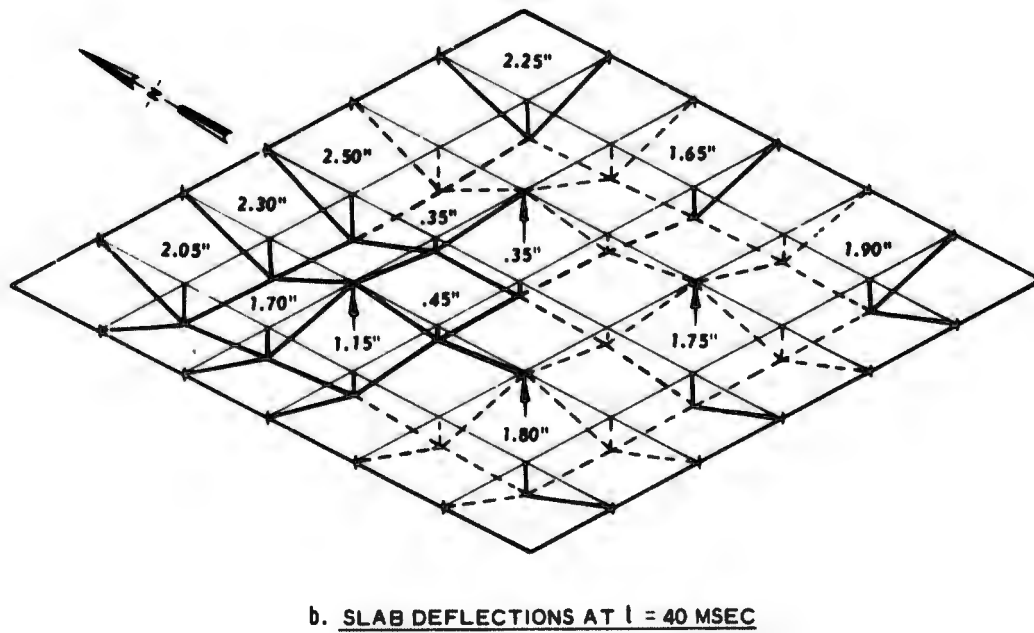
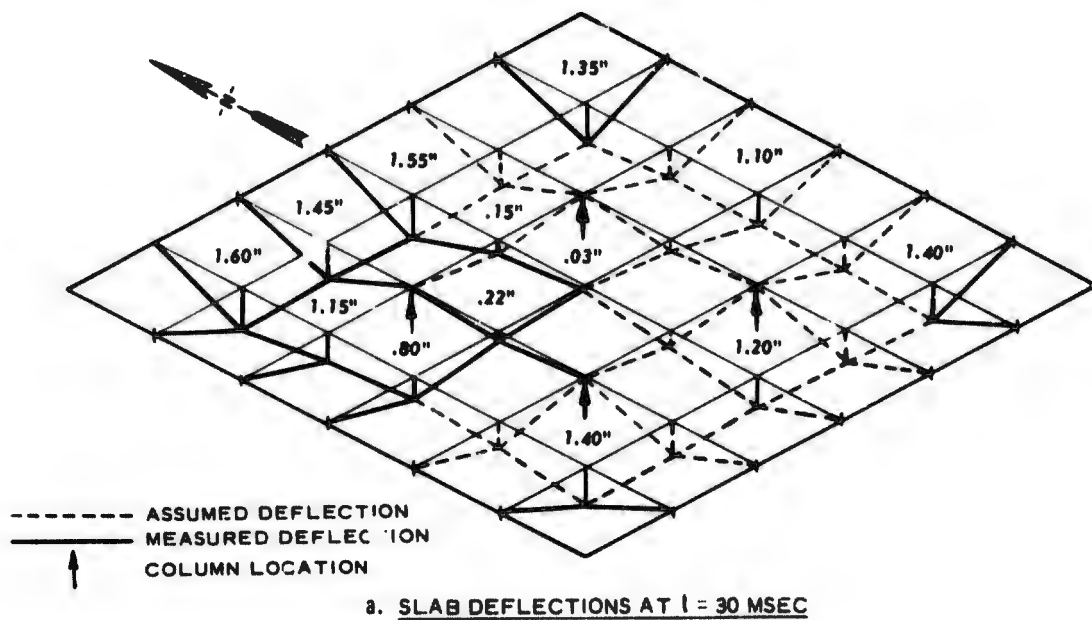
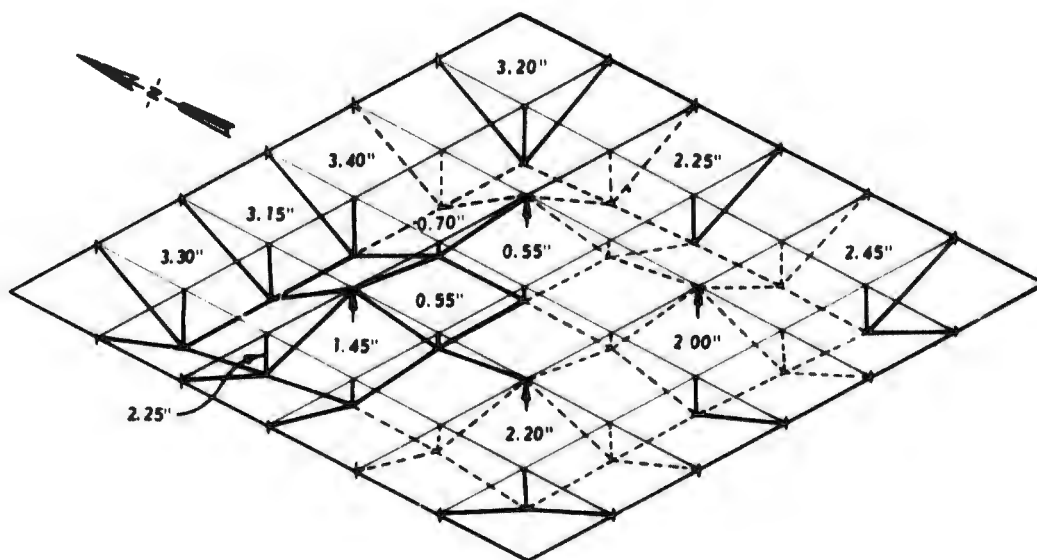
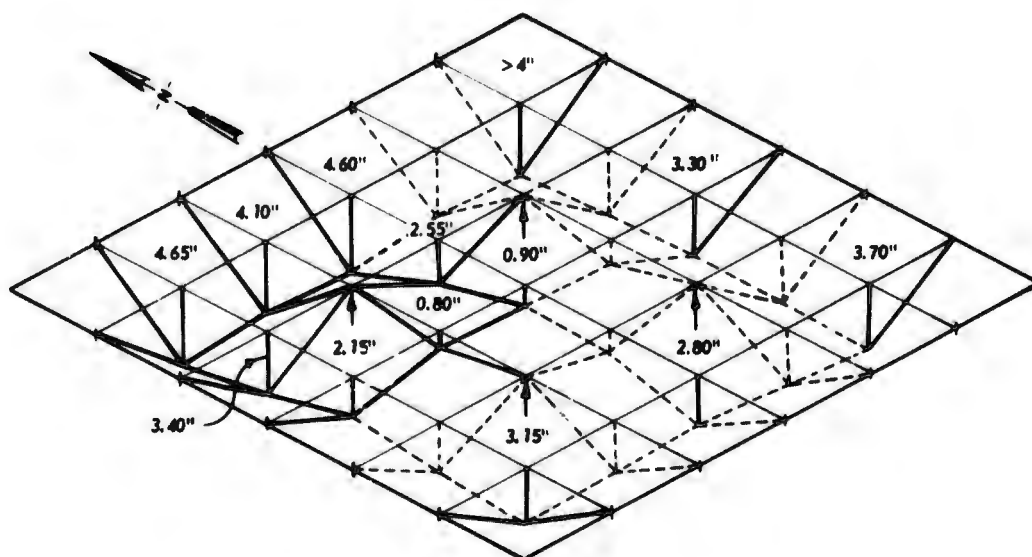


Figure 6.4 Slab deflections during Dynamic Test II (sheet 1 of 2).

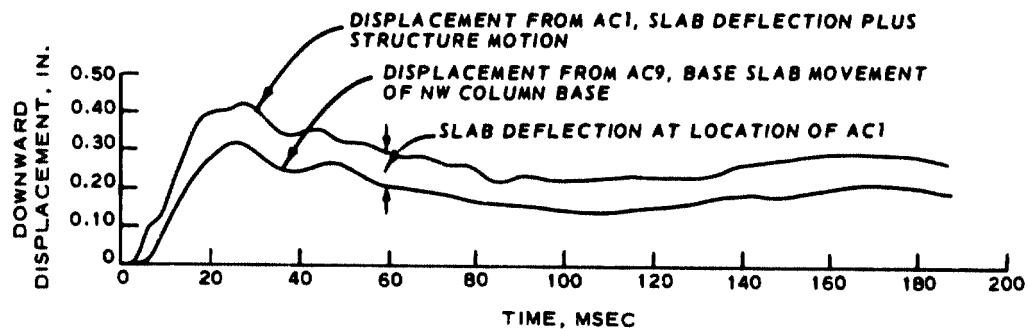


c. SLAB DEFLECTION AT $t = 45$ MSEC

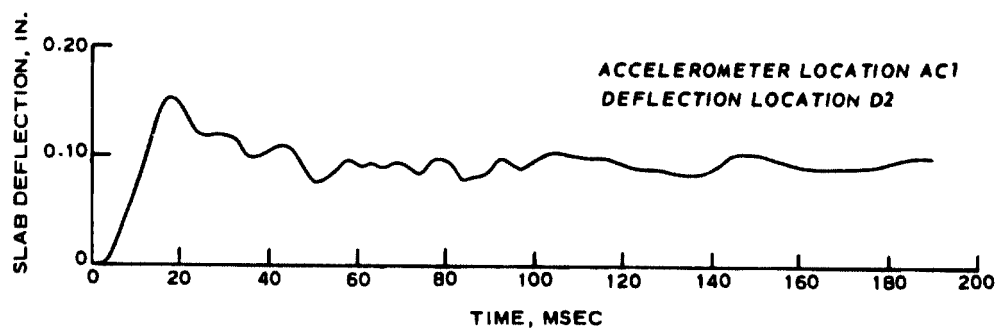


d. SLAB DEFLECTION AT $t = 50$ MSEC

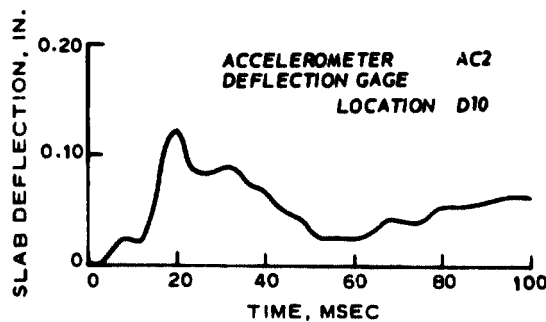
Figure 6.4 (sheet 2 of 2).



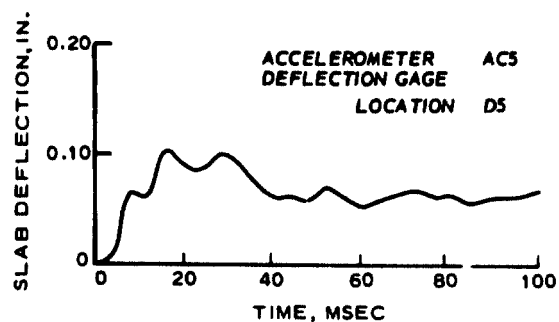
a. DETERMINATION OF SLAB DEFLECTIONS



b. DEFLECTION AT CENTER OF OUTER PANEL



c. DEFLECTION AT CENTER LINE OF COLUMN-TO-WALL LINE



d. DEFLECTION AT CENTER OF CENTER PANEL

Figure 6.5 Slab deflection plots for Dynamic Test I.

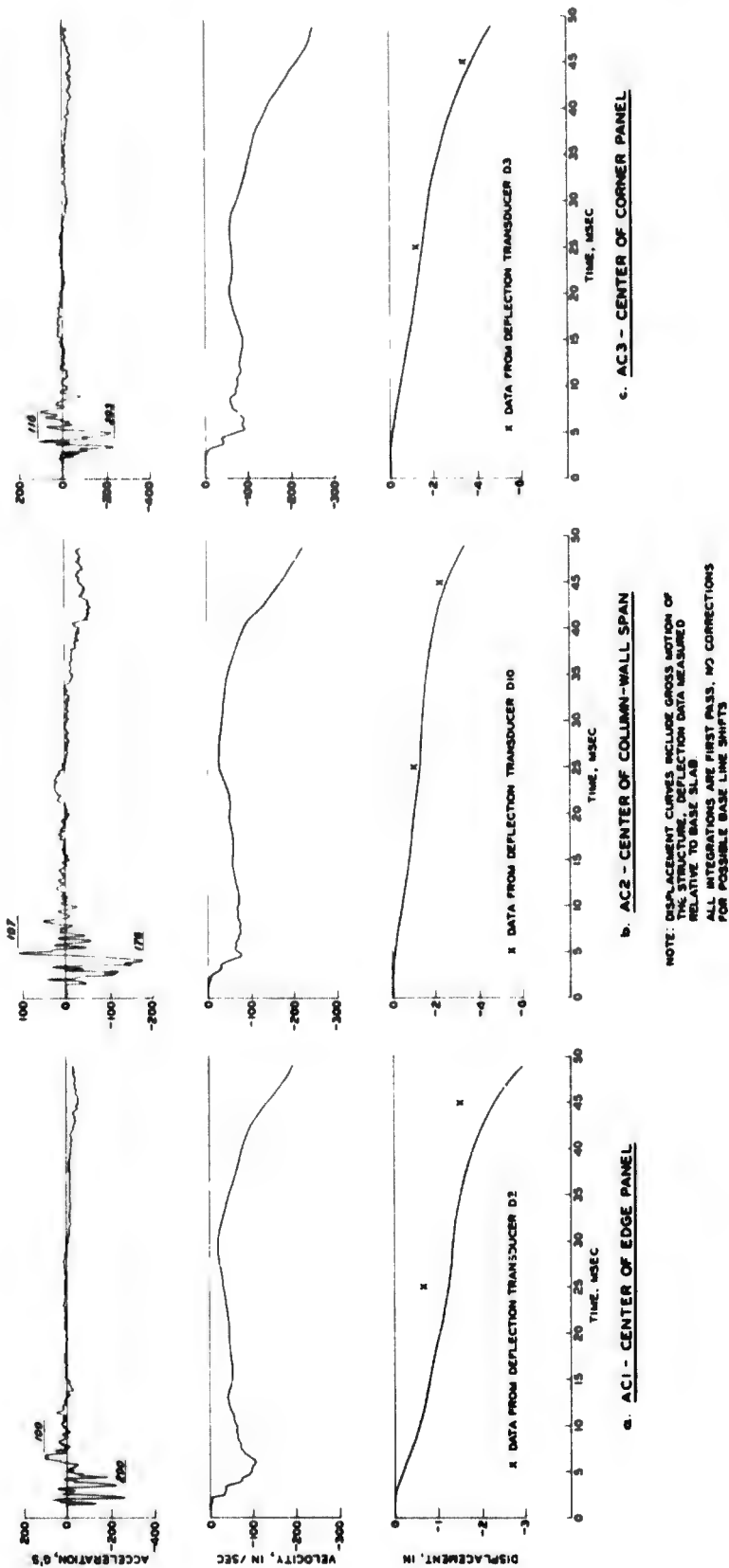


Figure 6.6 Acceleration-, velocity-, and displacement-time curves, Gages AC1, AC2, and AC3, Dynamic Test II.

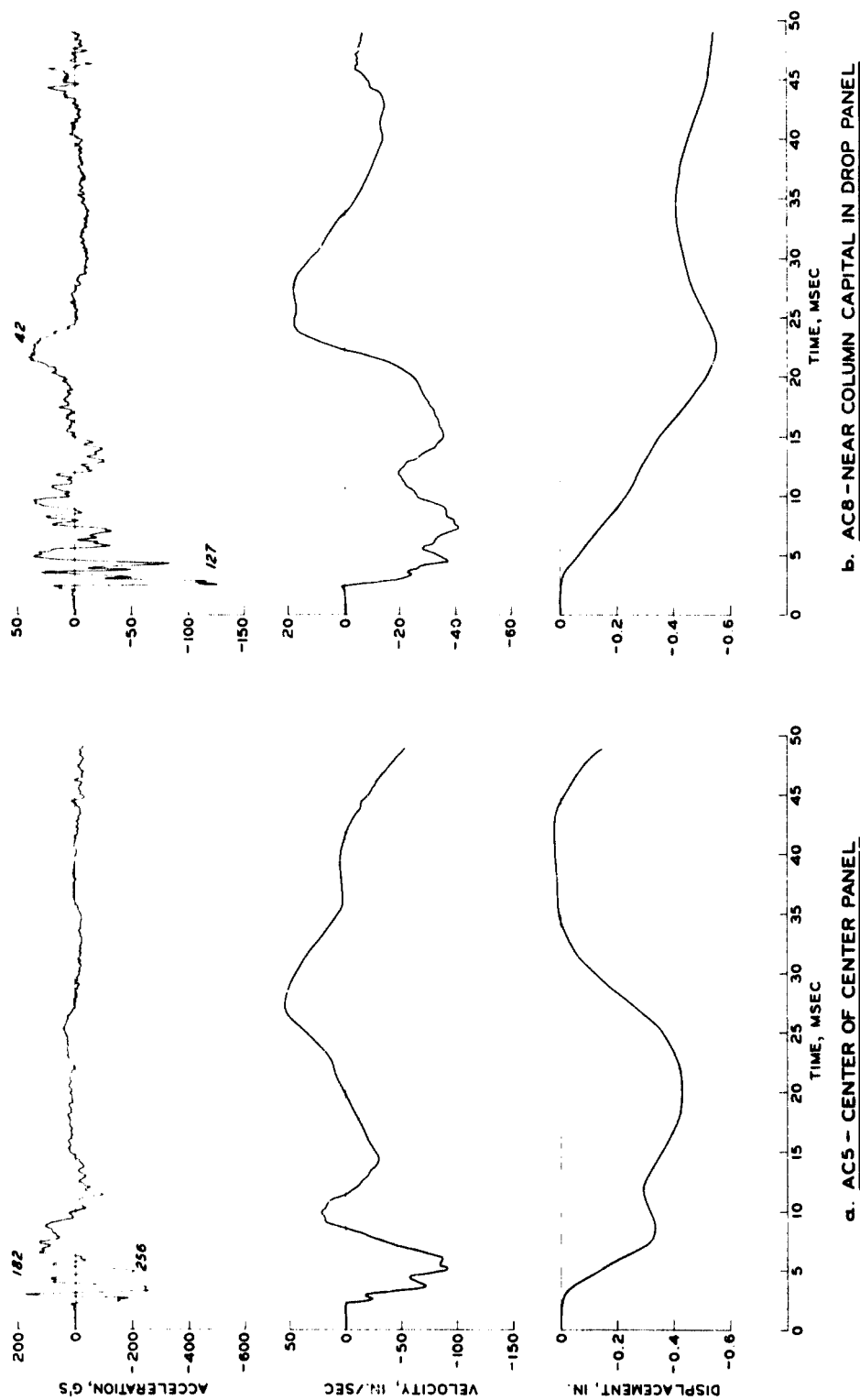
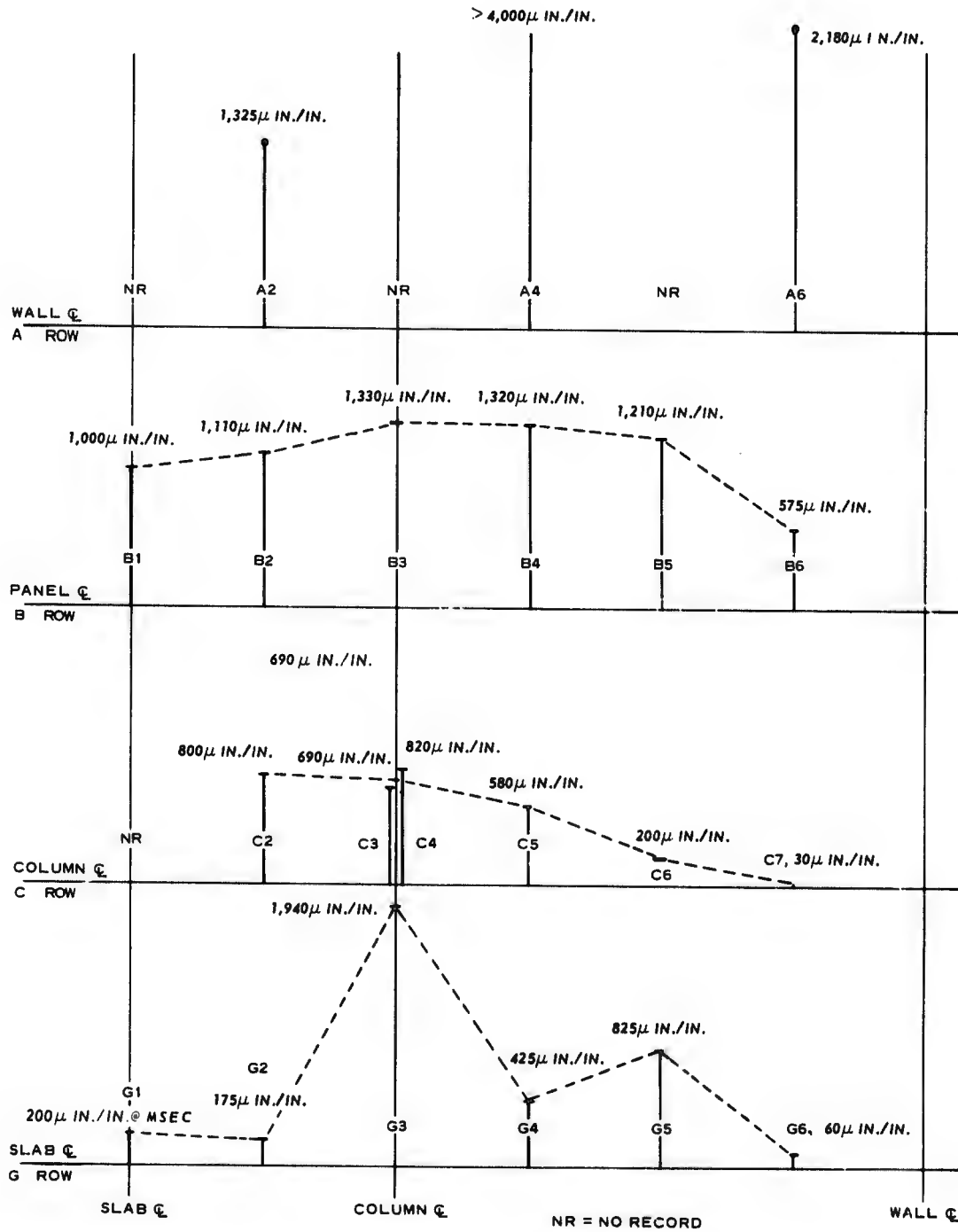


Figure 6.7 Acceleration-, velocity-, and displacement-time curves, Gages AC5 and AC8, Dynamic Test II.



NOTE: TIME OF PEAKS VARY FROM THE
NOMINAL 20 MSEC VALUE. SEE
FIGURES B.10 THROUGH B.12

Figure 6.8 Tensile reinforcement strain distribution at peak strains at approximately 20-msec elapsed time, Dynamic Test I.

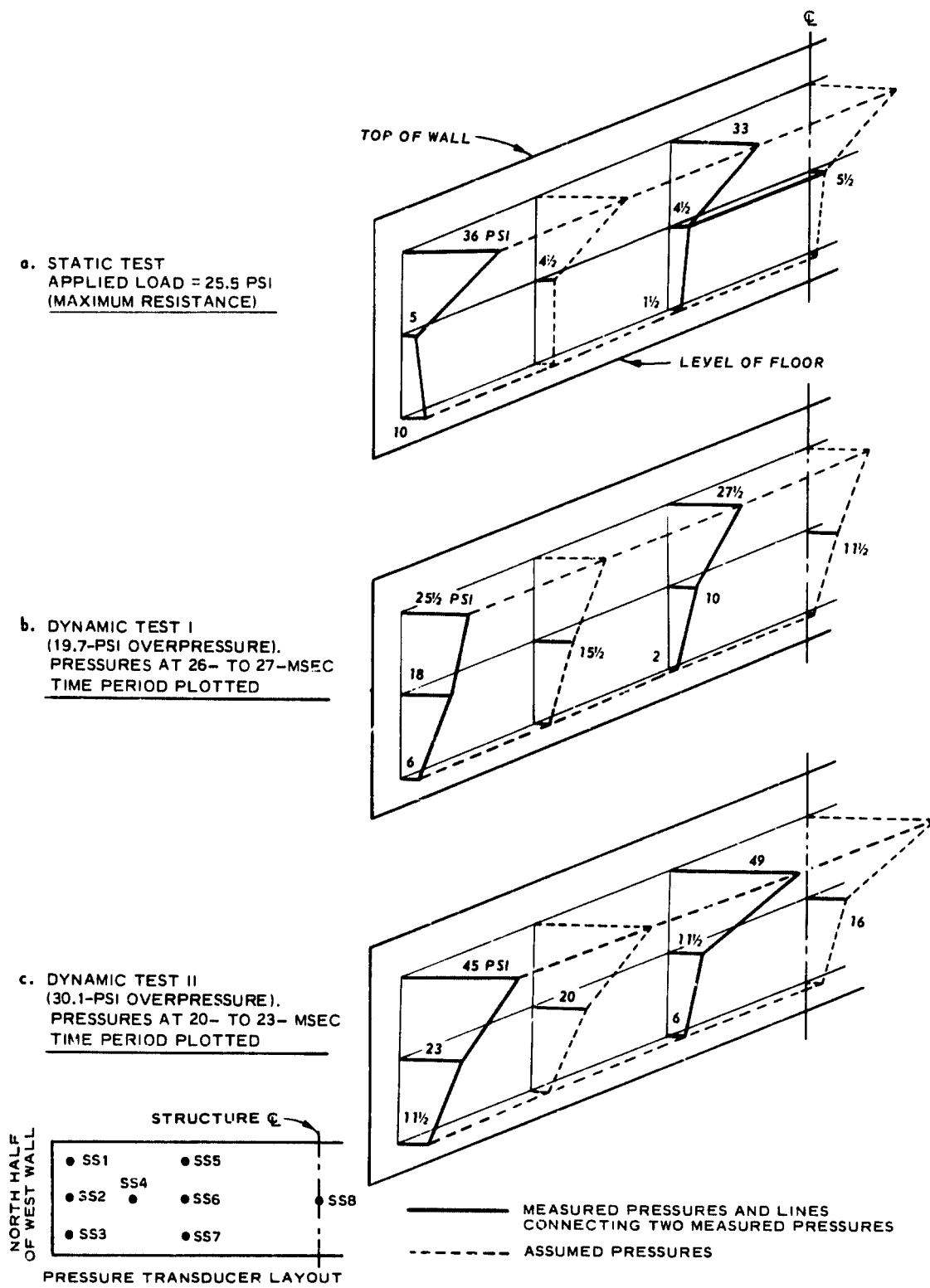


Figure 6.9 Soil pressures acting on the west wall of the model.

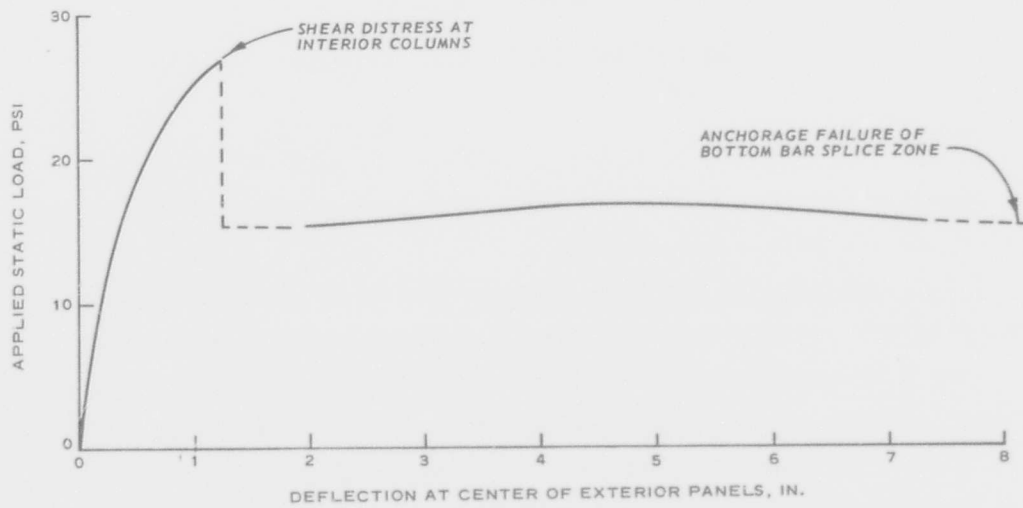


Figure 6.10 Resistance curve for statically tested slab.



Figure 6.11 Debris from the southwest column area, Dynamic Test II.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

Based on the results of the tests discussed herein, the following conclusions are believed warranted:

7.1.1 Adequacy of the Slab System. The slab system designed and modeled in the project exceeded the design requirements of providing a roof over a basement shelter area that could insure survival of at least 85 to 95 percent of the shelter occupants when the shelter area is exposed to a 15-psi airblast overpressure level. This level of protections probably could be achieved with the slab system, as designed, up to the 25-psi level and certainly at an overpressure level of 20 psi.

7.1.2 Slab Type. A slightly modified flat slab system closely resembling a conventional slab designed for heavy loads can adequately resist the blast load of interest and is cost-competitive with one- and two-way slab systems including beams between columns. The slab system tested fits well within the "fully slanted" shelter concept where protection from blast loads is provided at minimum cost through the use of minor modifications to parts of new buildings being constructed primarily to serve noncivil defense functions. Desirable large-deflection behavior appears to be more difficult to achieve with the flat slab system than with the two-way system, as was expected.

7.1.3 Design of the Slab. The use of design methods closely tied to standard methods for designing slowly loaded structures and not including elaborate dynamic analyses is feasible for the routine design of slab structures to withstand blast loadings of relatively long durations (durations on the order of 1 sec or more) and overpressures as high as 30 psi. For slabs over basement areas subjected to long-duration blast loading, the dynamic load factors appear to be nearly offset by the increased material strengths due to strain rate effects that occur at the time of the maximum dynamic load effects. A complex dynamic analysis for each structure is not needed; greater design uncertainties will

usually exist in evaluating the magnitude and effects of slab edge restraint and in-plane loading and in adequately designing the so-called "details" of the slab system.

7.1.4 Flexural Strength of Statically Loaded Slab. The observed static strength of 26.6 psi appreciably exceeded the design strength. This increase is believed to have resulted from conservative assumptions in the flexural design procedure regarding the effective slab depth and yield-line placement and, most importantly, the effect of in-plane compressive forces acting within the slab system. At maximum load, the flexural strength of the slab was approached, although the yield-line pattern was not completely developed.

7.1.5 In-Plane Slab Forces. The in-plane compressive slab forces resulted primarily from the horizontal soil pressure acting on the perimeter wall of the shelter. The computed slab strength due to the in-plane loads accounted for about 30 percent of the 48 percent increase in the slab strength of the statically tested model over the computed yield-line strength disregarding in-plane forces. The magnitudes of the in-plane forces are difficult to evaluate and will vary with the type, compaction, and water table conditions of the soil adjacent to a prototype structure of the type modeled.

7.1.6 Slab Moment Distribution. The strength of the wall was sufficient to allow positive moment yielding to develop around the perimeter of the slab. Positive moments along the center row of panels were considerably smaller relative to the design moments than those in the outer panels.

7.1.7 Shear Strength. The ultimate nominal shearing stress supported at a section $d/2$ out from the column capital of the statically loaded model was 377 psi, 5.83 times $\sqrt{f'_c}$. This value was higher than might be expected because of the beneficial effects of compressive in-plane slab forces, the round rather than square column capital outline, and the increased slab flexural strength. Punching shear distress developed around the columns as the flexural capacity of the slab was approached and limited the resistance of the statically tested model. Shear strength of the slab at the wall appeared adequate, and no need was seen for stirrups throughout the slab.

7.1.8 Continuity of the Slab After Maximum Resistance. The bottom reinforcement extending through the column capital area was successful in holding the slab-column areas together by forming a suspension net configuration in these areas after the shear crack opened around the column. The rupture of the slab, which terminated the static test, resulted from the anchorage failure of bottom bar splices near the column. Anchorage conditions at the splice locations were greatly aggravated because these locations were mostly within the local tensile membrane zones around the column.

7.1.9 Large Deflection Behavior. The resistance of the statically loaded slab system dropped to 15 to 17 psi after shear distress developed in the column areas. This decrease resulted largely from the effective loss of much of the negative moment capacity in the column areas. General tensile membrane forces sufficient to significantly increase the resistance of slab system did not develop because of the expected inability of the wall to anchor these tensile forces and the lack of stiff beams along the column lines. Deflections of the model slab away from the failure area exceeded 7 inches, 13 percent of the column spacing.

7.1.10 Dynamic Capacity of the Slab. The level of dynamic overpressure needed to fail the slab system was bounded by the 19.7- and 30.1-psi values of the two dynamic tests. The test results suggest that the dynamic capacity was slightly closer to the upper value, a value close to the static strength of the model. The 19.7-psi test resulted in only minor cracking of the slab and local reinforcement yielding around the perimeter of the slab.

7.1.11 Characteristics of the Slab Resistance Curve. For the long-duration dynamic loads of interest, the extensive slab ductility maintained at a resistance considerably less than the maximum level would be of little or no benefit. A long-duration blast loading sufficient to cause shear distress around the column areas would not decrease fast enough to allow the lower branch of the resistance curve at large deflections to halt the downward slab motion. Consequently, a small difference in overpressure level could change the postloading condition of a given slab from one of minor damage to one of nearly total

destruction. The large ductility and resulting energy absorbing capacity of the slab would be usable for resisting an impact or other short-duration dynamic loading.

7.1.12 Benefits of the Slab Drop Panels. The drop panels around the column capitals had mixed benefits on the slab behavior. Their contribution to increasing the shear strength of the column areas was sizable, but the drop panels became useless after shear distress developed around the columns. Most of the drop panel concrete below the bottom reinforcement mat dropped from the slab when local membrane action developed near the columns. As a result, the drop panel areas were the source of much debris, some of considerable size. Omission of the drop panels would have resulted in a resistance curve shape close to the more desirable elastoplastic shape because of the decreased shear capacity and smaller value of the maximum resistance peak. However, this omission would decrease the overpressure level needed to fail or severely damage the slab system.

7.1.13 Importance of the Walls. The walls around the perimeter contributed to the slab strength in two major ways: (1) they provided the restraint needed for the ultimate moment of the edge of the slab to be fully utilized, an effect most difficult to achieve when the edge of the slab is supported on spandrel beams, and (2) they served as a means of gathering the soil forces, thus providing in-plane compression on the slab.

7.1.14 Design of the Wall. The wall must be designed as a structural member. The design of the wall reinforcement must provide adequate resistance to the lateral soil loads and slab moments acting on the top of the wall. The use of the slab to support the top of the wall is important for economical design of the wall itself in addition to the resulting beneficial slab in-plane forces. The design of a wall with adequate lateral strength to allow general tensile membrane action to be reached in the slab is not practical with the wall configuration used. The fixity of the wall base in the model exceeds what could reasonably be expected in a full-scale structure.

7.1.15 Soil Pressures. The measured soil pressures acting on the walls were greater than the overpressure near the top of the slab and

decreased at a decreasing rate to low values near the base of the wall. The soil forces on the walls before the slab began to pull on the walls could be roughly represented as decreasing linearly from a value of 125 percent of the applied load near the top of the wall to less than 25 percent at the base of the wall. Soil forces at midheight of the wall, especially for the static test, were smaller than the values from the linear variation described above. The slab moments acting on the top of the wall were sufficiently large to deflect at least the upper portions of the wall against the surrounding soil, thus mobilizing some passive soil pressure. Magnitudes of soil forces acting on the wall would be influenced by soil properties, compaction, and water table conditions; hence, soil pressures could vary significantly from those resulting with the fine, clean sand backfill used in the model tests.

7.1.16 Column Design. The column distress observed in Dynamic Test II underscores the importance of conservative column design. The columns of the model were barely adequate. The magnitude of the column loads, but not the column strength, is influenced by in-plane forces. The use of conservative assumptions in the slab design that act to increase the slab capacity cause the column design to be less conservative. Unless the effects that could possibly increase slab strength are considered, the failure mode of the entire shelter area could easily be controlled by column failures.

7.2 RECOMMENDATIONS

7.2.1 Improving the Slab Design. Recommendations and suggestions for improving the slab design in addition to those implied in the previous section include those listed below. Several items listed would improve the large deflection behavior of the slab, but would be of practical use only if steps were taken to insure that the slab resistance at large deflection would be near the maximum slab resistance.

To allow the maximum deflection capability of the slab, including the ability to develop and sustain local membrane action along supporting walls and around columns, bottom reinforcement should be used throughout the entire slab. If lap splices rather than positive methods

such as mechanical connections and welding are used, part of the reinforcement should be spliced on both sides of the columns so that some bars from each span would extend entirely through the column capital area. The probable loss of cover below the bottom bar mat when a local tensile membrane zone forms around the column should be considered. Although careful detailing of the reinforcement would not have significantly increased the strength of the slab model at large deflections, it could have helped to control the size of the rupture openings in the slab surface.

The role of the drop panel should be critically examined, especially if an elastoplastic resistance curve is desired. As a minimum, some small bar or light reinforcing mat should be placed in the bottom of a drop panel and anchored into the rest of the slab (Figure 7.1a) to prevent the drop panel from becoming debris. A slab geometry with the slab thickness uniformly increasing from at or near the span centerline to a maximum depth of the column should be considered as an alternative to using a drop panel.

Round column capitals should be used to avoid the concentrations of shearing forces that occur at the corners of rectangular supports. Placement of one or more hoops near the top of the capital (Figure 7.1a) to prevent the otherwise unreinforced capital from splitting, especially when local tensile membrane action forms and causes the slab to be hung from the perimeter of the capital, is also recommended.

The use of several forms of shear reinforcement in the column areas should be considered not only to add to the shear strength but also to efficiently tie the structure together if shear cracking does form. Bending up of some bottom bars across potential shear cracks (Figure 7.1b) would be beneficial. Minimizing the size of crack opening needed to mobilize the reinforcement suspension mode of shear transfer into the column would act to decrease the moment capacity drop in the column area. It should be remembered that shear reinforcement in column areas can force the shear failure to occur outside the shear reinforcement zone, and the substantial drop in slab strength with the completion of shear cracking could still form unless shear reinforcement is spread

over a fairly wide ring around the column capital. Shear reinforcement for conventional construction-type slabs is receiving more attention and in the future will no doubt become more practical.

The use of interior walls in this type slab system should be considered as a means of increasing the strength of the system. For large shelter areas, structural walls set between columns could help contain a local panel failure or rupture that otherwise might result in a failure over a large area due to the type of column tilting seen in the model and caused by unbalanced lateral loads on the columns near the distressed area.

The exterior walls and adjoining perpendicular interior walls could be designed to serve as a part of an anchorage system for a general tensile membrane action at large deflections. Such a scheme is shown in Figure 7.1c.

Portions of the slab adjacent to walls could be intentionally designed as deep horizontal beams or diaphragms to help support membrane action and prevent the outside walls from being pulled or pushed in. The design depth of the slab would usually offer adequate width for this deep horizontal beam. Such a beam could span several bays; indeed, three bays were spanned in the models by such a beam, although this behavior was not considered in the design. These "beams" would allow many of the short structural walls shown in Figure 7.1c to be omitted providing those remaining could develop the needed reactions for these horizontal beams.

The center portions of the column strip could also be designed as wide horizontal beams to help anchor tensile forces, contain local failures, and act as struts spacing the columns. This would approach using a two-way slab system with very shallow but wide beams, but with these beams designed to withstand vertical and horizontal loads as well as axial loads.

7.2.2 Future Research. Tests of other slab configurations are needed to study and demonstrate their performance. An experimental study similar to the study reported using only a two-way slab system with beams should be considered. The study of the flat slab system

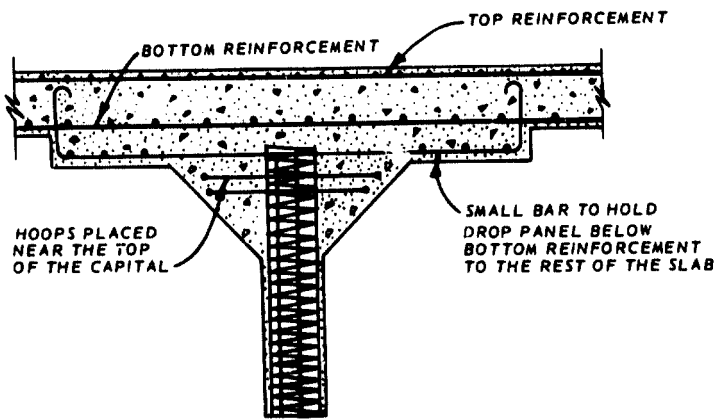
showed only that the flat slab system will work, not that it is necessarily the best system. Hybrid slab systems designed to best combine the advantages of the flat slab and two-way slab systems should also be explored.

The behavior in shear of the slab regions close to columns when the flexural strength is approached and when deflections are large is not well understood and needs further study. It appears that the development of shear distress around the columns may be difficult to prevent when flexural rotations in the column area are large. A very significant but undefined dependency exists between the shear and flexural behavior of slabs weaker in flexure than in shear. The so-called "secondary shear" failure of such slabs is a matter of primary importance for structures where total collapse is the measure of failure.

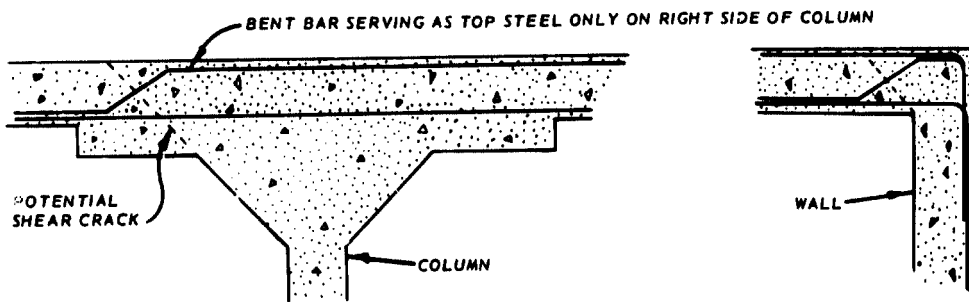
More work is necessary to adequately understand and design practical shear reinforcement in column areas and to evaluate the effects of support shape and eccentric loadings on the shear strength of column connections.

The importance of details in designing efficient blast-resistant slab systems must be stressed, and guidelines for detailing such structures should be established.

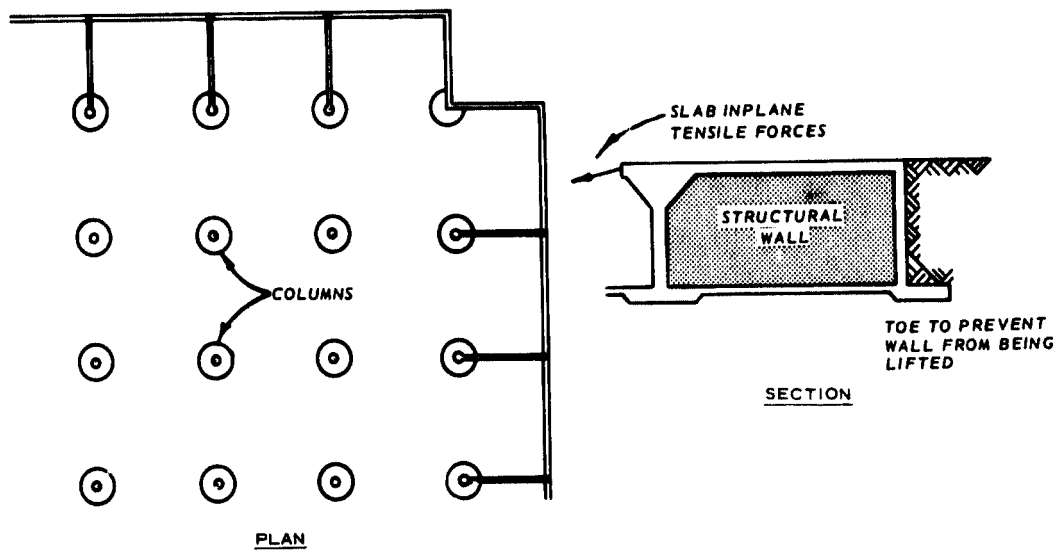
Because a blast-resistant structure usually is not considered to have failed until at or near its total collapse, concepts and methods for developing strength at large deflections such as utilizing tensile membrane action are of a much more practical nature than is the case for conventional structures. Future identification and development of these nonconventional concepts and methods, along with further study of how structures behave at failure and collapse, will result in significant advances in the design and analysis of blast-resistant construction.



a. DETAILS OF THE CAPITAL AND DROP PANEL



b. BENT BARS FOR STRUCTURAL CONTINUITY



c. METHOD FOR ANCHORING SLAB TENSILE FORCES

Figure 7.1 Possible slab improvements and modifications.

APPENDIX A
RECORDED DATA FOR STATIC TEST

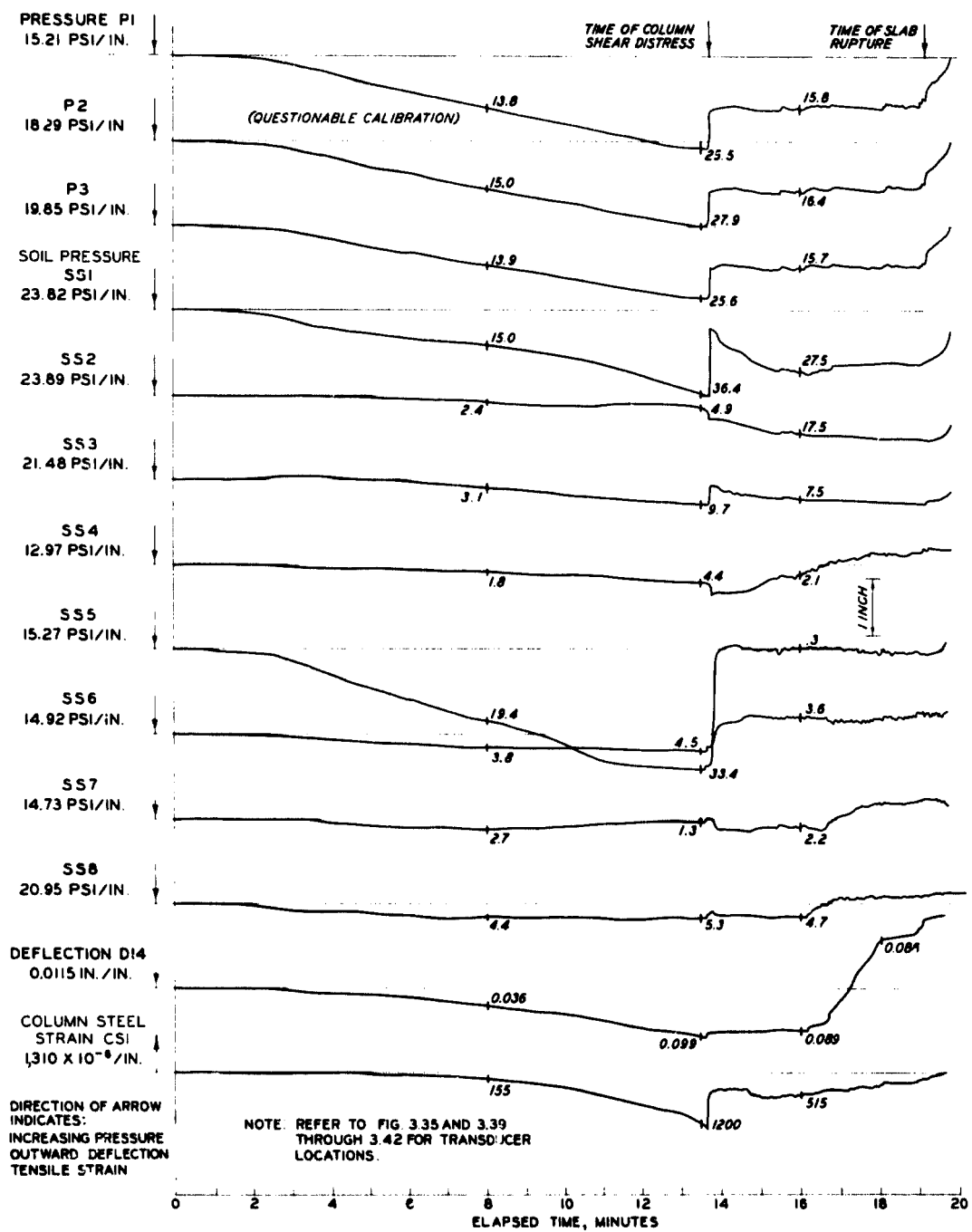


Figure A.1 Tracings from Recorder 1, static test.

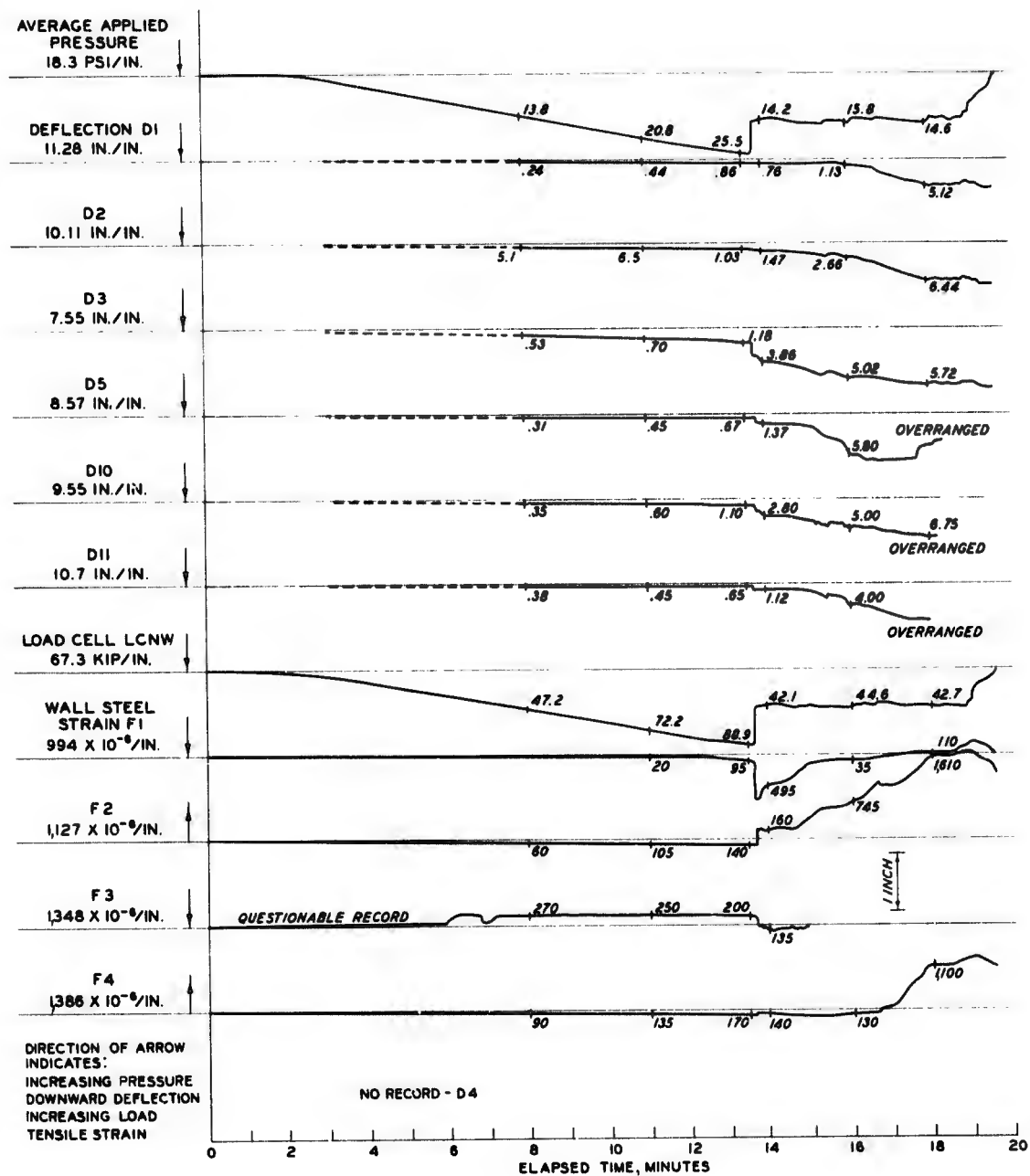


Figure A.2 Tracings from Recorder 2, static test.

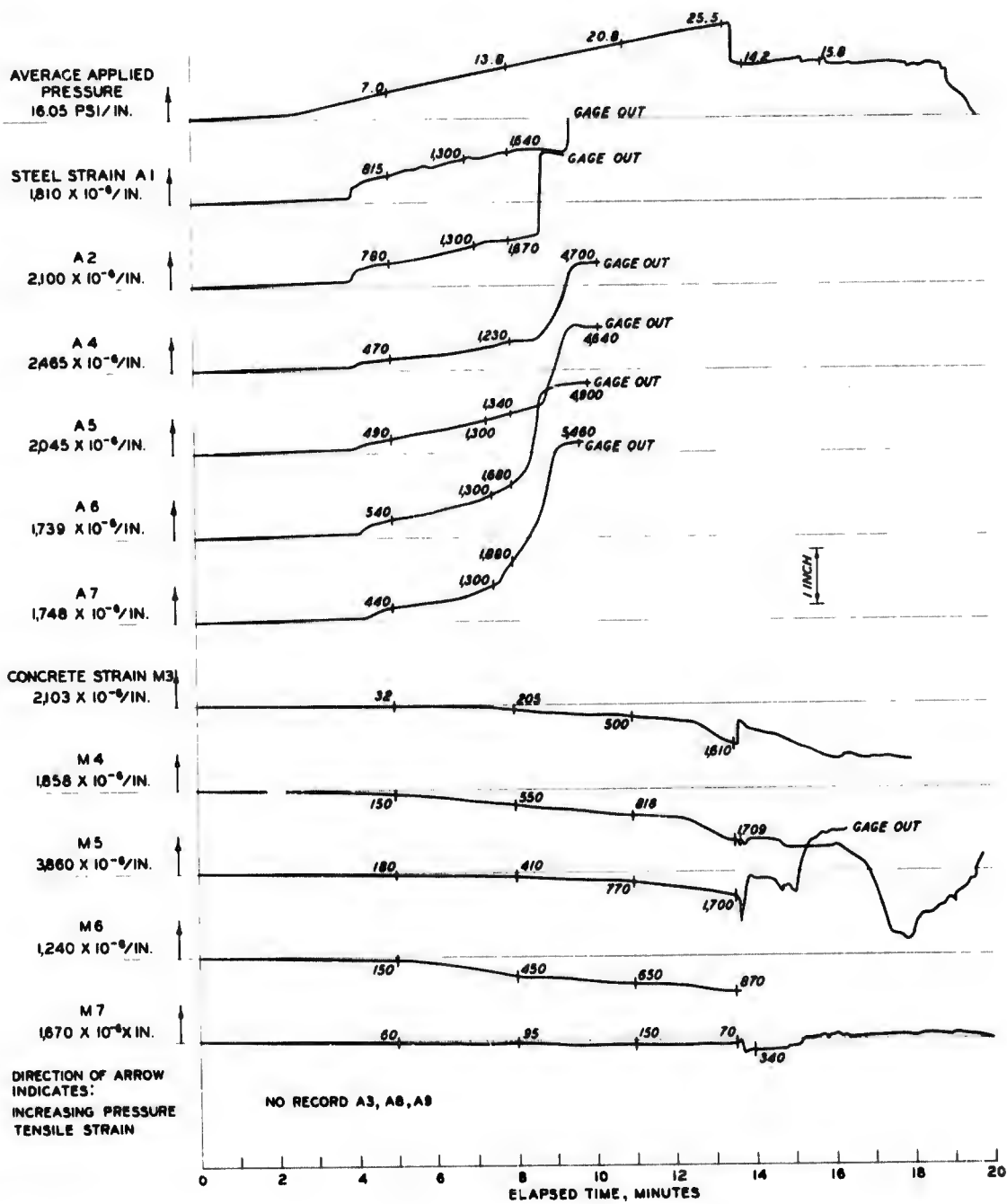
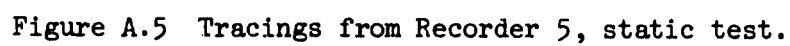


Figure A.4 Tracings from Recorder 4, static test.



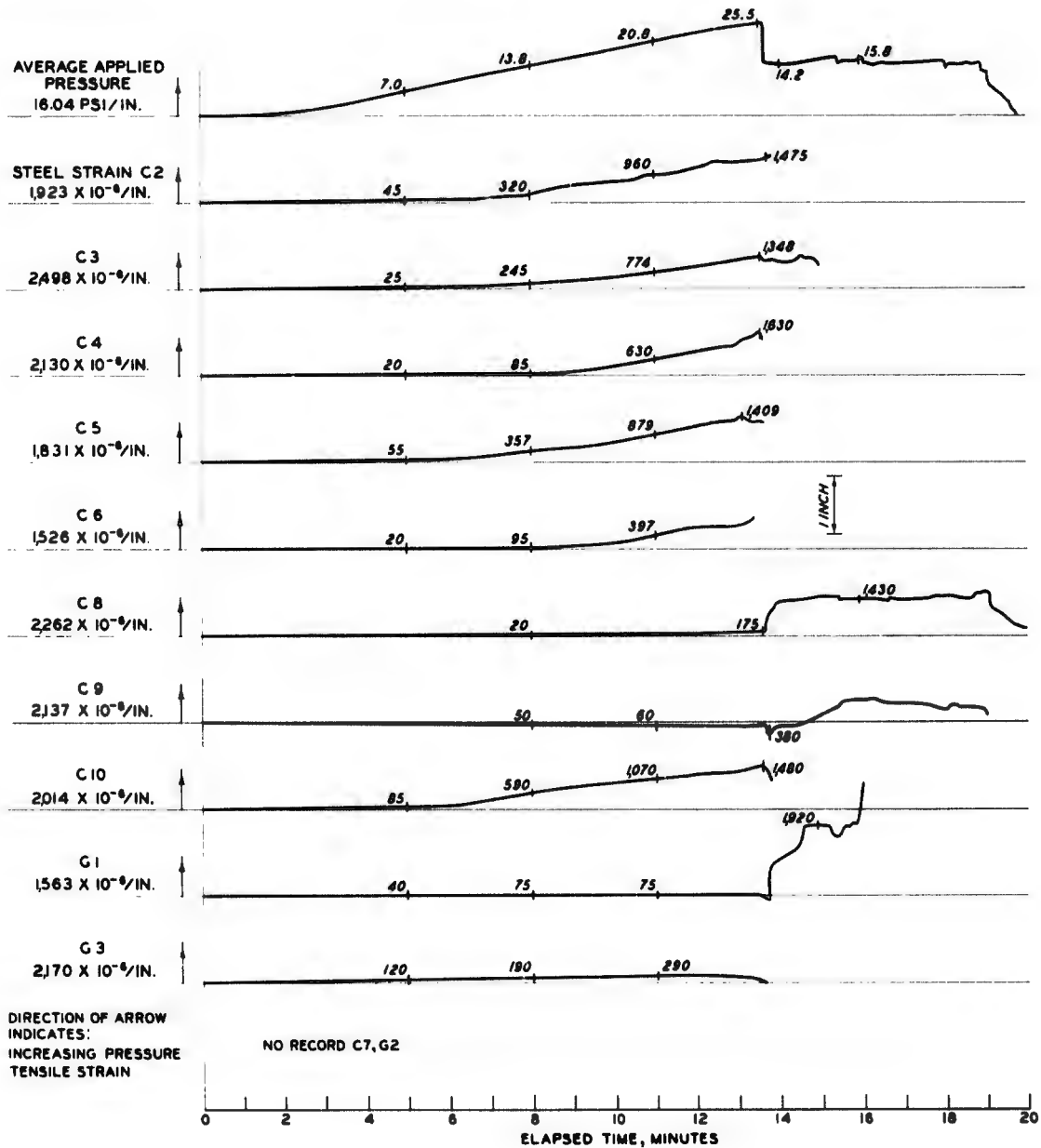


Figure A.6 Tracings from Recorder 6, static test.

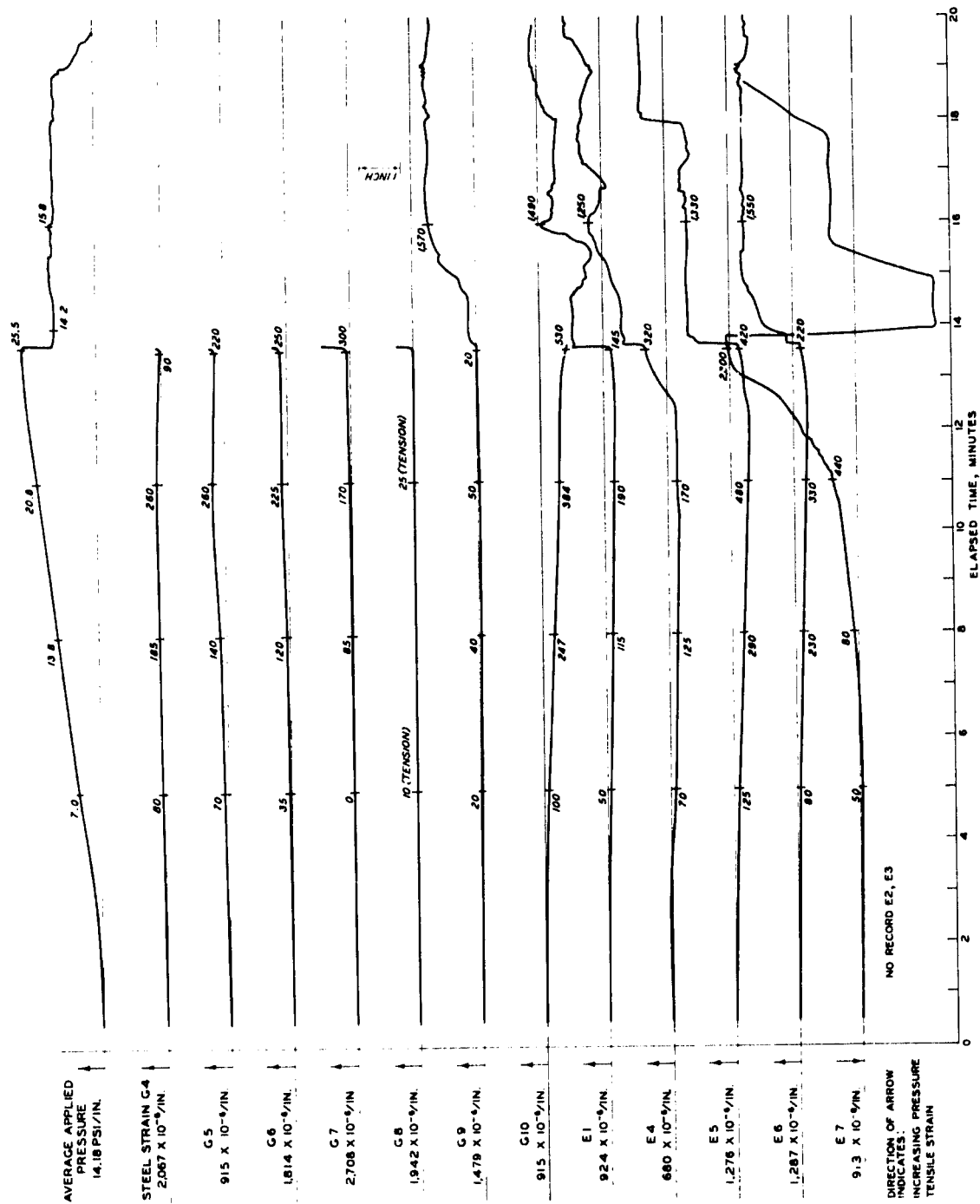
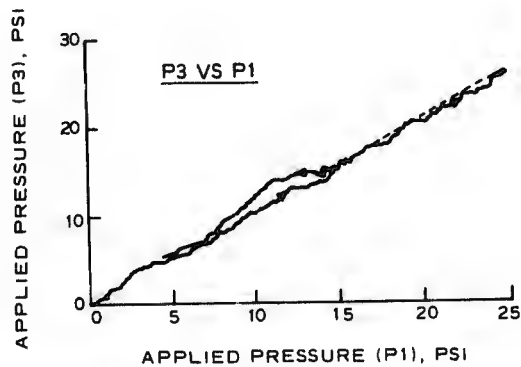
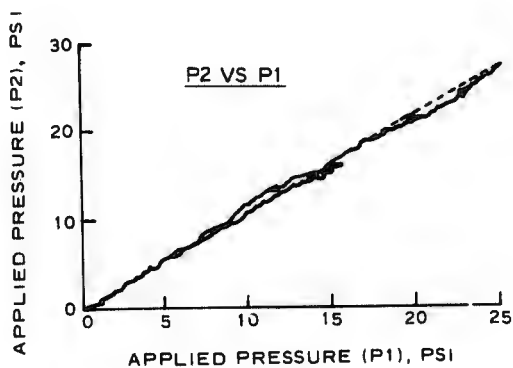
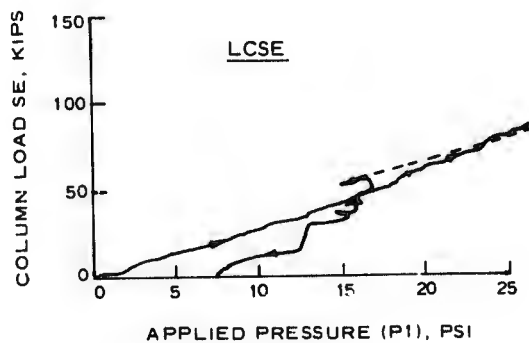
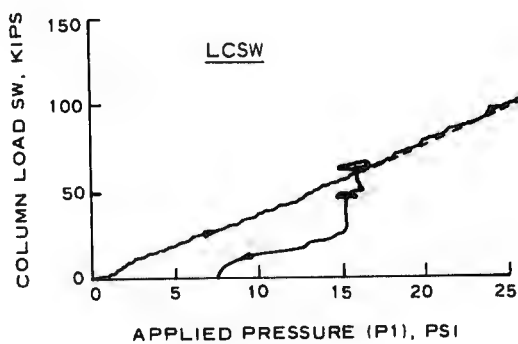
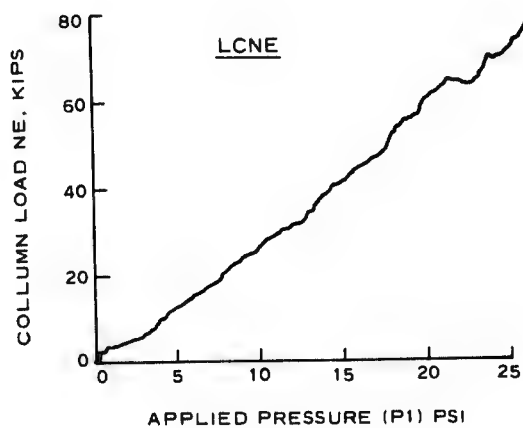
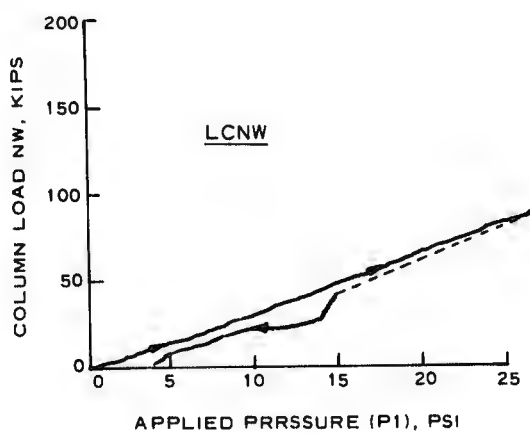


Figure A.7 Tracings from Recorder 7, static test.



a. APPLIED PRESSURE RECORDS



b. COLUMN LOAD CELL RECORDS

Figure A.8 Applied pressure and column loads, static test.

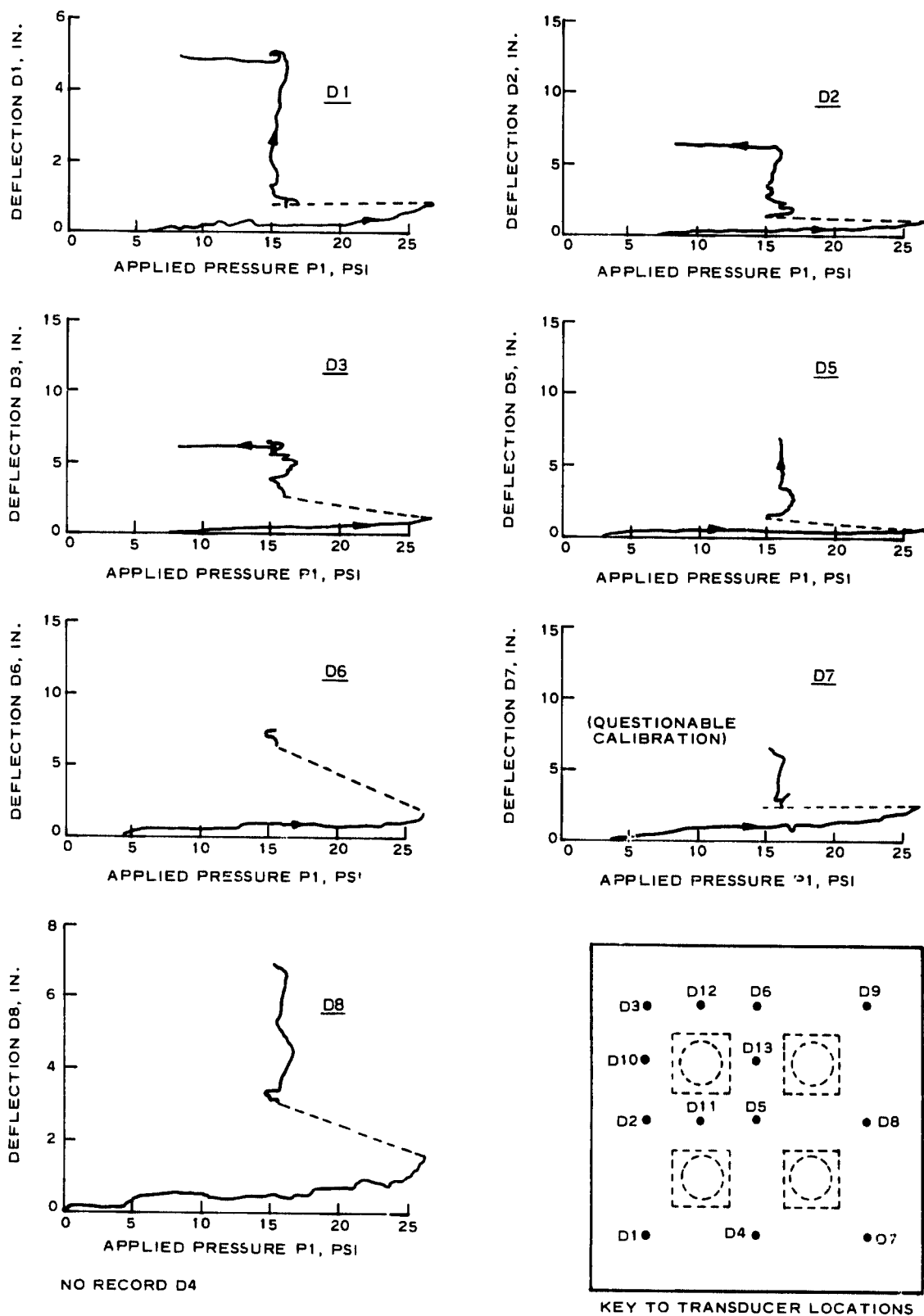
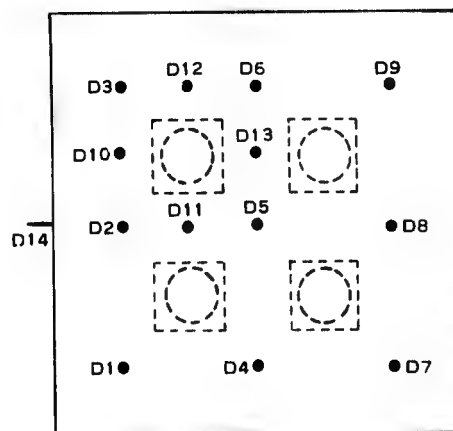
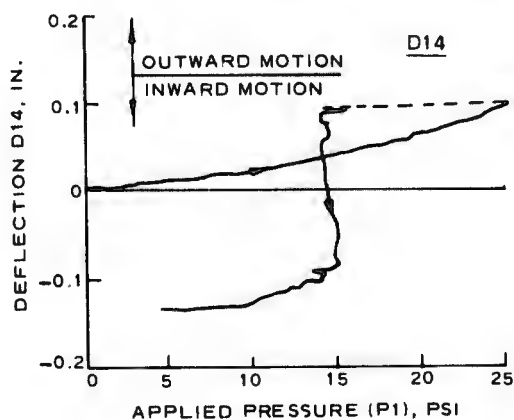
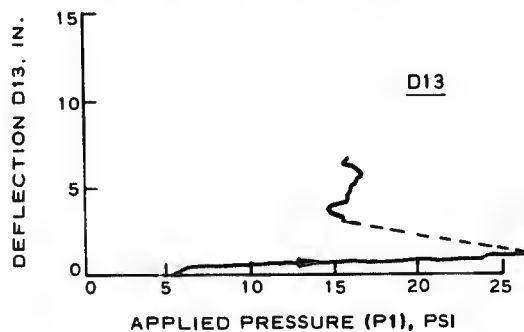
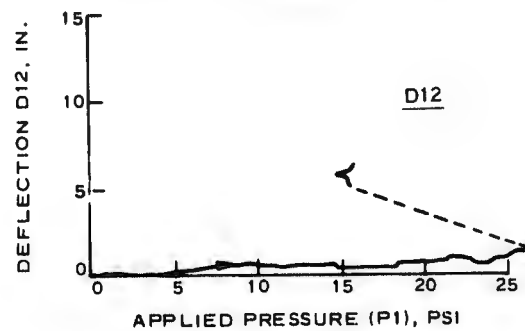
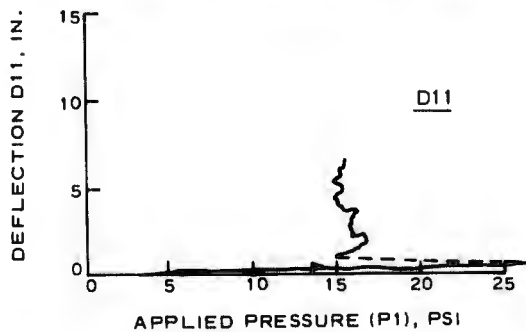
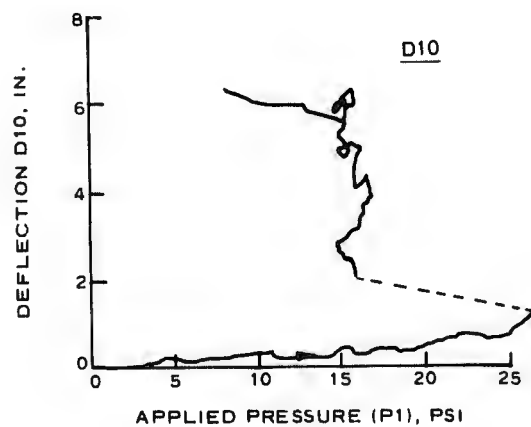
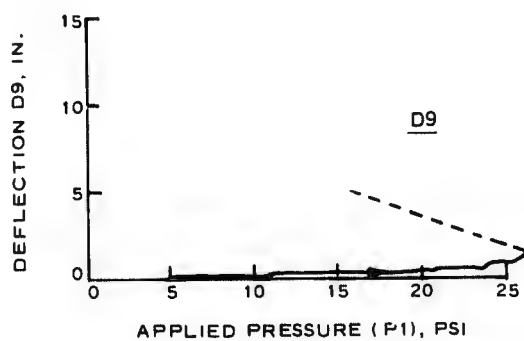


Figure A.9 Deflections, Gages D1 through D3 and D5 through D8, static test.



D14 MOUNTED HORIZONTALLY AT MIDHEIGHT, CENTER OF WEST WALL

KEY TO TRANSDUCER LOCATIONS

Figure A.10 Deflections, Gages D9 through D14, static test.

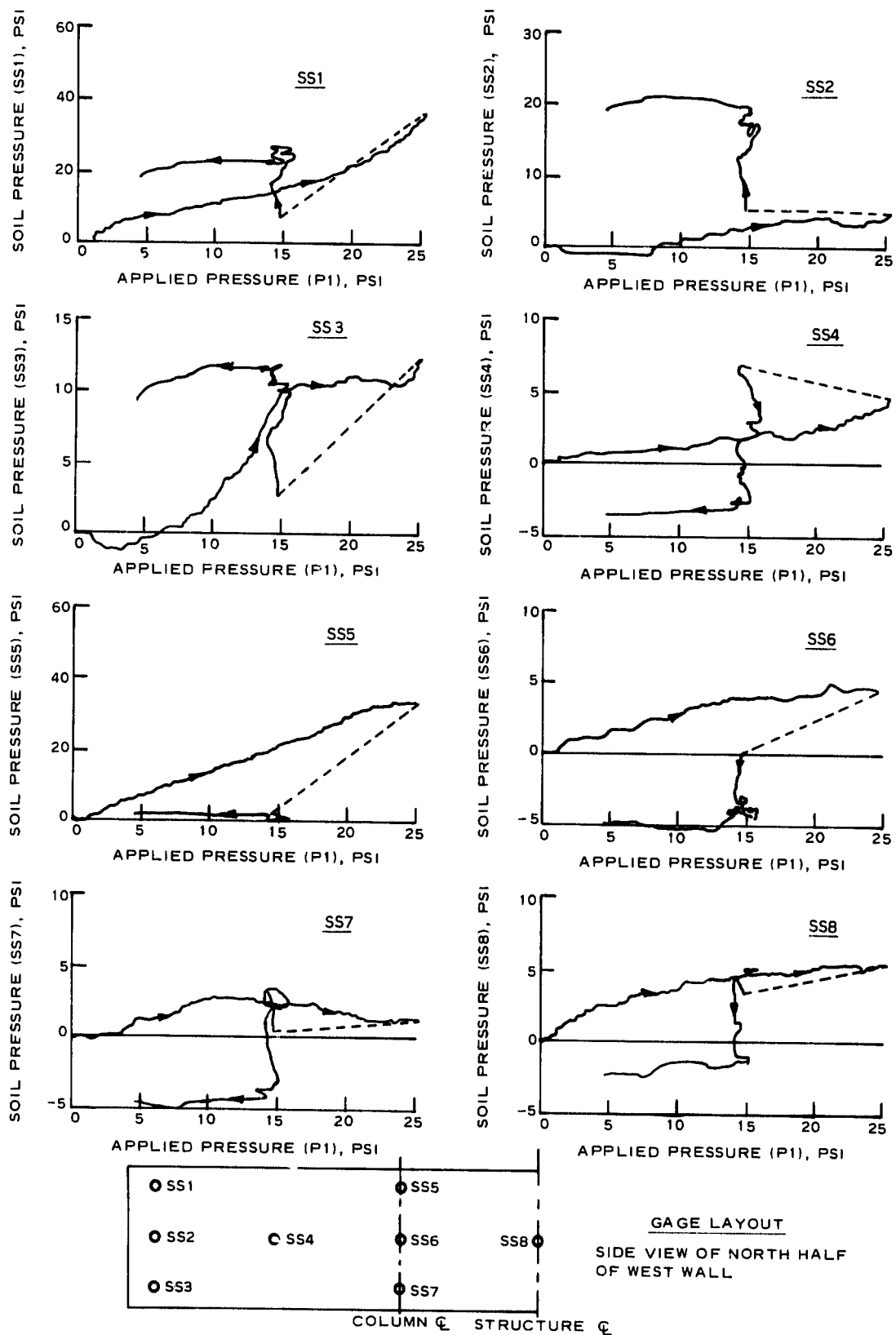


Figure A.11 Soil-structure interface pressures, static test.

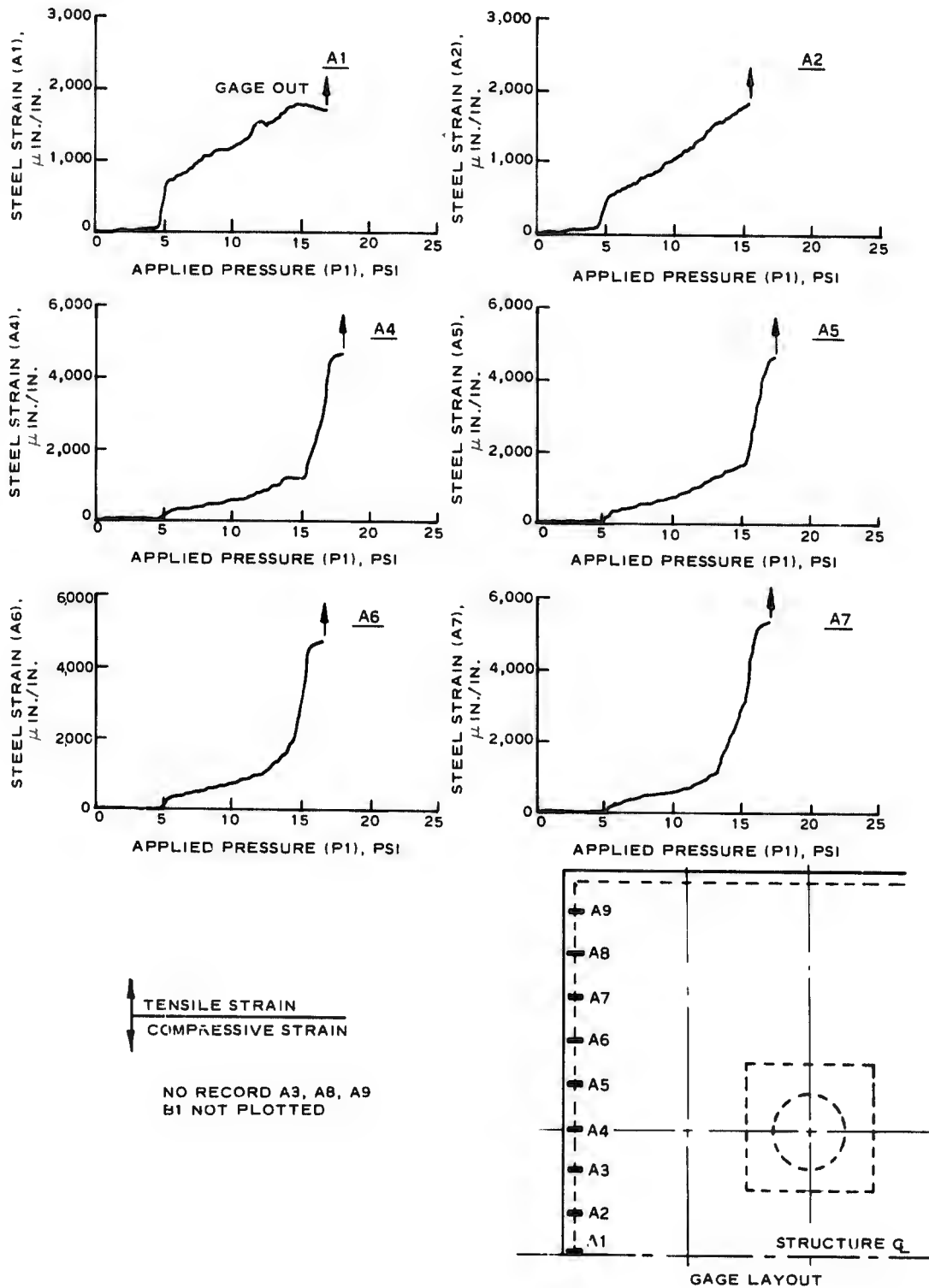


Figure A.12 Tensile reinforcement strains, Gages A1 and A2 and A4 through A7, static test.

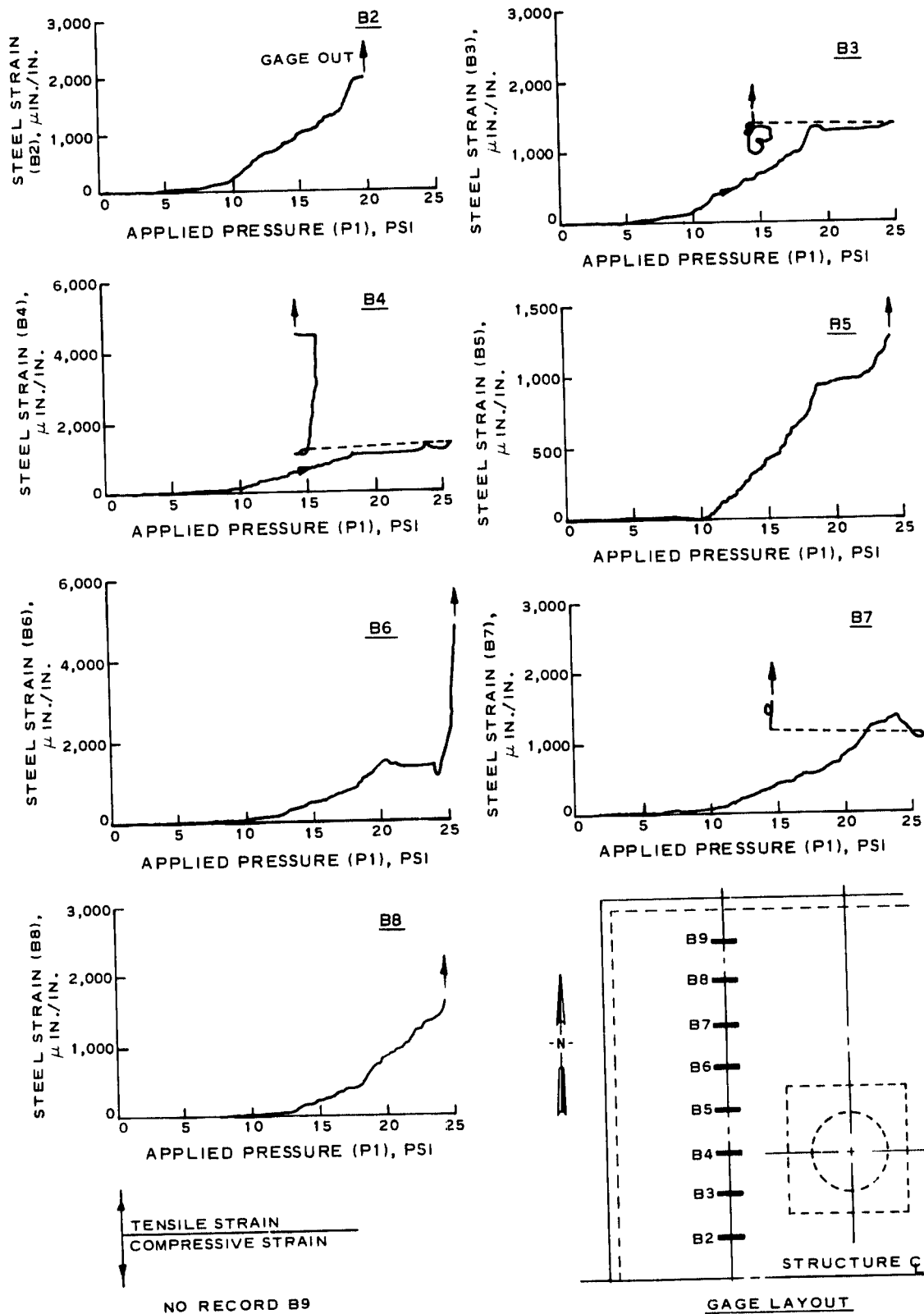


Figure A.13 Tensile reinforcement strains, Gages B2 through B8, static test.

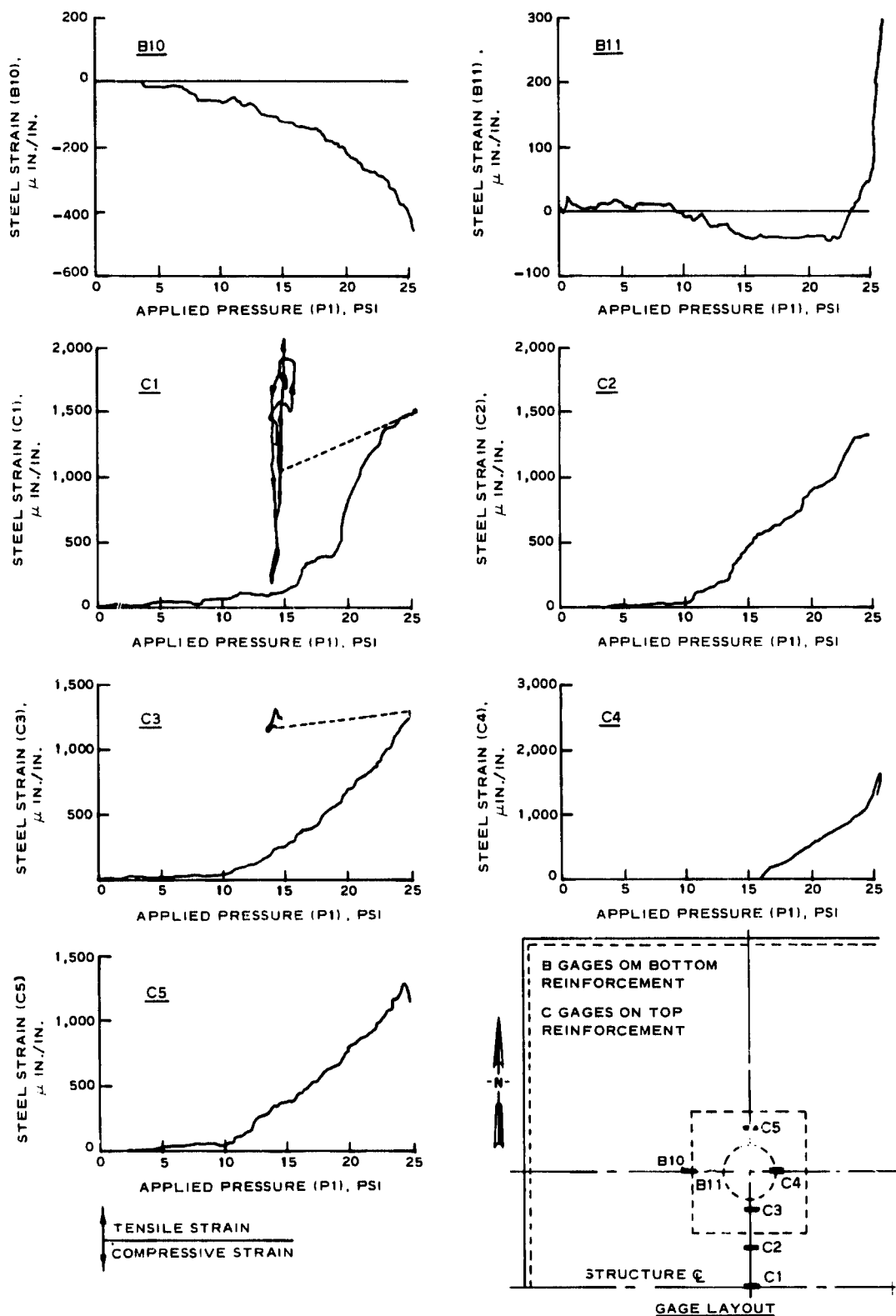


Figure A.14 Tensile reinforcement strains, Gages B10, B11, and C1 through C5, static test.

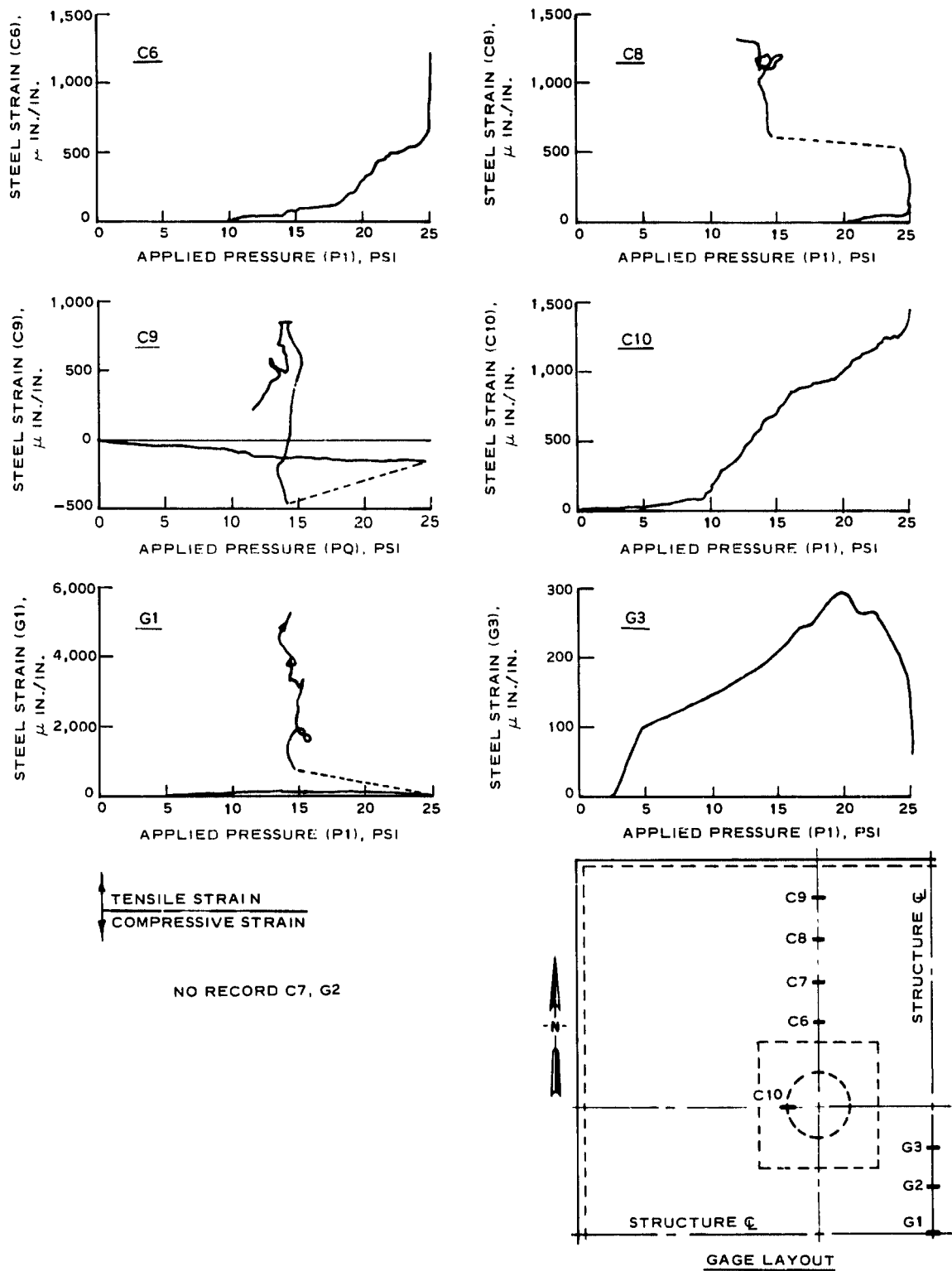


Figure A.15 Tensile reinforcement strains, Gages C6, C8 through C10, G1, and G3, static test.

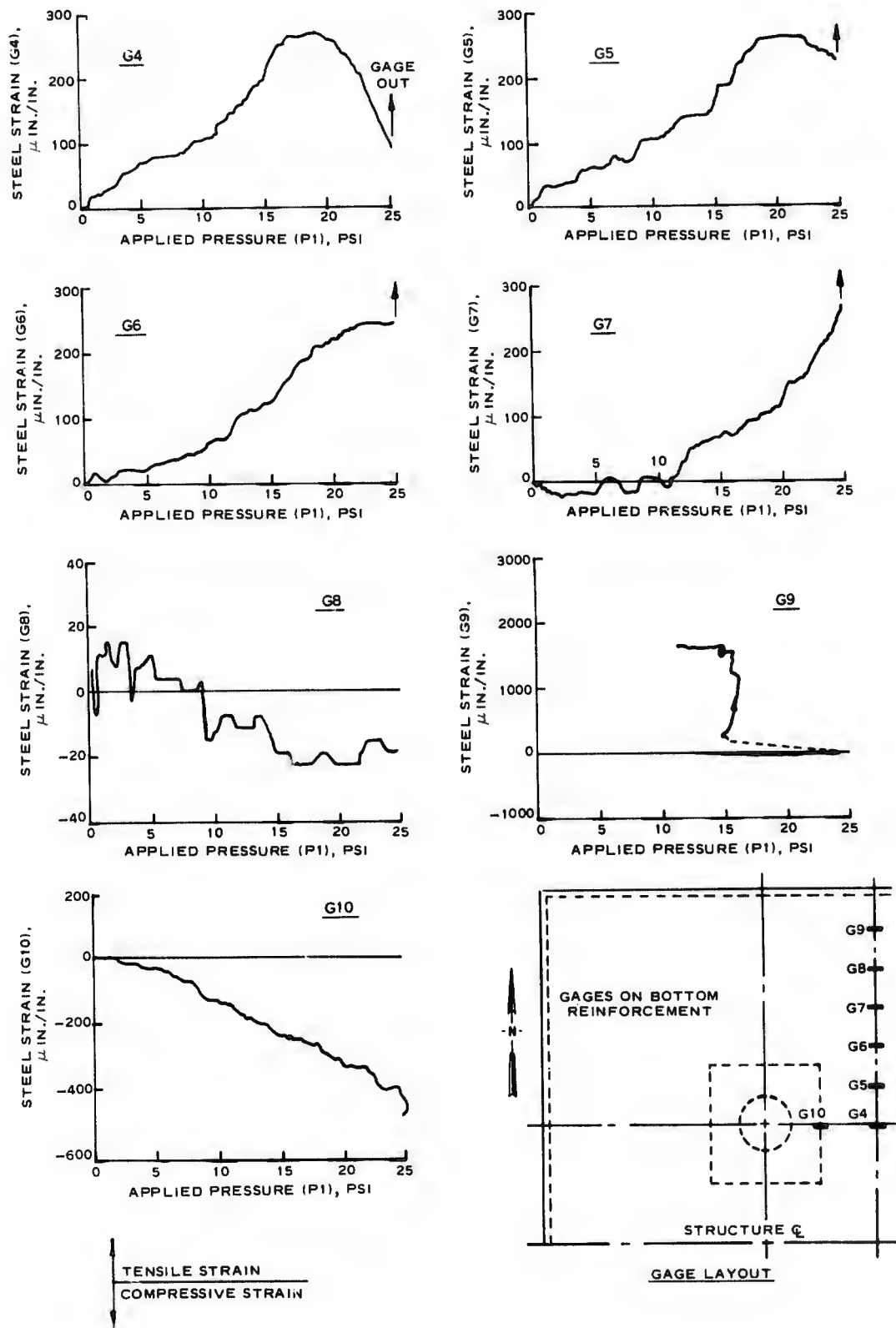


Figure A.16 Tensile reinforcement strains, Gages G4 through G10, static test.

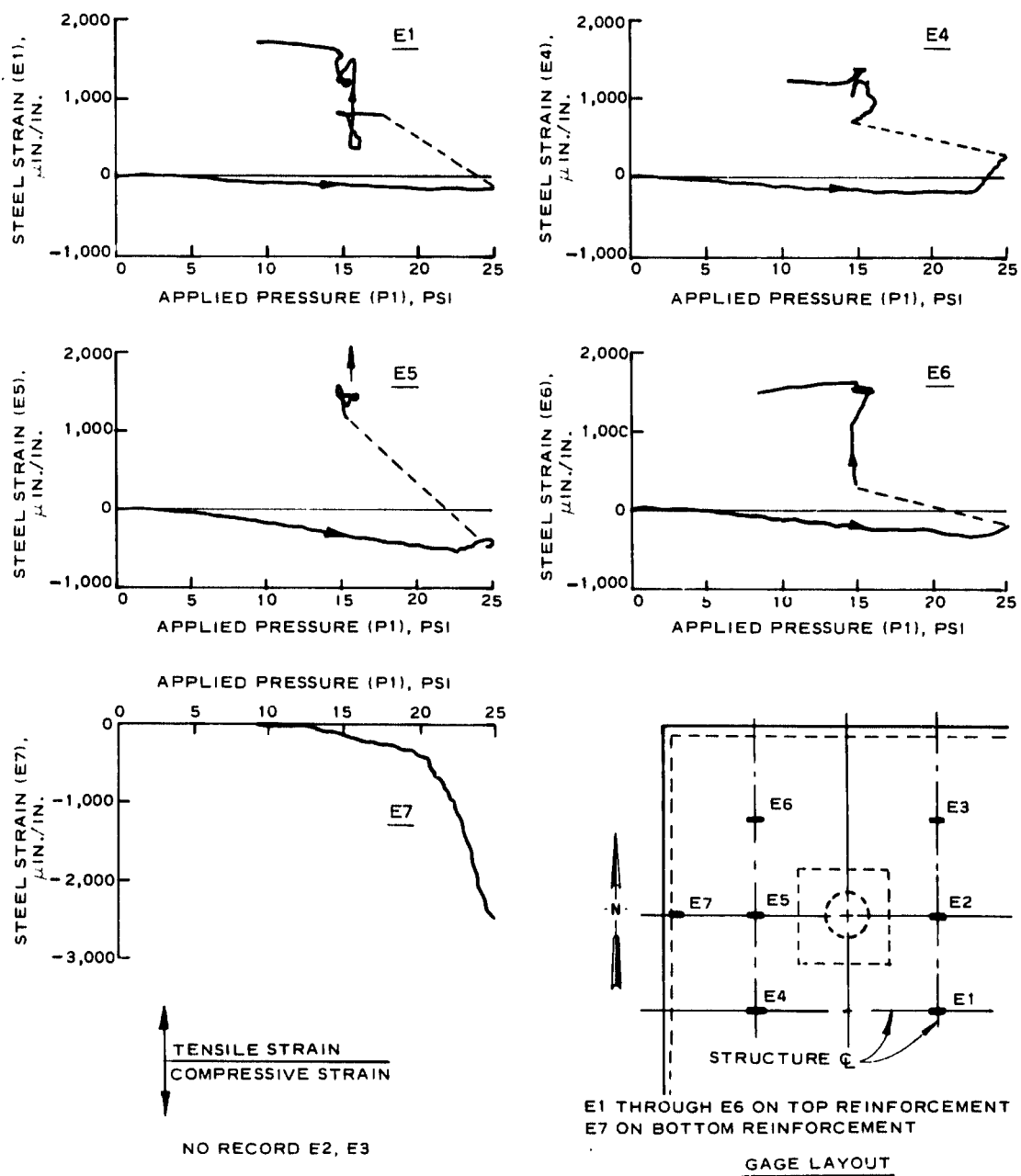
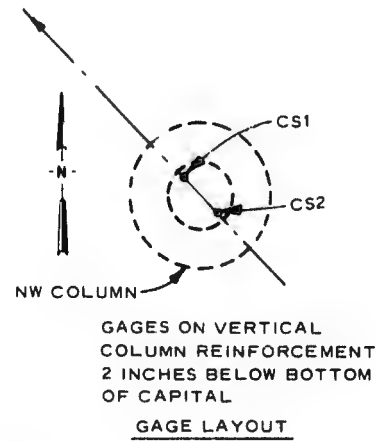
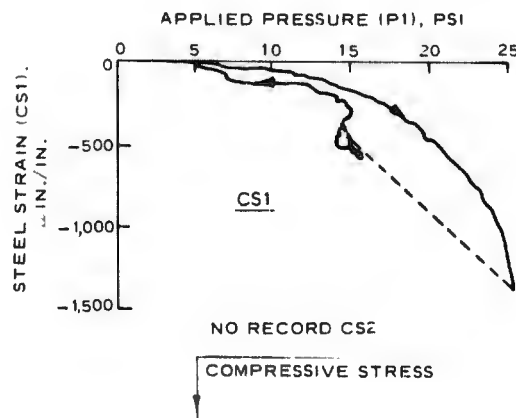
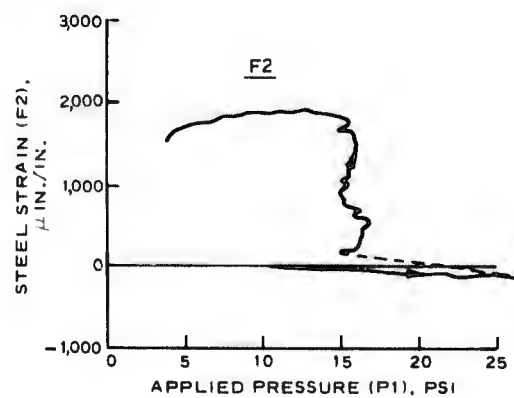
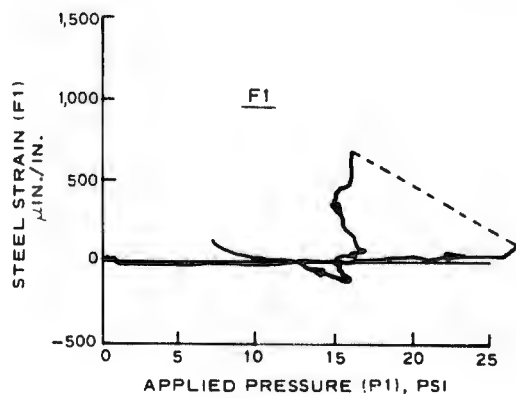


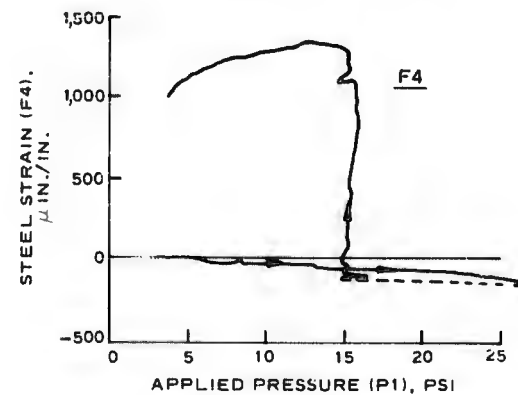
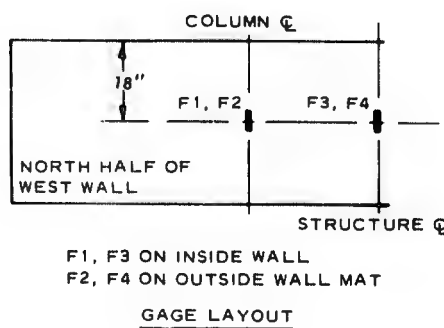
Figure A.17 Compressive reinforcement strains, static test.



a. COLUMN REINFORCEMENT STRAINS



NO RECORD F3



b. WALL REINFORCEMENT STRAINS

TENSILE STRAIN
↓
COMPRESSION STRAIN

Figure A.18 Wall and column steel strains, static test.

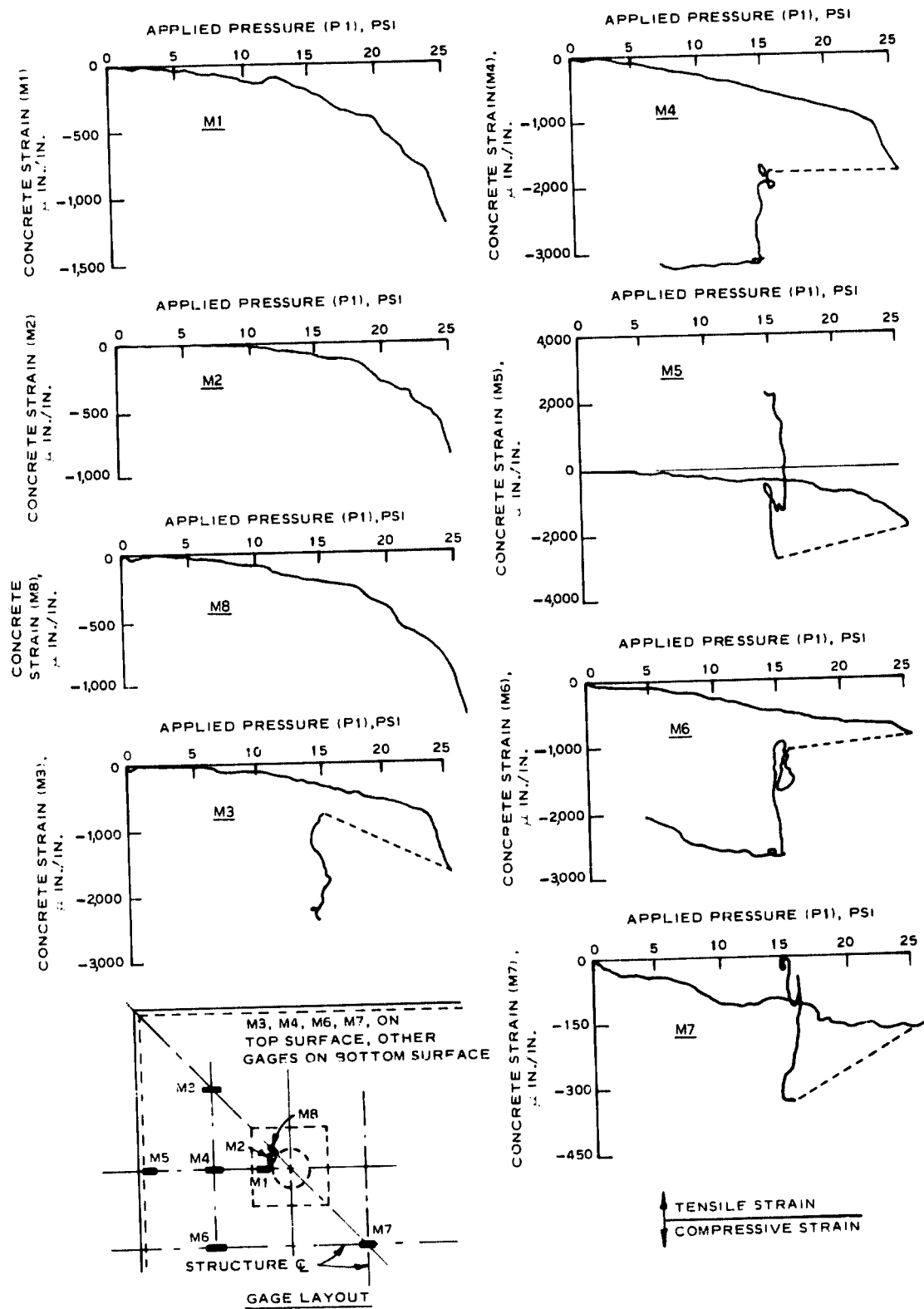
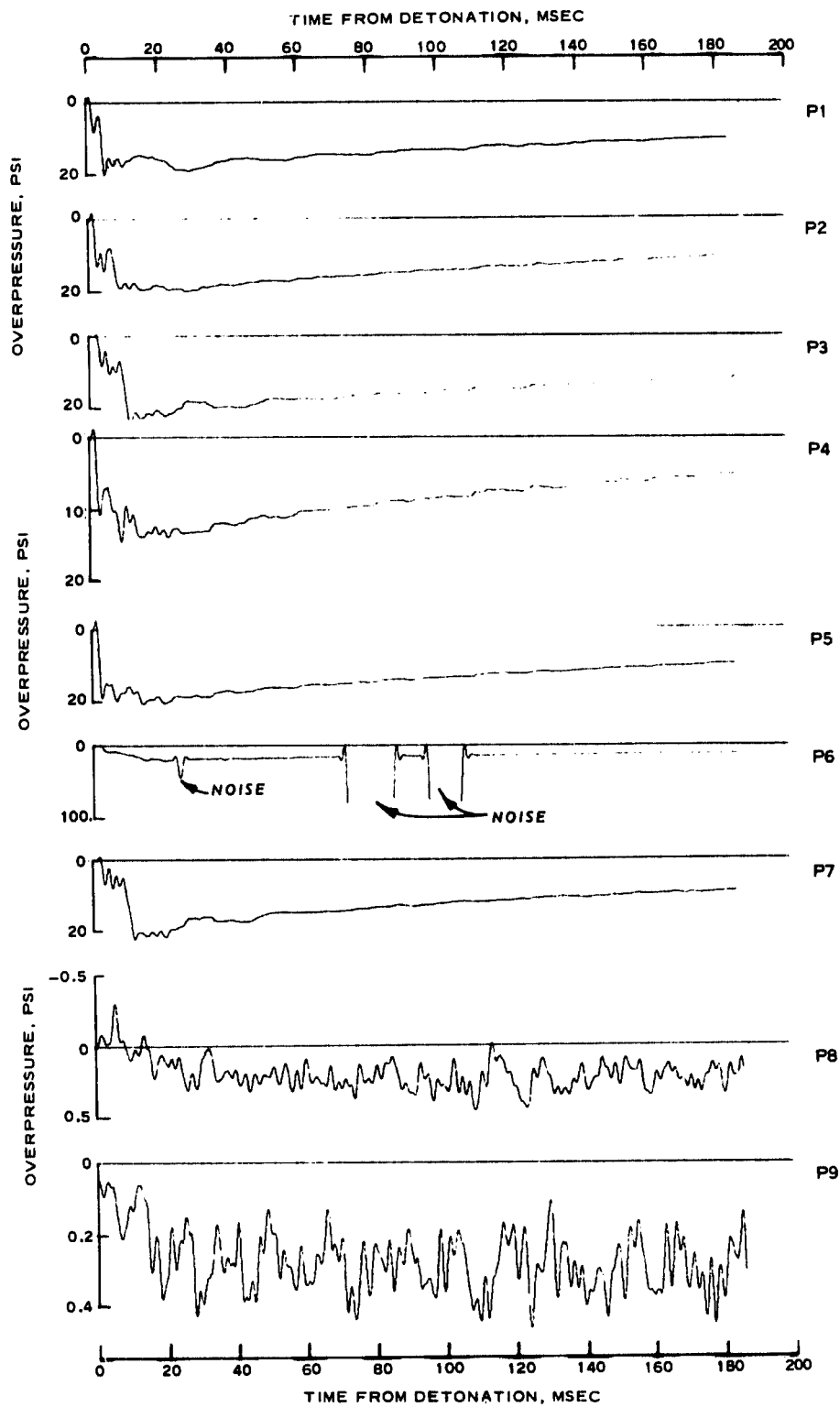


Figure A.19 Concrete strains, static test.

APPENDIX B
RECORDED DATA FOR DYNAMIC TESTS



NOTE: GAGES P1 THROUGH P7 LOCATED ON SLAB SURFACE (SEE FIG. 3.35)
GAGES P8 AND P9 LOCATED INSIDE STRUCTURE

Figure B.1 Overpressures, Dynamic Test I.

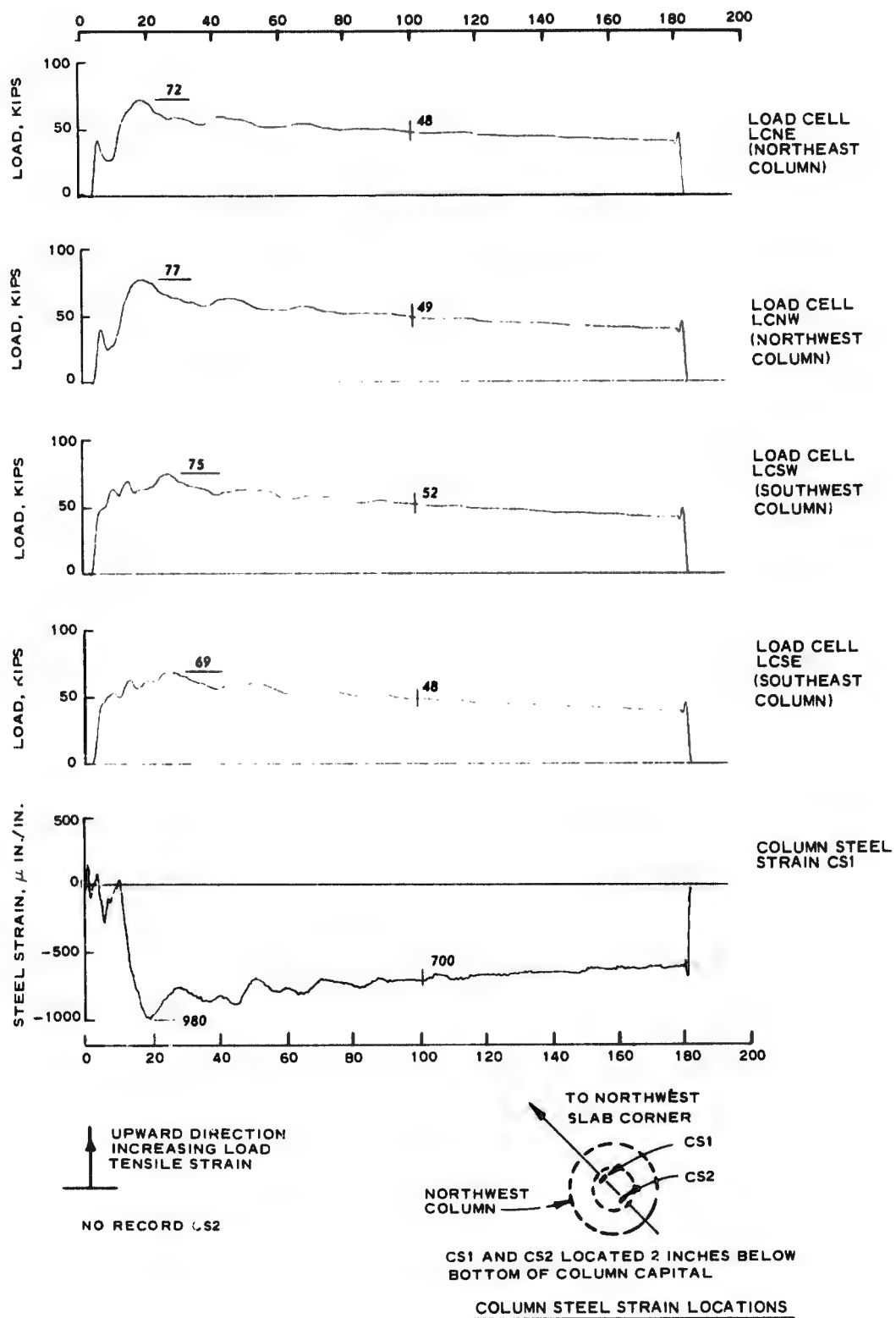
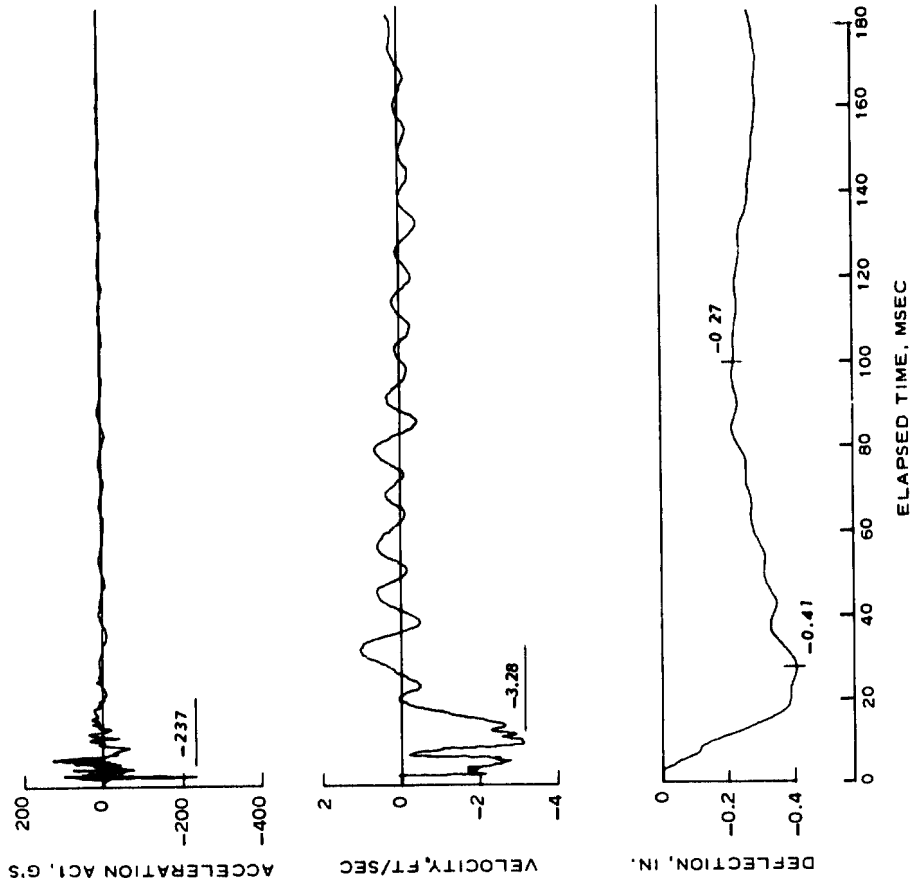
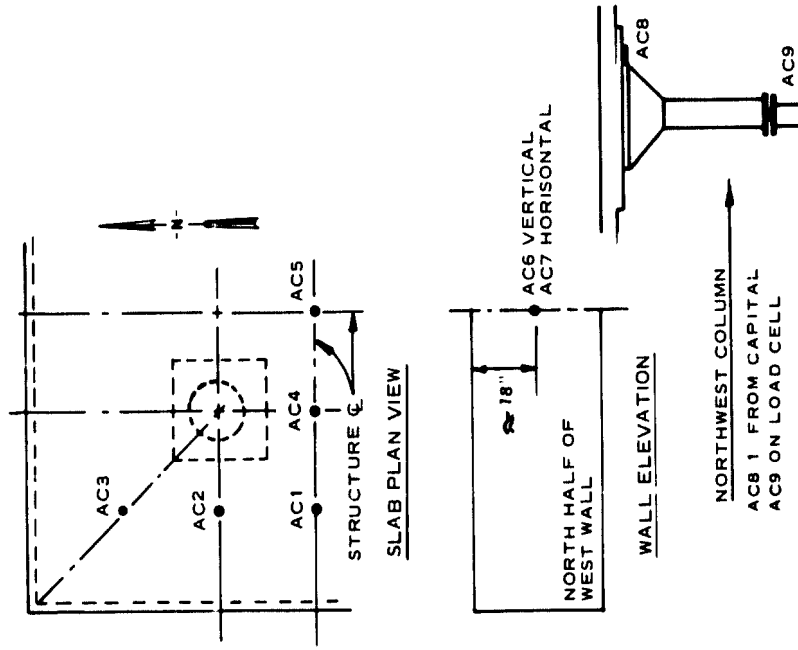


Figure B.2 Column loads and column reinforcement strains, Dynamic Test I.



a. AC1 - CENTER OF SIDE PANEL

NOTE: FOR EXACT TRANSDUCER LOCATIONS, SEE FIG. 3.35 AND 3.41



b. LOCATION OF ACCELEROMETERS

Figure B.3 Acceleration-, velocity-, and displacement-time curves, Gage AC1, Dynamic Test I.

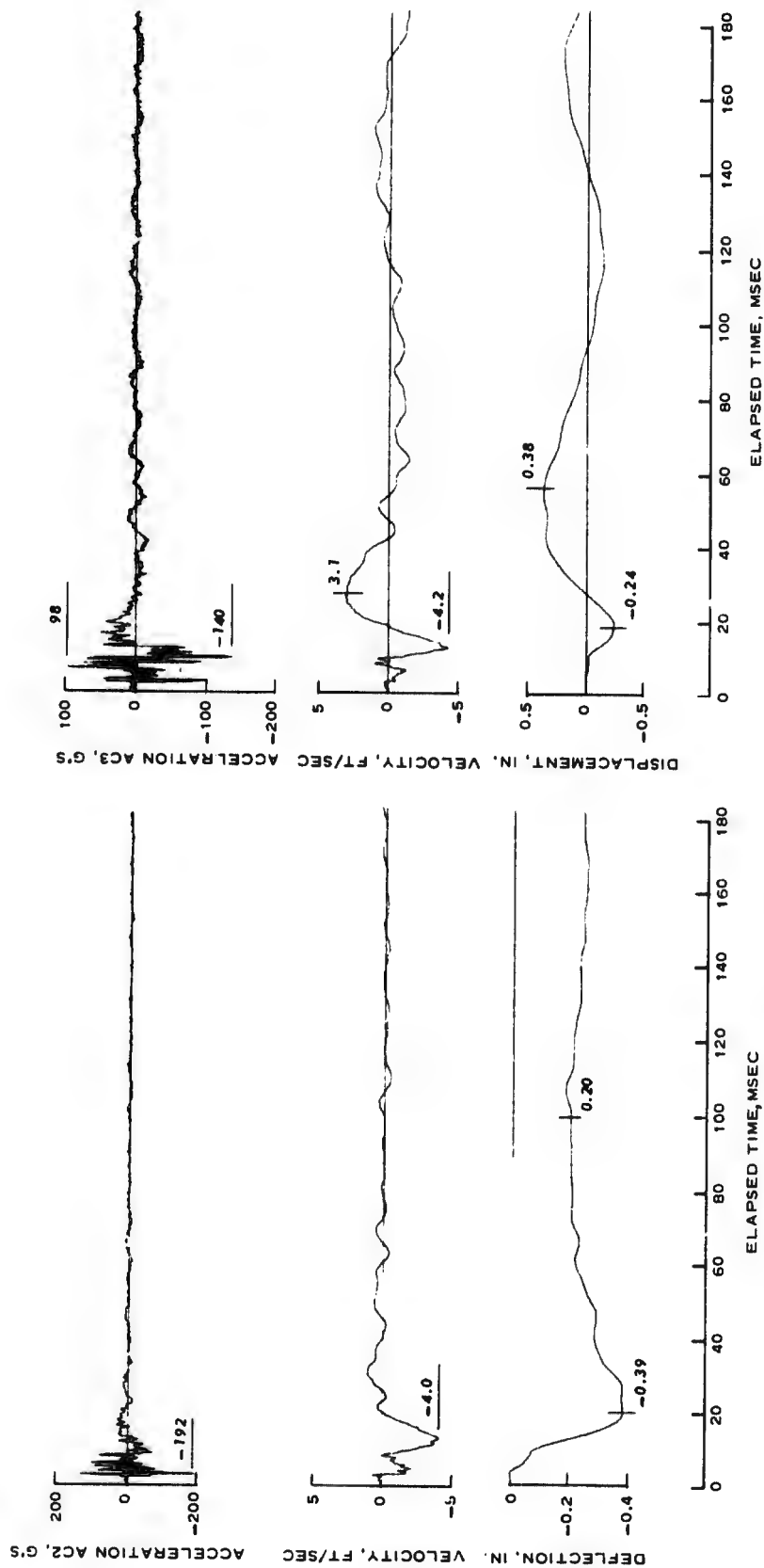
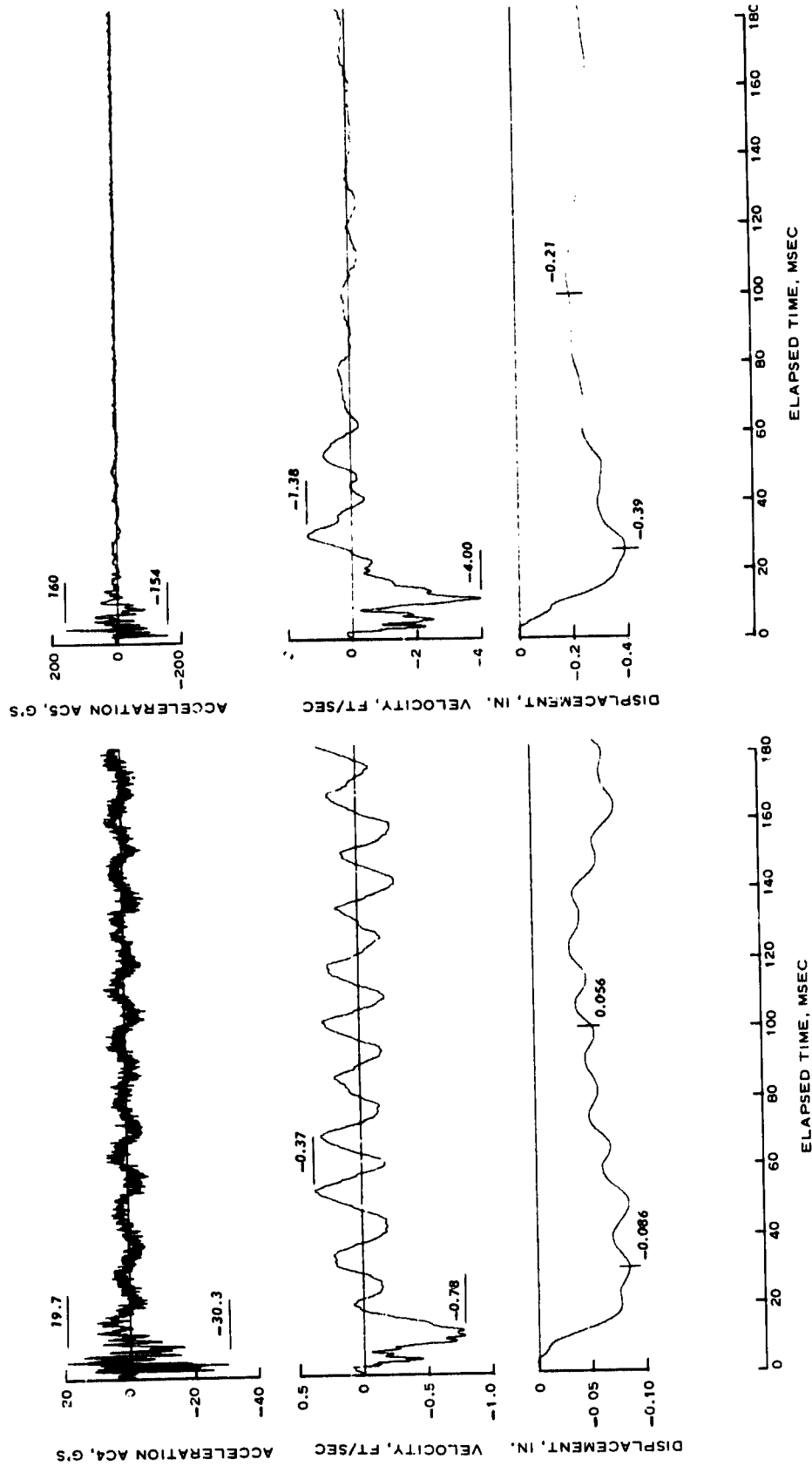


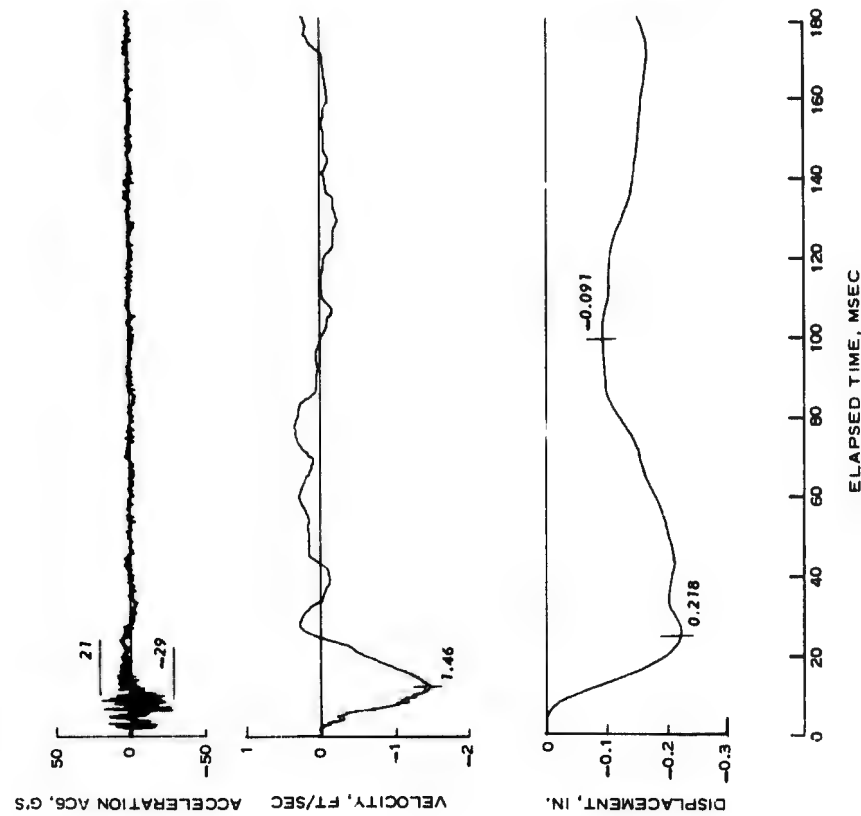
Figure B.4 Acceleration-, velocity-, and displacement-time curves, Gages AC2 and AC3, Dynamic Test: I.



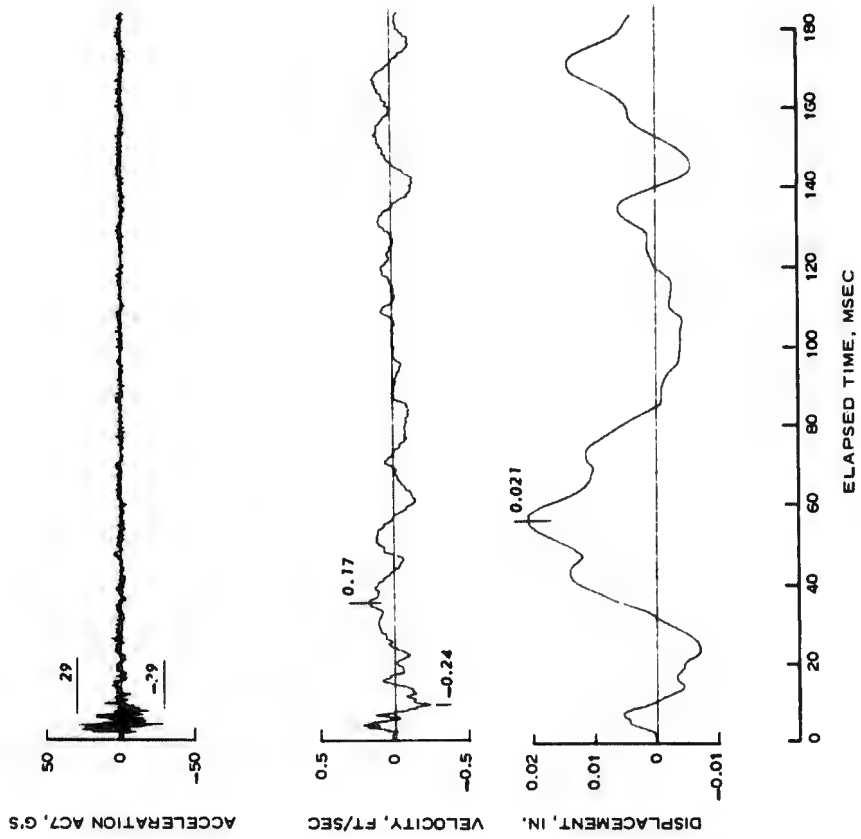
a. AC4 - CENTER OF INTERIOR COLUMN LINE SPAN

b. AC5 - CENTER OF INTERIOR SPAN

Figure B.5 Acceleration-, velocity-, and displacement-time curves, Gages AC4 and AC5, Dynamic Test I.



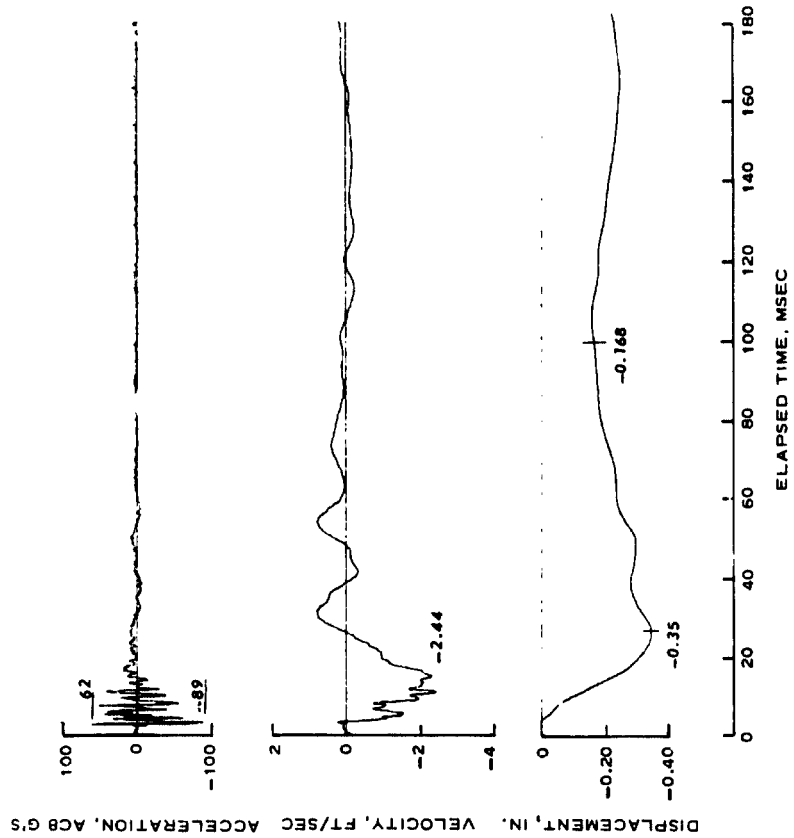
a. AC6 - VERTICAL MOTION OF CENTER, WEST WALL



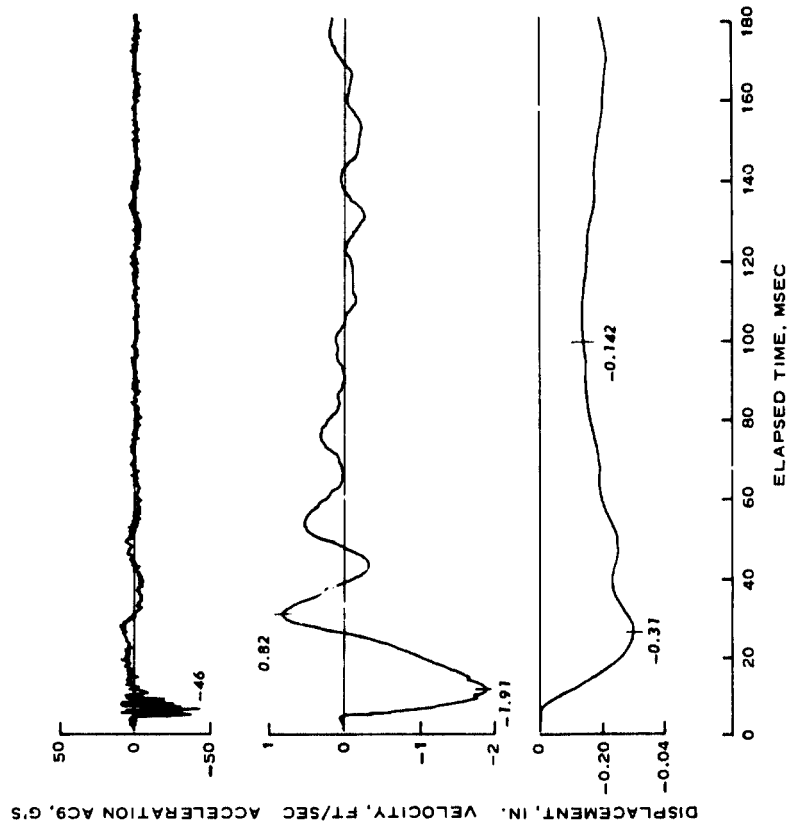
NOTE: NEGATIVE DIRECTION MEASURED INTO STRUCTURE

b. AC7 - HORIZONTAL MOTION OF CENTER, WEST WALL

Figure B.6 Acceleration-, velocity-, and displacement-time curves, Gages AC6 and AC7, Dynamic Test I.



a. AC8 - DROP PANEL OF NORTHWEST COLUMN



b. AC9 - BASE OF NORTHWEST COLUMN

Figure B.7 Acceleration-, velocity-, and displacement-time curves, Gages AC8 and AC9, Dynamic Test I.

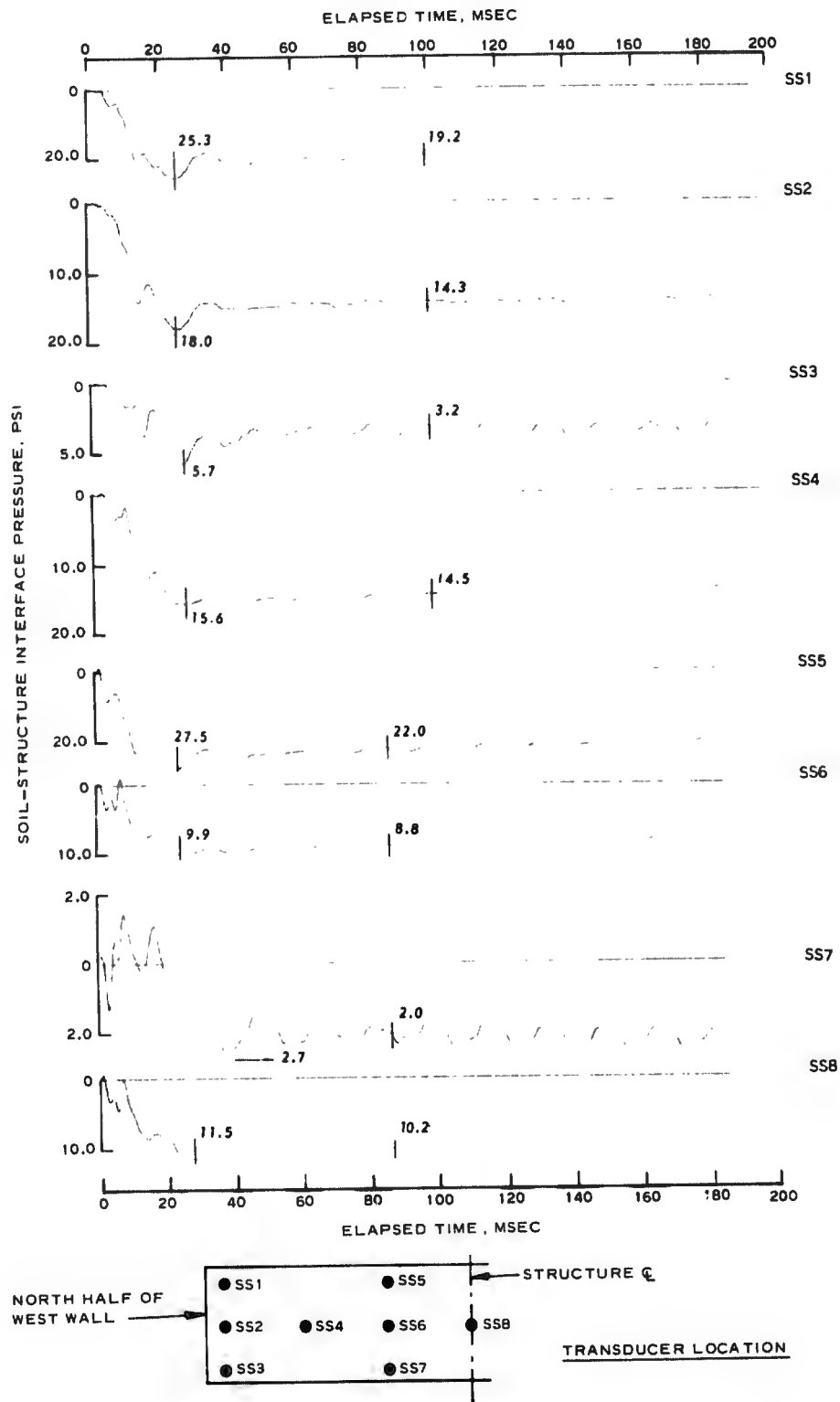


Figure B.8 Soil-structure interface pressures, Dynamic Test I.

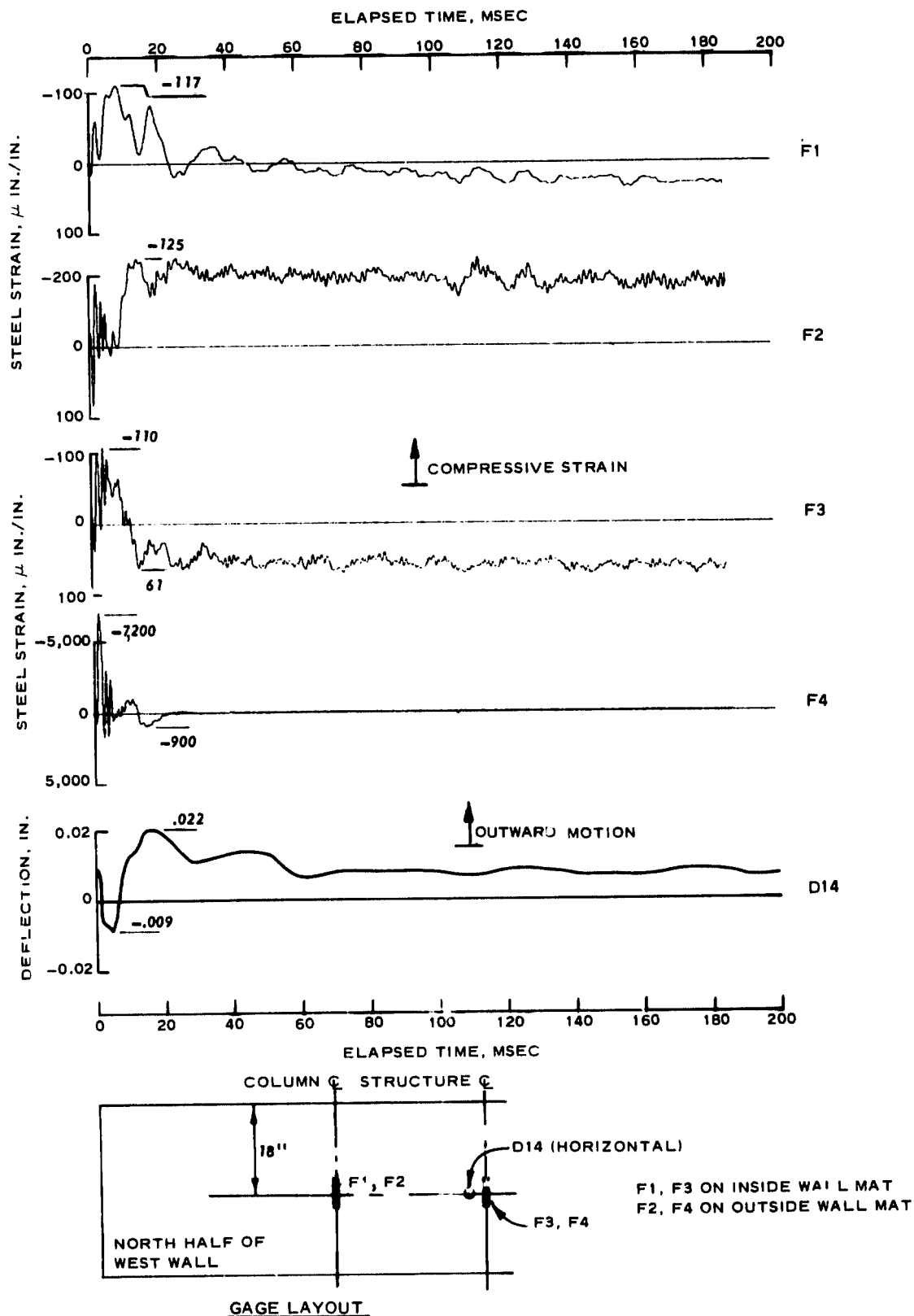


Figure B.9 Wall steel strains and deflections, Dynamic Test I.

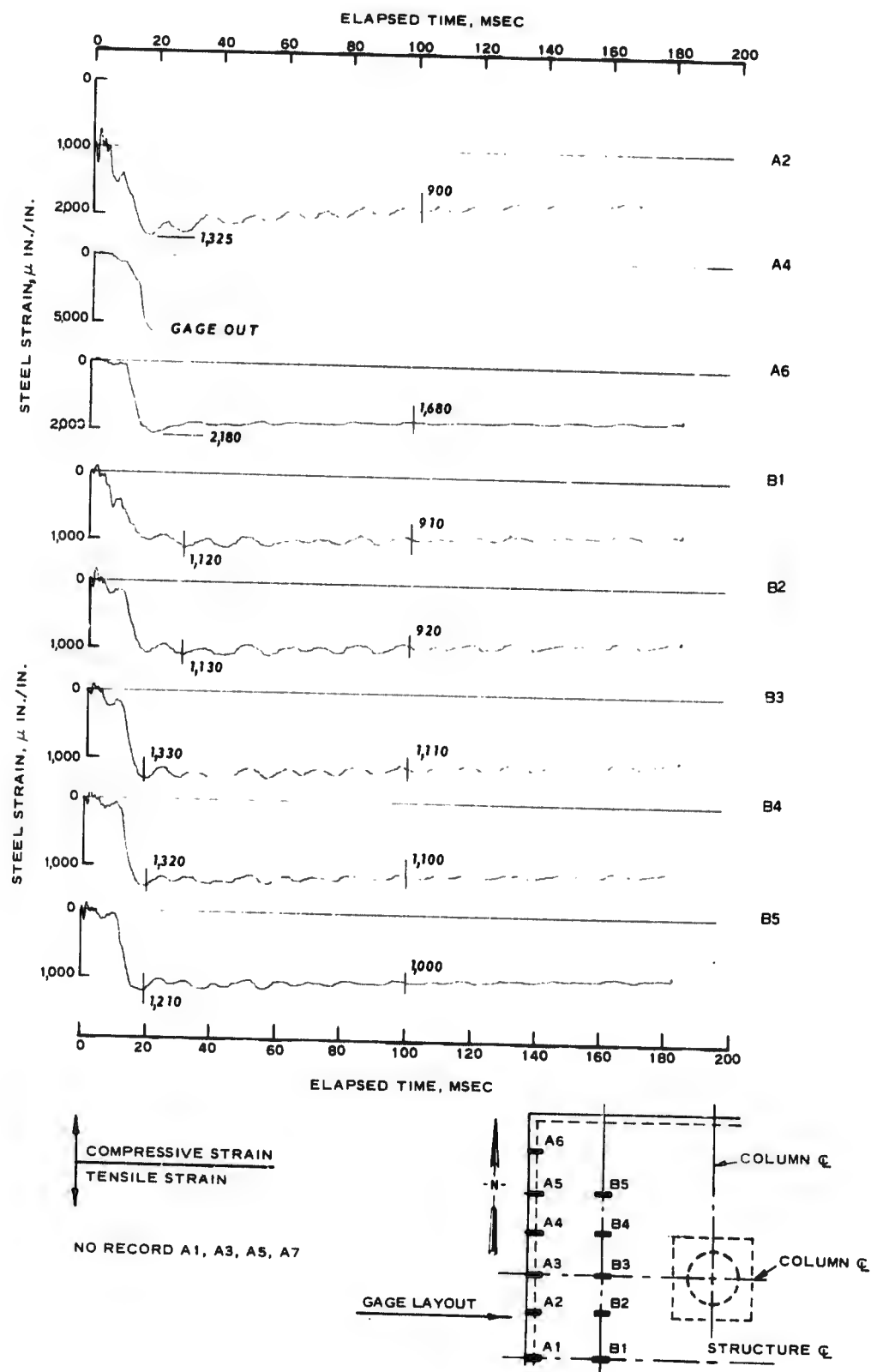


Figure B.10 Tensile reinforcement strains, Gages A2, A4, A6, and B1 through B5, Dynamic Test I.

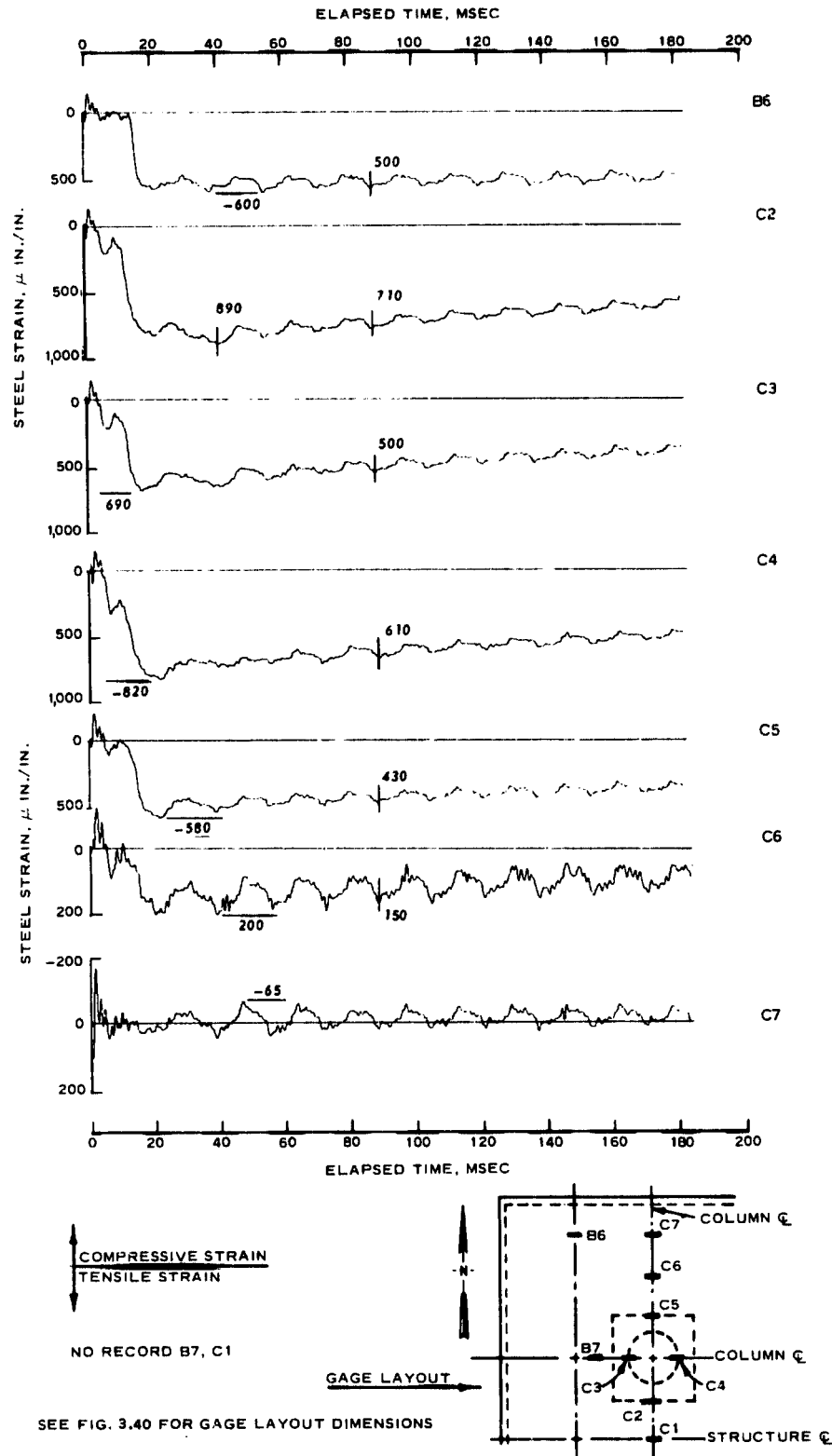


Figure B.11 Tensile reinforcement strains, Gages B6 and C2 through C7, Dynamic Test I.

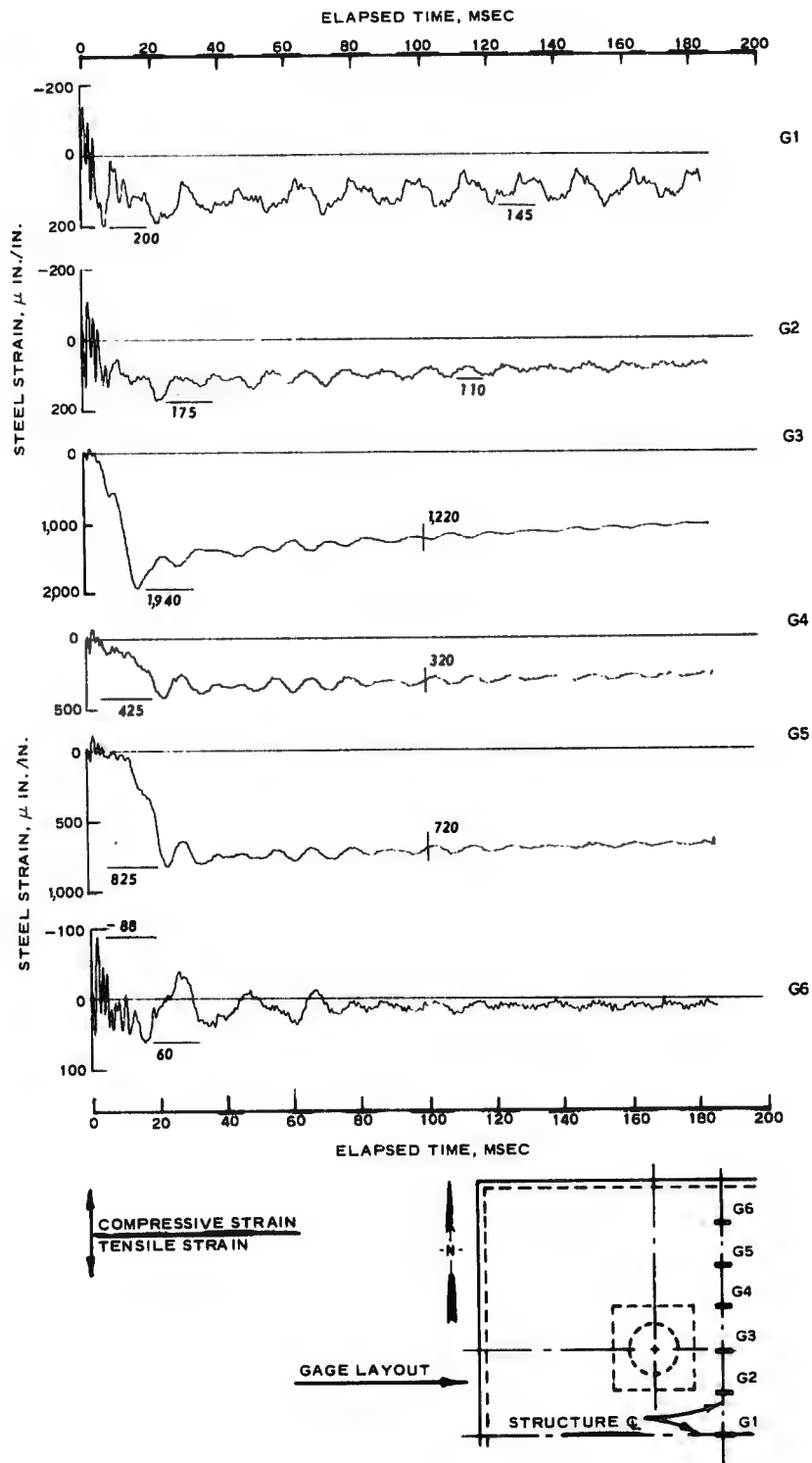


Figure B.12 Tensile reinforcement strains, Gages G1 through G6, Dynamic Test I.

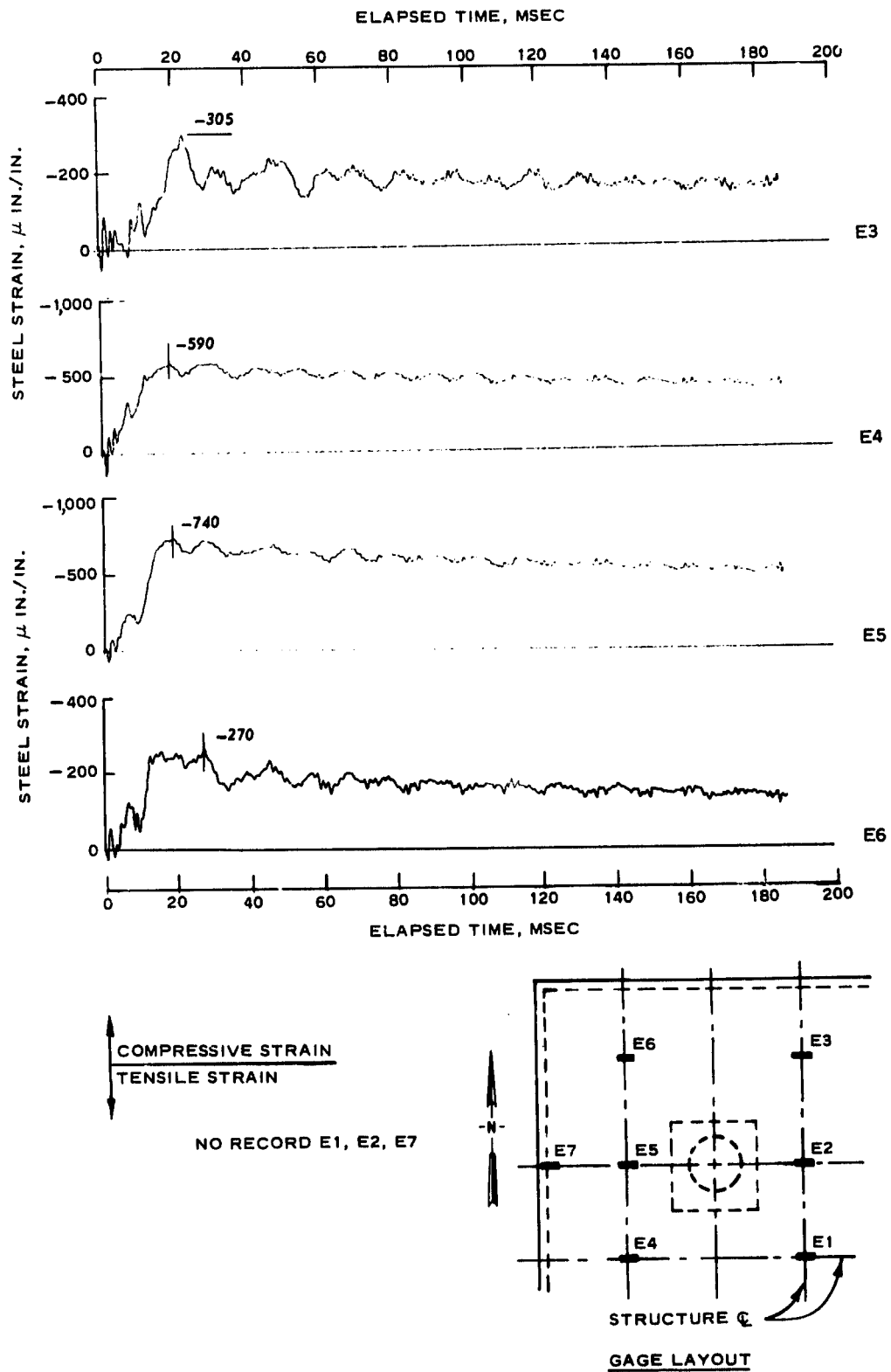


Figure B.13 Compressive reinforcement strains, Dynamic Test I.

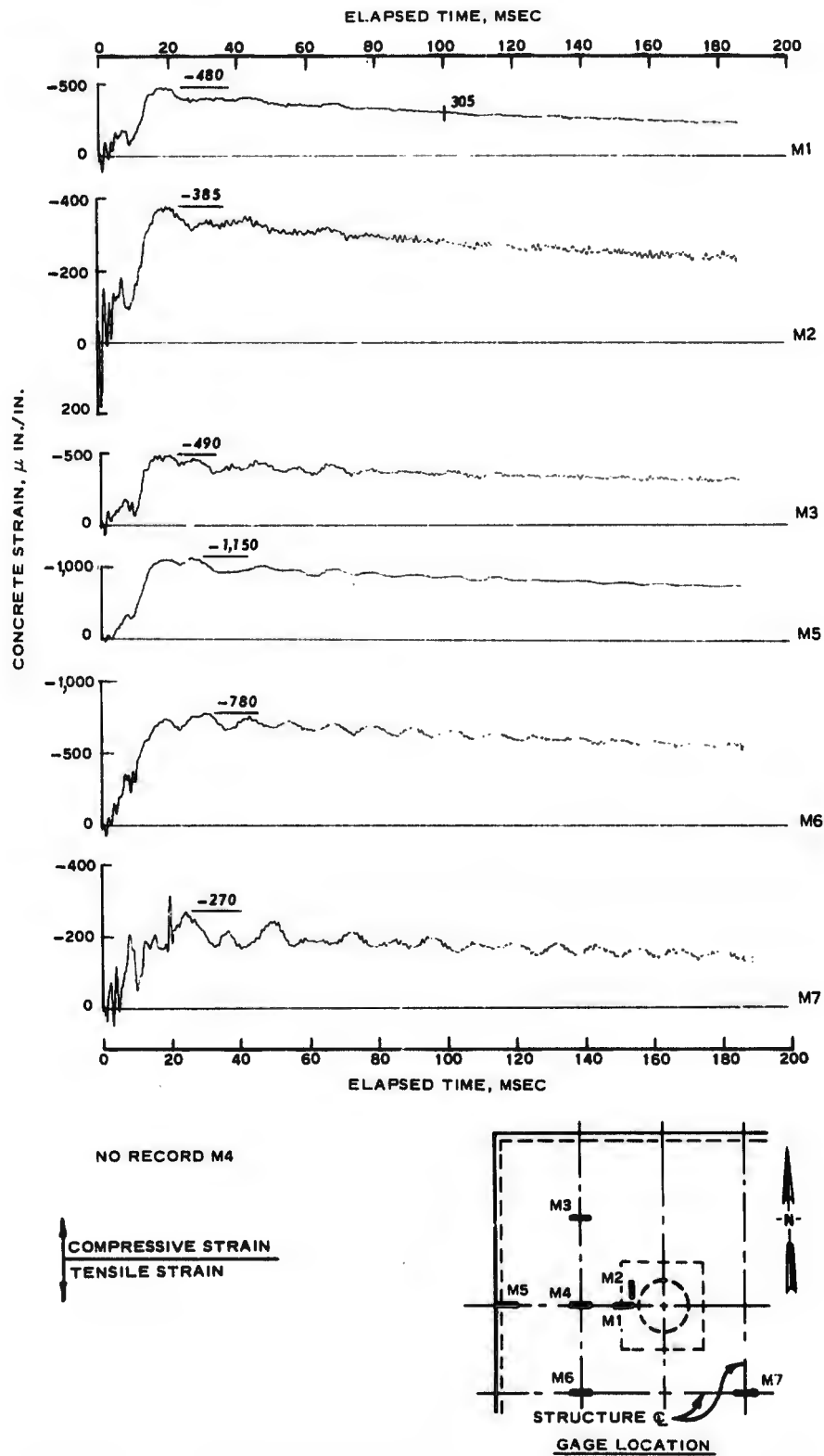
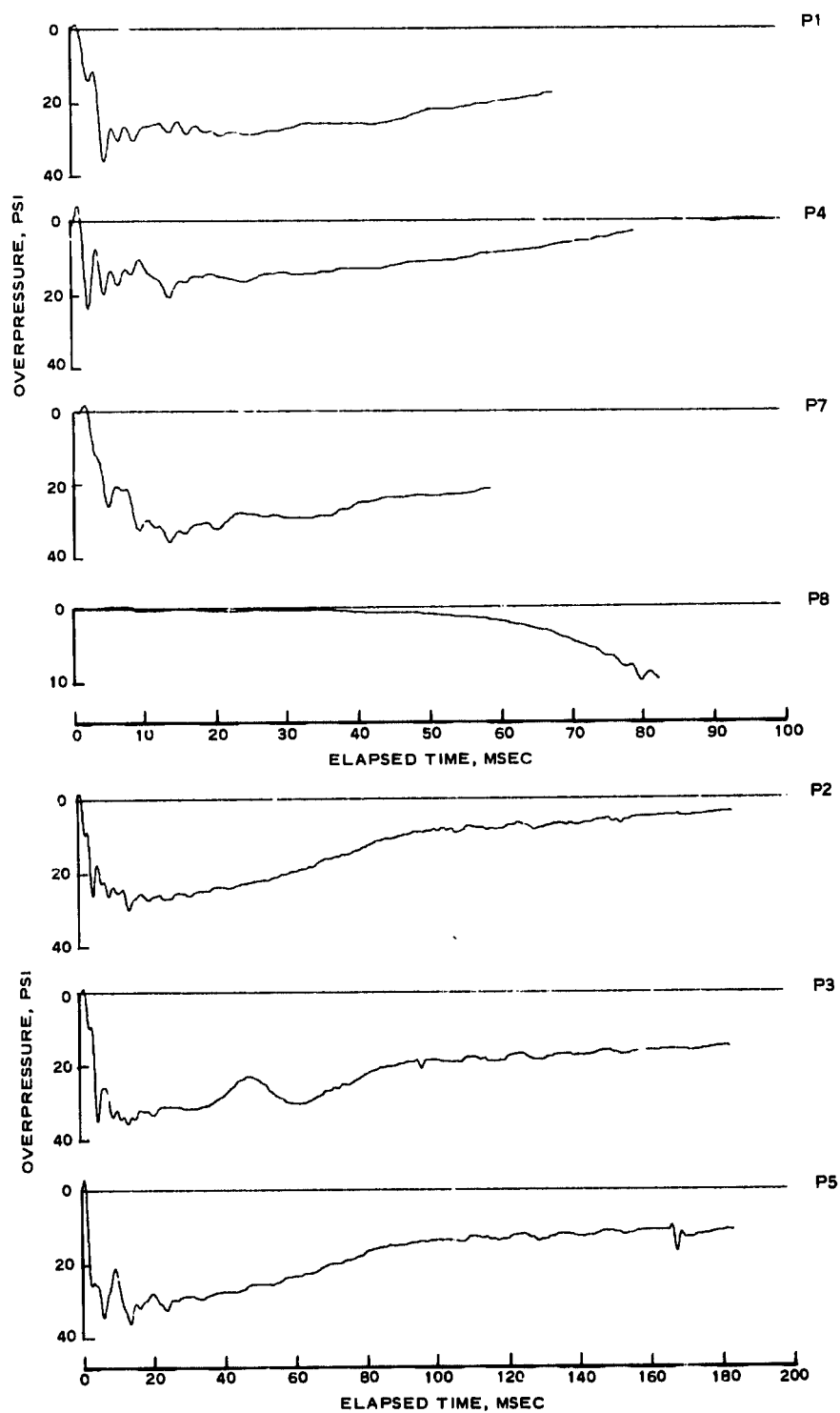


Figure B.14 Concrete compressive strains, Dynamic Test I.



NOTE: GAGE P8 LOCATED INSIDE STRUCTURE, REMAINDER OF PRESSURE GAGES ON SLAB SURFACE. SEE FIG. 3.35 FOR GAGE LAYOUT

Figure B.15 Overpressures, Dynamic Test II.

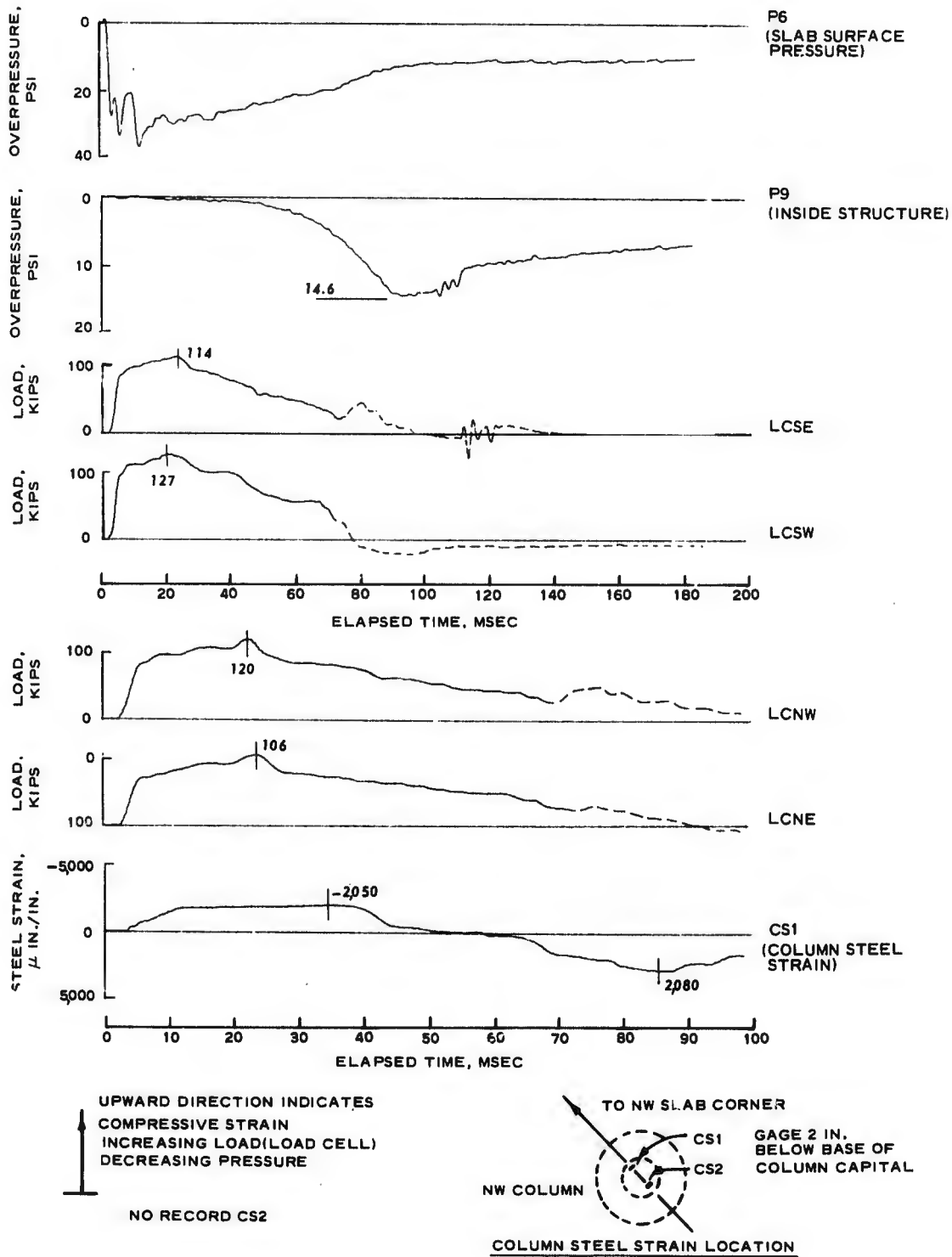


Figure B.16 Pressures, column loads, and column steel strains, Dynamic Test II.

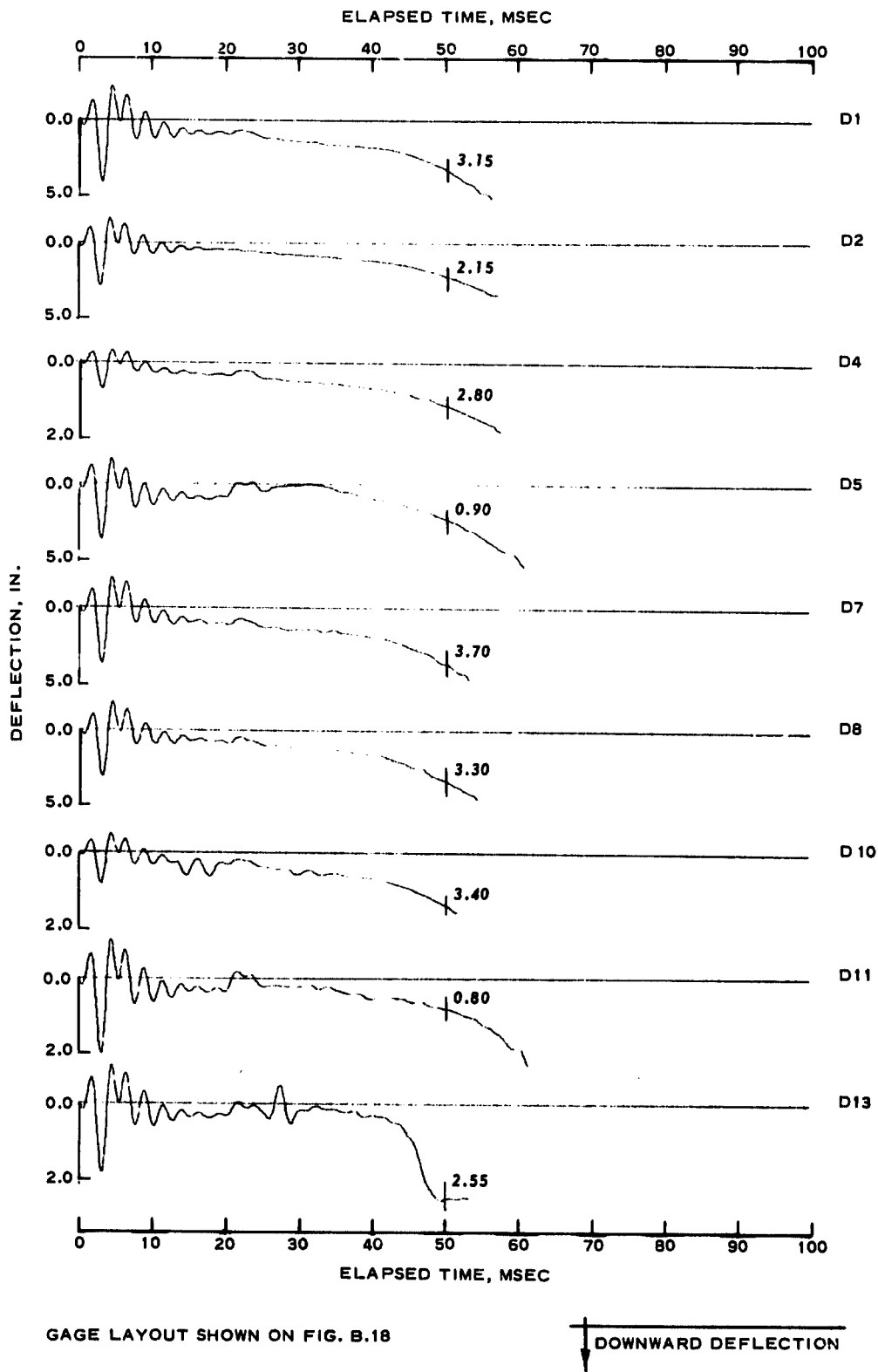
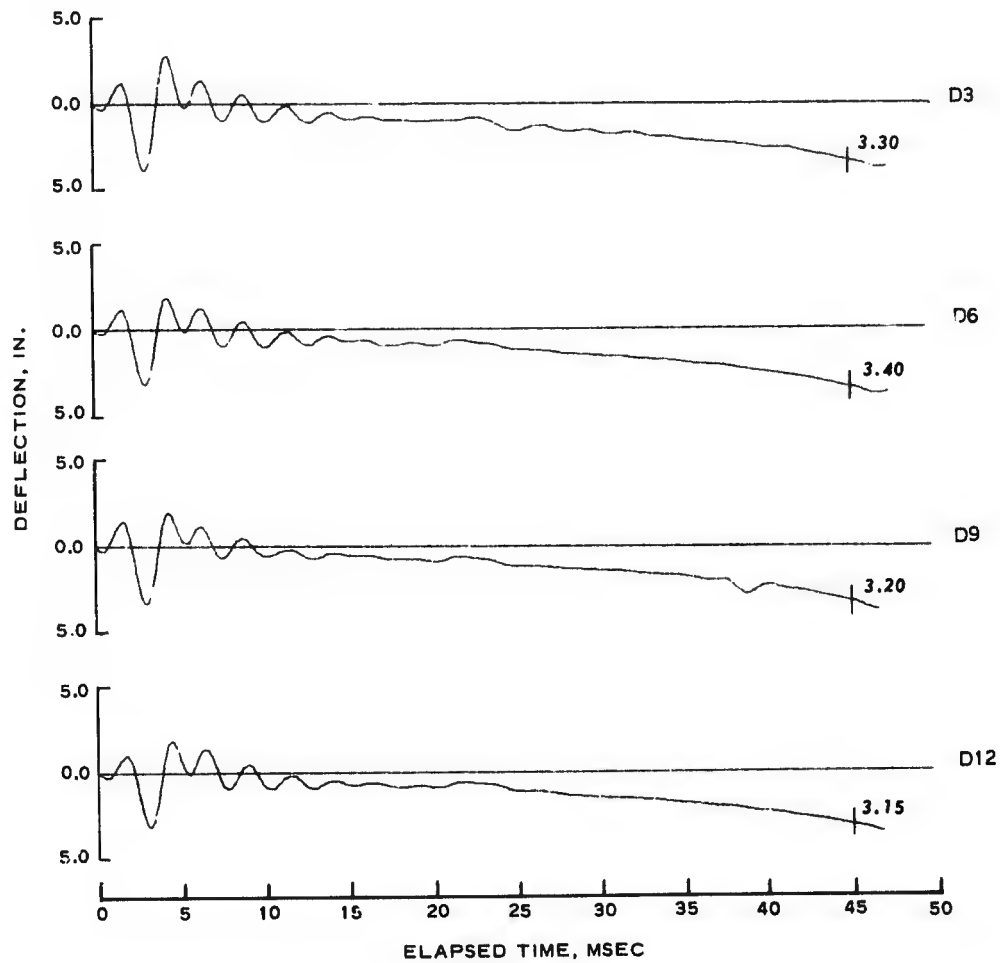


Figure B.17 Slab deflections, Gages D1, D2, D4, D5, D7, D8, D10, D11, and D13, Dynamic Test II.



NOTE: OSCILLATIONS IN FIRST 15 MSEC OF RECORDS ARE FROM ELECTRICAL NOISE

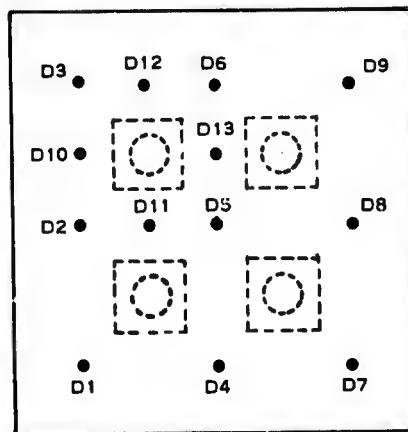


Figure B.18 Slab deflections, Gages D3, D6, D9, and D12, Dynamic Test II.

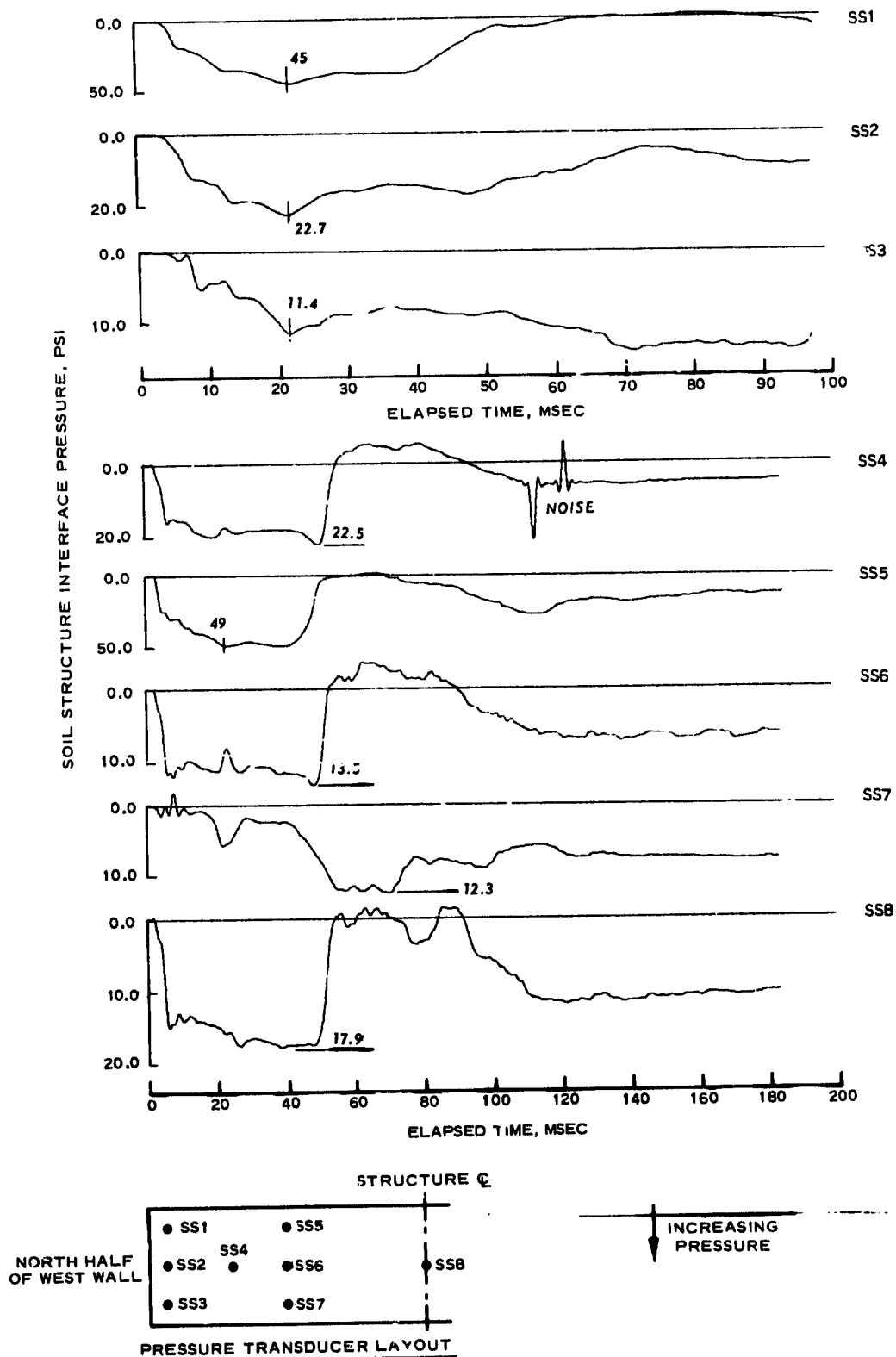


Figure B.19 Soil-structure interface pressures, Dynamic Test II.

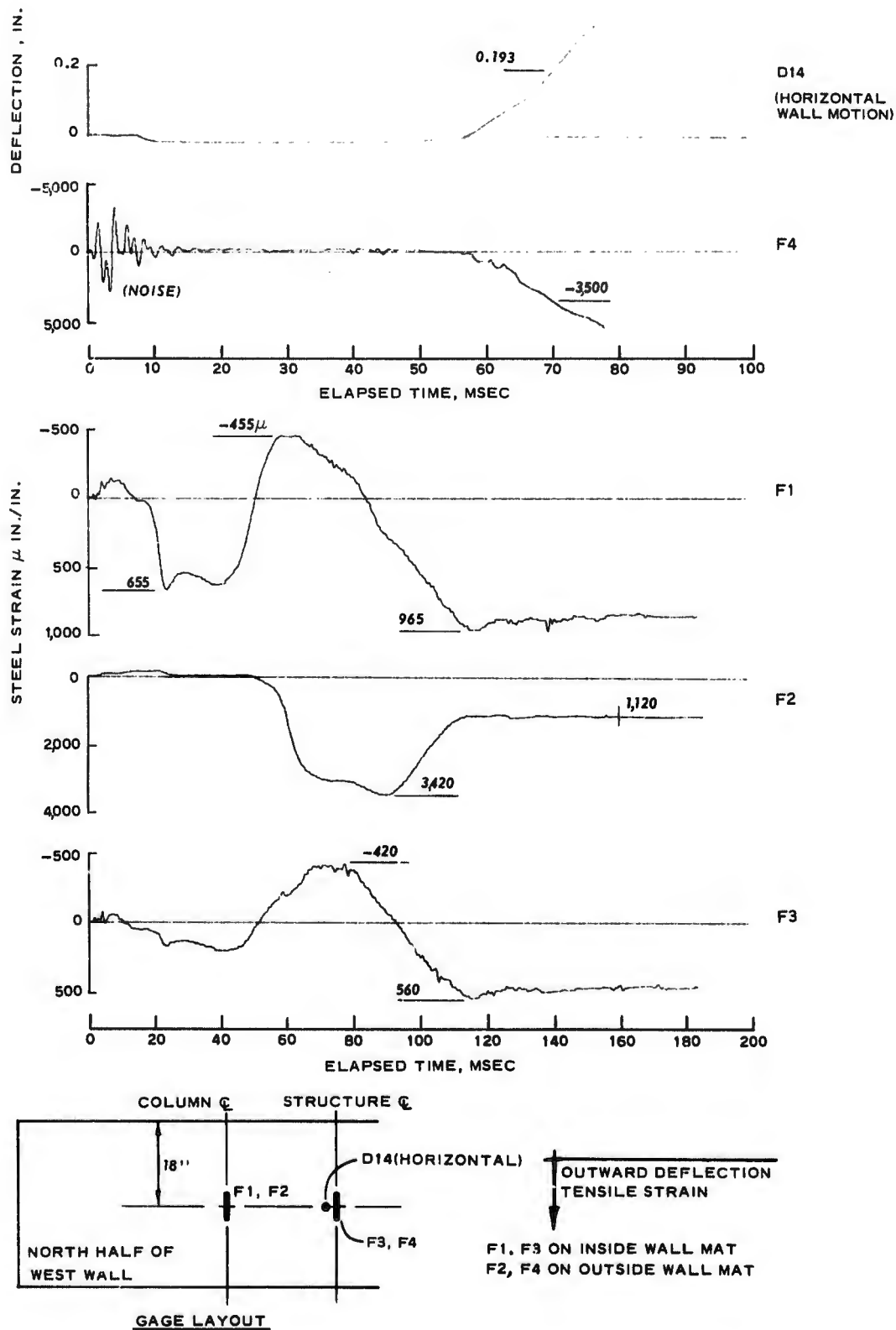


Figure B.20 Wall steel strains and deflections, Dynamic Test II.

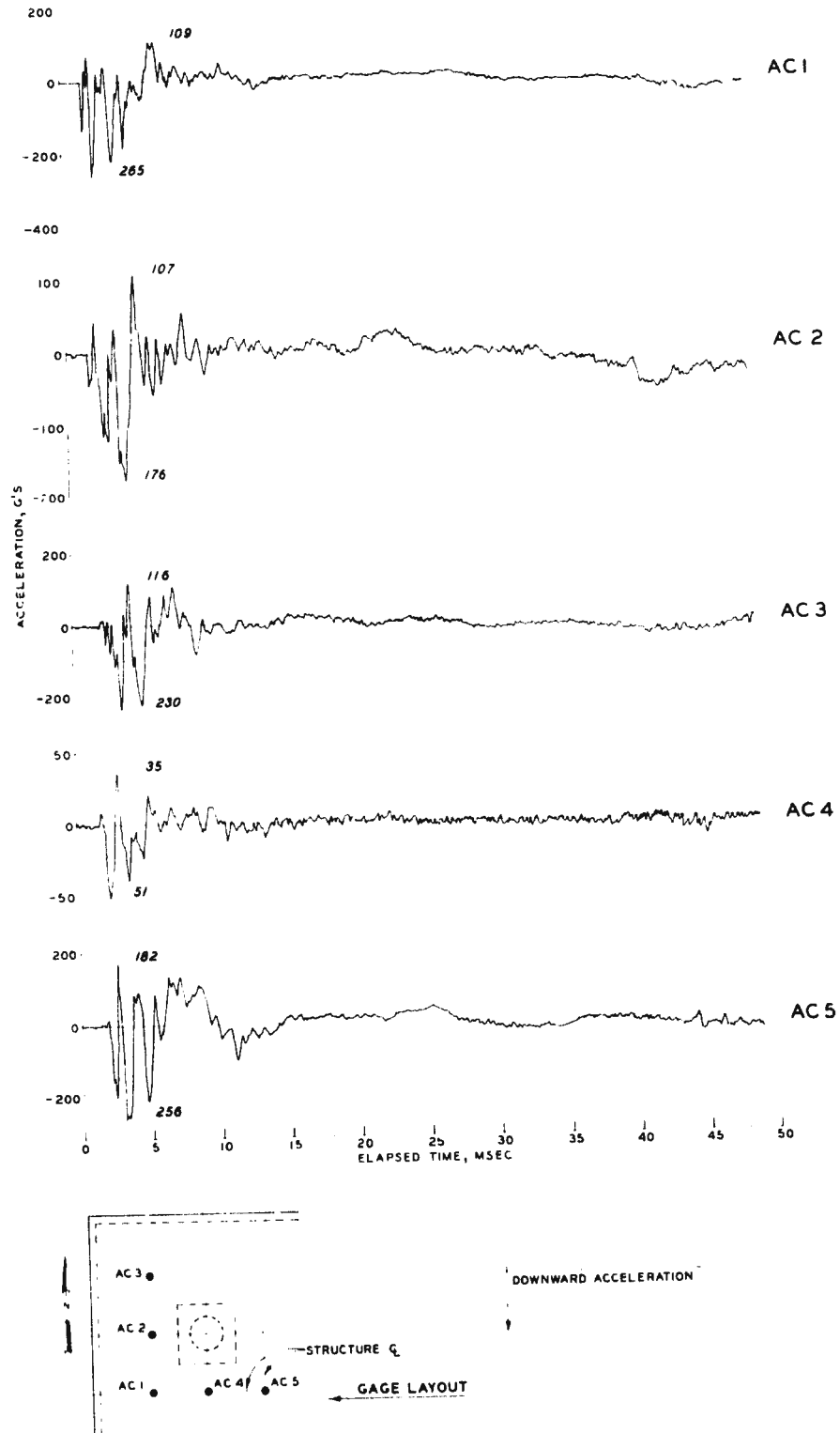


Figure B.21 Acceleration of slab at midspan points, Dynamic Test II.

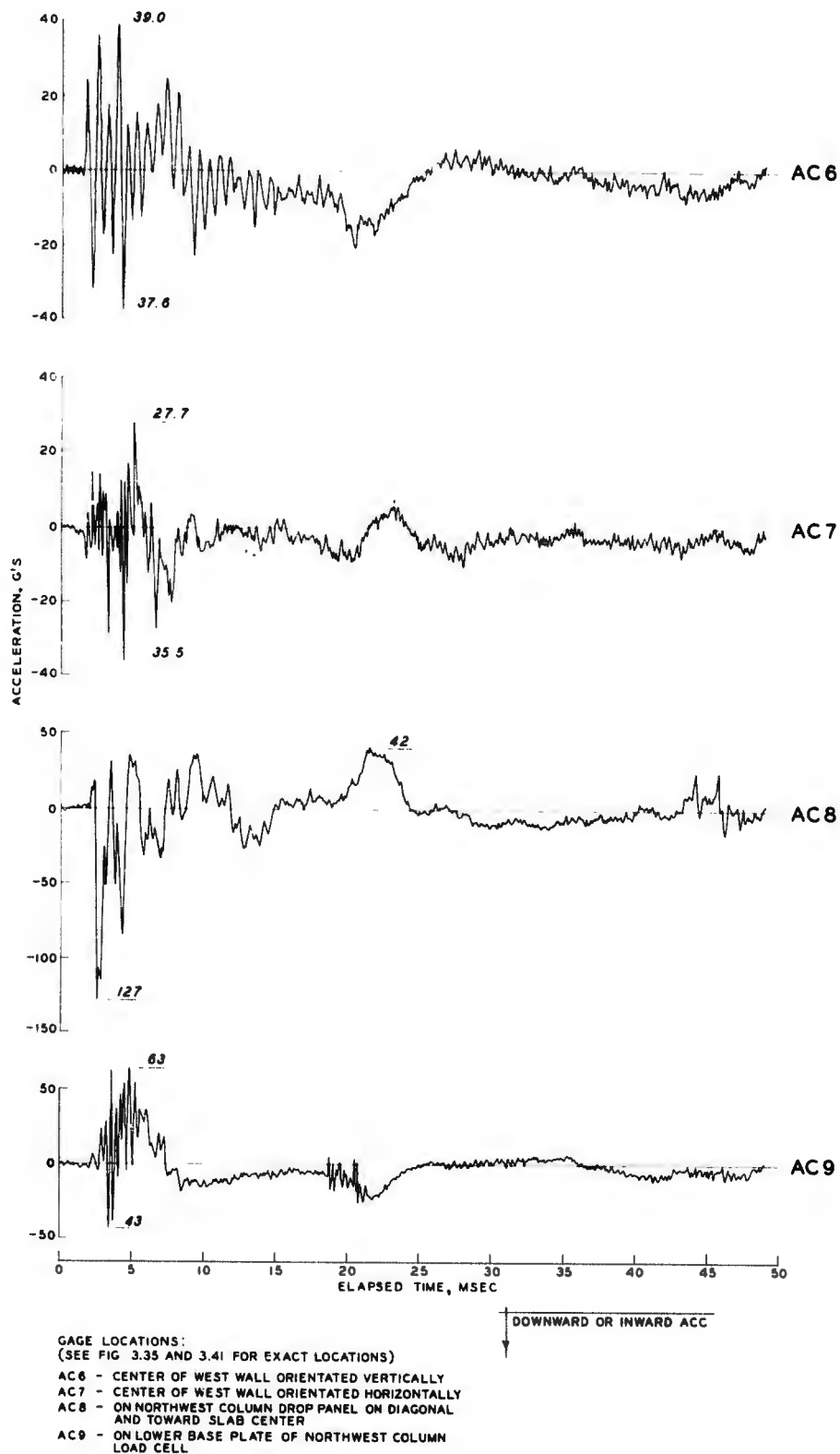


Figure B.22 Accelerations of wall and column vicinities, Dynamic Test II.

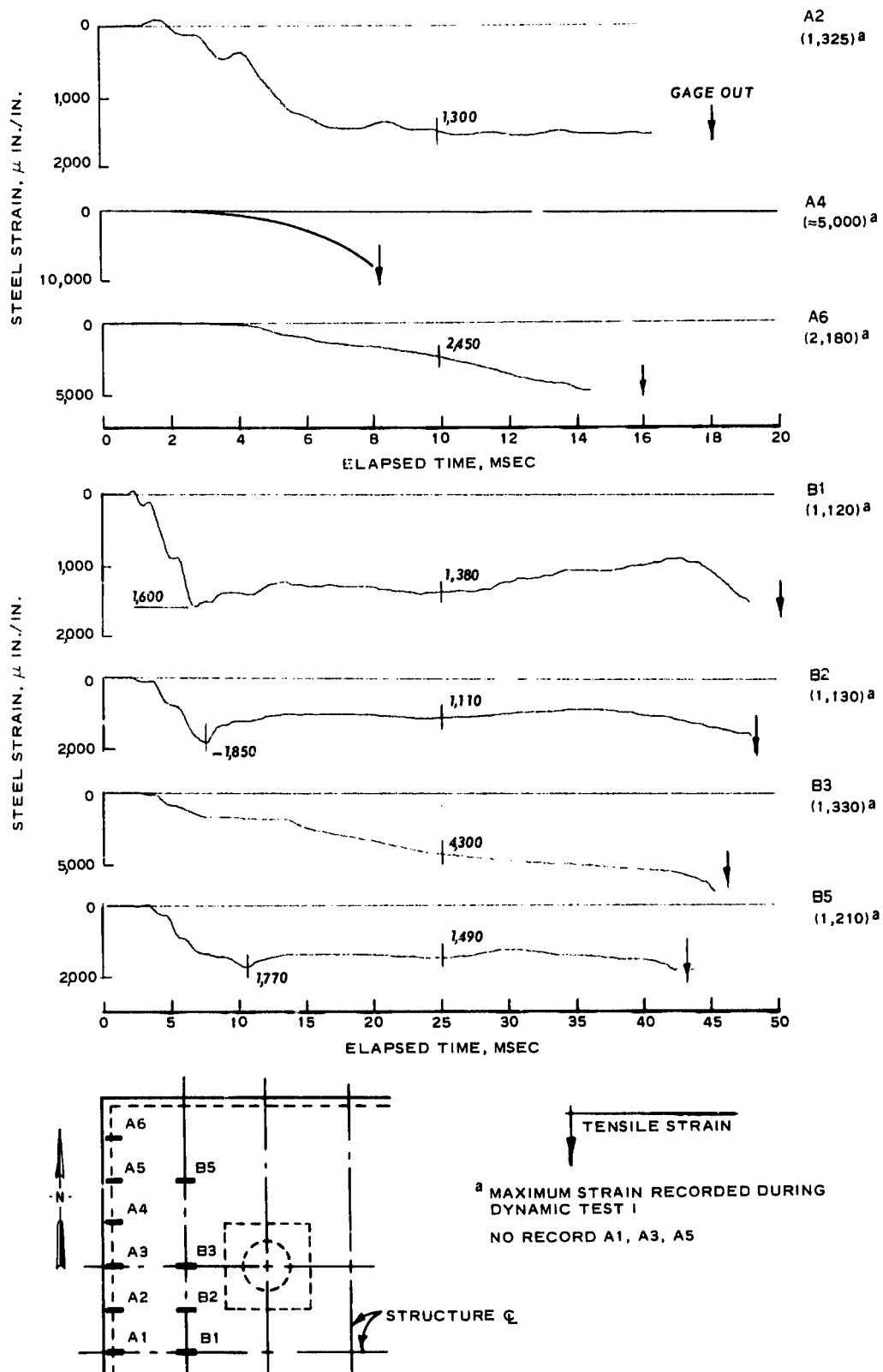


Figure B.23 Tensile reinforcement strains, Gages A2, A4, A6, B1, B2, B3, and B5, Dynamic Test II.

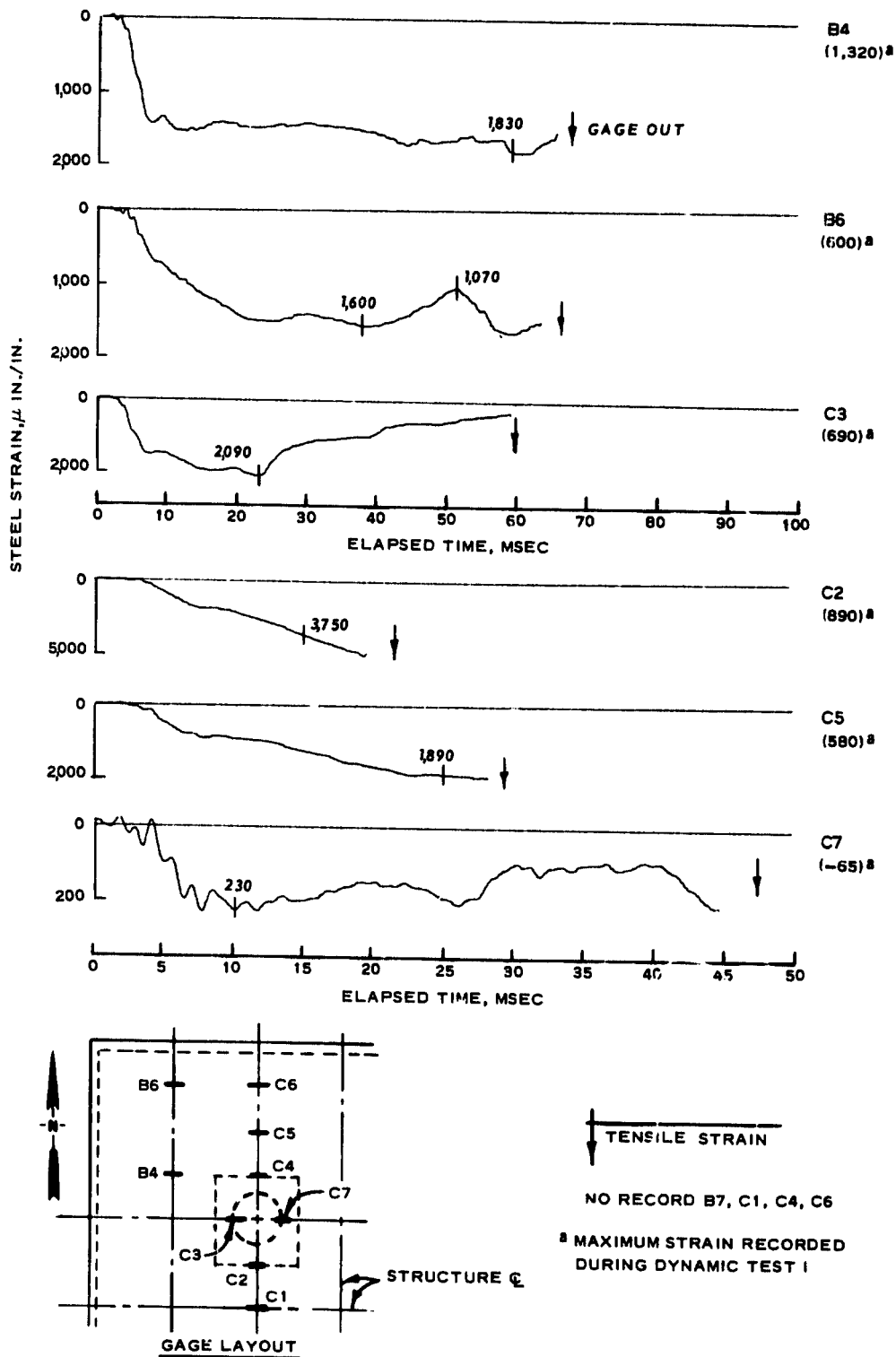


Figure B.24 Tensile reinforcement strains, Gages B4, B6, C2, C3, C5, and C7, Dynamic Test II.

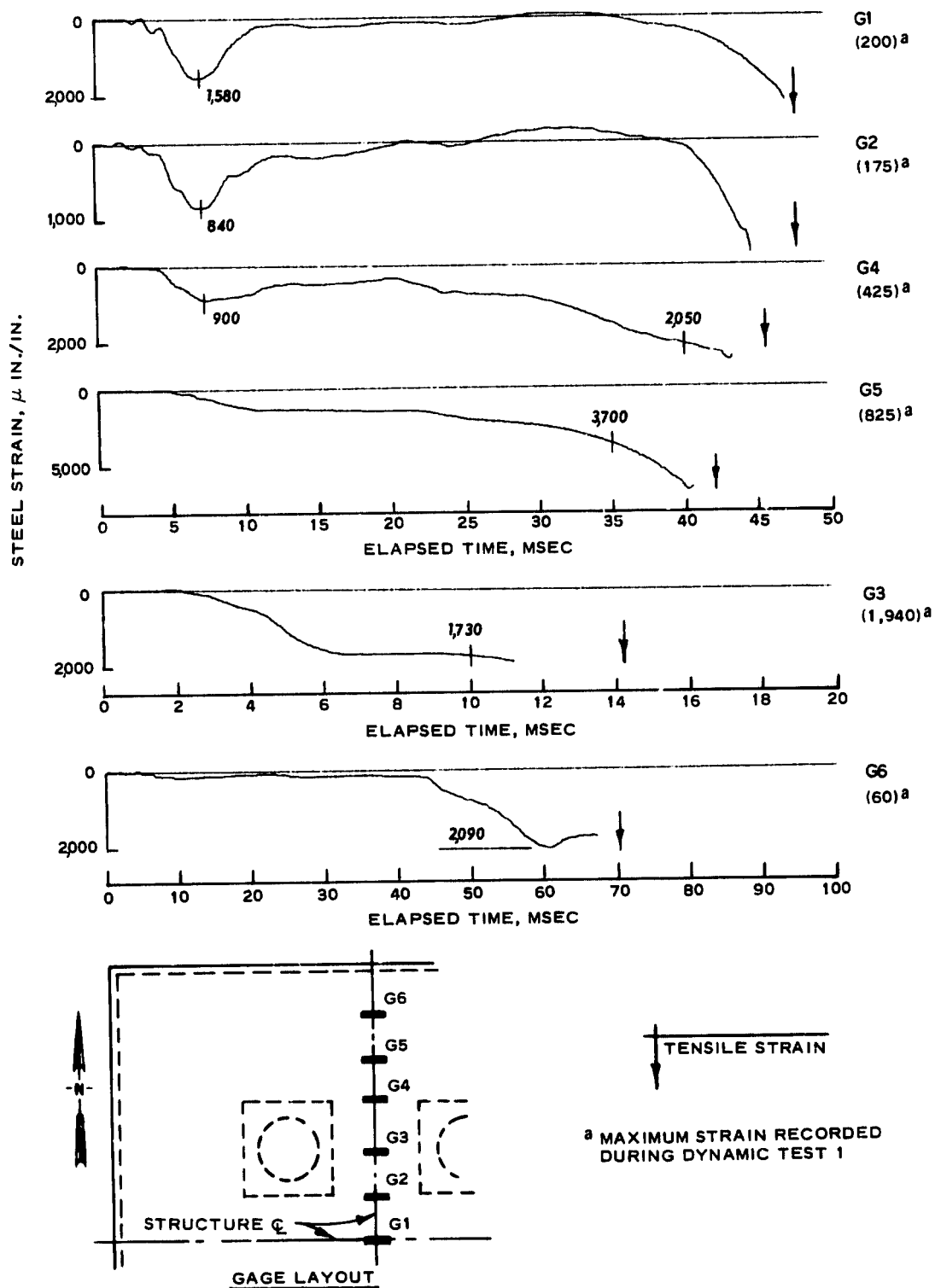


Figure B.25 Tensile reinforcement strains, Gages G1 through G6, Dynamic Test II.

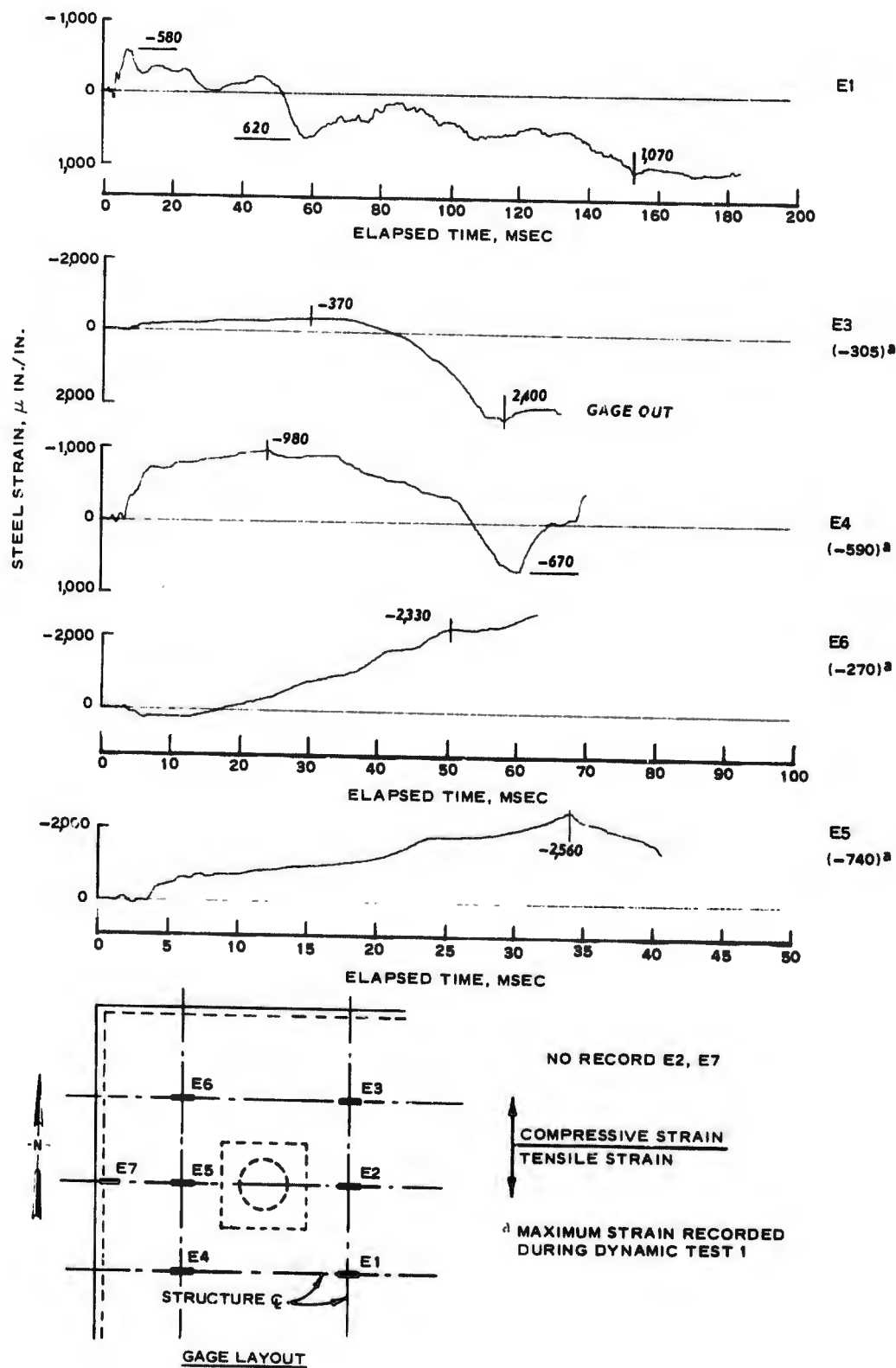


Figure B.26 Compressive reinforcement strains, Dynamic Test II.

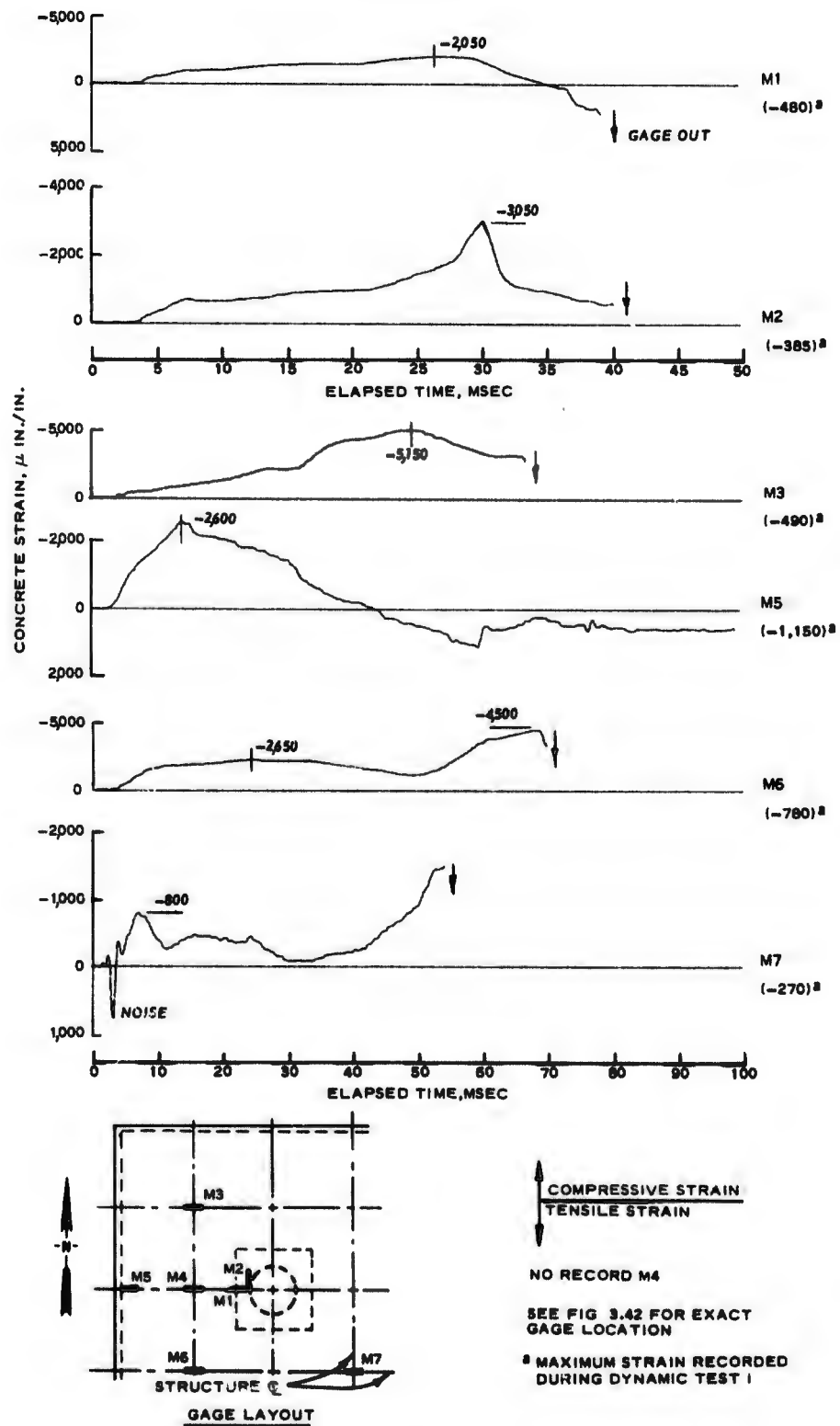


Figure B.27 Concrete strains, Dynamic Test II.

APPENDIX C
REVIEW OF PREVIOUS TESTS OF REINFORCED
CONCRETE SLAB SYSTEMS

Tests of several multiple-panel slabs and slab models have been reported in the literature. Many of these slabs were loaded to or slightly above the service load levels and not to ultimate conditions.

C.1 EARLY FLAT SLAB TESTS

From the time the first flat slab was constructed in 1906 (Reference 7) until 1921, many flat slab systems were load tested to demonstrate their structural integrity. The behavior of the flat slab and how it supports load, although widely discussed during this period, was not well understood (Reference 7).

The first reported flat slab test was the 1910 test of the Deere and Weaver Building in Minneapolis (Reference 6). Eight adjacent interior panels were loaded with a 350-psf superimposed load. Design values were 225-psf live load plus 115-psf dead load. Several other slab tests to service load or overload conditions followed.

In 1914, limitations for the moments needed in flat slabs were presented (Reference 35) based on statics, but these limitations were not accepted by many as being valid but as being unrealistically high. The steel strains observed in most slab tests were extremely small because the slabs were uncracked and the concrete tensile strengths were available to help provide the moment capacity over most of the slab areas, a behavior largely overlooked by the experimenters of that time.

In 1917, an 8-year-old flat slab scheduled to be razed and designed for a total load of 350 psf was loaded to near its strength (Reference 34). Four interior bays of this building, the Western Newspaper Union Building in Chicago, were loaded to slightly over 1,000 psf, including dead load. Steel stresses exceeding 50 ksi were measured in reinforcement passing over the columns.

Two flat slabs including four 16-foot-square panels were constructed at Purdue University in 1917 and were loaded up to the point of appreciable yielding of the tensile reinforcement. Details of these tests and of many of the acceptance tests on flat slabs are presented in Reference 35. Many early flat slab tests are also reviewed in Reference 36.

In Reference 35, the presentation of computed moments in flat slab systems and the reconciliation of slab test results with the requirements of statics by considering the influence of uncracked sections on observed reinforcement strains largely ended the need for load testing flat slab structures. However, the satisfactory performance and low observed reinforcement strains at service load levels of the early flat slab tests greatly influenced building code requirements. Flat slabs, and later flat plates, could in some cases be designed for considerably less than the requirements of statics (up to 28 percent below statics, Reference 7 through the 1963 ACI Code, Reference 1).

C.2 EARLY TWO-WAY SLAB TESTS

Because the mathematical solution for a rectangular plate simply supported on all edges has been known since the early 1800's, the need to load test structures in order to demonstrate the adequacy of the slab to the engineering profession was not necessary for the two-way slab system, which includes beams spanning between the columns. Indeed, inclusion of the effects of pattern loadings has long been recognized in the design methods for two-way slabs.

Reference 35 includes some discussion of a series of 52 single-panel slabs tested in Germany from 1911 to 1914 and of an 18-panel, full-scale, two-way structure in Waynesburg, Ohio, tested in 1920.

C.3 UNIVERSITY OF ILLINOIS NINE-PANEL SLAB TEST SERIES AND RELATED TESTS

An extensive theoretical and experimental investigation of slabs was conducted at the University of Illinois in the late 1950's and early 1960's. The purposes of these studies were to obtain a better understanding of the behavior of both the two-way slab system and the flat slab-flat plate slab systems and to provide information for formulating consistent design procedures for all slabs. The provisions for slabs in the 1971 ACI building code (Reference 2) are largely a result of this program.

C.3.1 Quarter-Scale Tests. Five 1/4-scale slabs consisting of

nine 5-foot-square panels arranged three on a side were constructed and tested as a part of the University of Illinois study. Many loading levels and patterns were used, including a test to failure with all panels loaded. All slabs were supported on only 16 columns. Test structures included a flat slab (References 36 and 37), a flat plate (References 36 and 38), that is, a flat slab without distinct drop panels or column capitals; a two-way slab with stiff beams (Reference 39), and a flat slab reinforced with welded wire fabric (Reference 40), all designed using the 1956 ACI Building Code (Reference 41), and a two-way slab with shallower beams (Reference 42) than were allowed by the 1956 ACI Code. The first four slabs were designed for a concrete strength of 3,000 psi and a working stress of 20,000 psi for the reinforcement.

A summary of the design loads, observed strengths, and failure modes is presented in Table C.1.

This series of five 1/4-scale test structures provided a wealth of information on the stress distributions and behaviors of common types of reinforced concrete slabs, demonstrated the higher factor of safety then demanded by design codes for slab systems with beams than for the flat slabs and flat plates (an inequity existing until the 1971 ACI Building Code was adopted), and exposed the vital role that punching shear around columns, torsional flexibility and failure of spandrel beams, and column adequacy in flexure have in determining the ultimate performance and strength of many reinforced concrete slabs.

C.3.2 Small-Scale Tests. A 1/16-scale model of the flat slab prototype previously constructed at 1/4-scale (Reference 37) was constructed and tested (Reference 43) to evaluate the feasibility of using this smaller scale for studies of strength and behavior and for investigating new or modified designs. Behavior of the 1/16-scale model under load was similar to that of the 1/4-scale model. Failure of the small model was initiated by distress in the side beam column connections at a total load of 620 psf and after a definite yield pattern had begun to form. This strength is approximately 70 psf above that of the 1/4-scale model, a difference largely explained by differences in material properties.

A second 1/16-scale flat slab model having the same dimensions as the first but with the reinforcement designed to provide the requirements of statics using a method proposed in Reference 37 was then constructed and tested. Deflections of this structure at design load levels were smaller than for the first model, and a maximum load of 730 psf on all panels was reached. Failure of the slab followed failure of several side columns in flexure. The center panel was then loaded to failure with loads approximately the same as the previous maximum being maintained on other panels. Yield of the center panel was reached at a deflection of nearly one slab thickness and at a load of 1,350 psf. Further deflection to four slab thicknesses produced little if any load increase. The results of this study demonstrate well how the strength of one local area of a slab can be much larger than the strength of the entire slab system.

Three 1/28-scale models of the flat plate prototype were constructed and loaded to failure in 1964 (Reference 44). These three models failed by punching shear at an interior column at 102 to 117 percent of the failure loading on the 1/4-scale model (Reference 38). Concrete strengths were similar.

Other small-scale, nine-panel slabs have been constructed and tested at the University of Oklahoma (Reference 45).

C.3.3 Three-Quarter-Scale Tests. A 3/4-scale model of the flat-plate prototype was constructed and tested at the Portland Cement Association (Reference 46). This 45-foot-square structure failed by punching shear at an interior column at a loading of 369 psf. This is only slightly above the strength of the 1/4-scale model, although the concrete strength of the larger structure was much higher (4,715 psi versus 2,510 psi).

C.4 OTHER LABORATORY TESTS OF REINFORCED CONCRETE SLAB SYSTEM MODELS

Many slab tests have been conducted recently to examine serviceability (deflections and cracking) and shear strength of the flat-plate system or to study the behavior of prestressed flat-plate systems. Structures using the two-way slab system have been used to investigate

the effects of in-plane forces on slab strength and behavior.

An experimental study of flat-plate structures was begun in Australia in 1959. Serviceability of this system including deflections and the use of prestressing and lightweight aggregate concrete were investigated. Four large multiple-panel slabs were included in the study (References 47 and 48). These were:

1. Mark I - A 3-1/2-inch-thick lightweight aggregate concrete slab containing three 9-foot bays and three 12-foot bays with end cantilevers. The slab was conventionally reinforced and supported on steel columns (Reference 47).

2. Mark II - A normal 4-inch-thick lightweight aggregate concrete flat plate containing six 9-foot-square panels (arranged 2 by 3) and supported on concrete columns (Reference 47).

3. Mark III - A 3-inch-thick straight-tendoned, posttensioned flat plate with three 9-foot spans in one direction and two 12-foot spans in the other, lightweight aggregate concrete (Reference 49).

4. Mark IV - A 3-inch-thick, normal-weight flat plate with draped tendons and containing two 12-foot spans in one direction and four 9-foot spans with 3-foot overhangs in the other (Reference 50).

At least the last three of these slabs have been loaded to failure. The Mark II slab failed at 330 psf when a shear failure developed at an edge column and before flexural yield lines had developed (Reference 47).

The Mark III slab failed at a load of 200 psf when an interior column punched through the slab (Reference 49). A yield-line pattern had not started to form, and the maximum deflection was approximately 2/3 inch.

Yield-line formation and a rapidly increasing deflection of over

2 inches was reached at 250-psf superimposed load with the Mark IV slab (Reference 50). When this load was released and reapplied, this slab failed at 240-psf load when a shear failure developed at a 9-inch-square interior column. The behavior of a prestressed slab was also reported in 1959 (Reference 51).

A 2-inch-thick flat plate with nine 6-foot-square panels supported on 3- by 14.4-inch edge columns and 3- by 28.8-inch interior columns has been tested to study the effects of elongated columns (Reference 52).

The design loads for the prototype for this 1/3-scale model included 50-psf live load plus approximately 75-psf dead load. At 250-psf load, the test was discontinued because of large deflections. Because of the very elongated columns, the slab behaved very much like a one-way slab. Shear distress at a corner column at a load of 220 psf was also observed. The center panel only was then loaded and supported 444 psf.

Considerable research has been reported on the effects of in-plane forces arising from lateral restraint of a single slab panel. The resulting data have been used in the design of a two-way slab system to include enhancement of strength due to compression membrane forces (Reference 53). This design was subsequently studied using a 1/4-scale, nine-panel, reinforced concrete model measuring 13-1/2 feet square and with a 1-15/16-inch slab thickness. With all panels loaded, a flexural failure occurred in the center panel at 850 psf, a load 118 percent above the strength given by a yield-line analysis in which in-plane forces were neglected. Tension membrane action had developed in the center panel at the termination of the test.

C.5 RECENT LOAD TESTS ON FULL-SCALE STRUCTURES

Load tests of a two-way slab floor in a large hospital building scheduled for demolition (Reference 54) demonstrated that the effects of in-plane forces and restraint on the strength of such a slab system can be very sizable, particularly if only a part of the slab is loaded.

After the 1964-65 New York World's Fair, a large waffle slab (essentially a 24-inch-thick flat plate structure with some concrete in the bottom of the slab removed) became available for testing. The structure was approximately 118 by 180 feet in plan with an average interior bay size of approximately 28 by 33 feet and was designed for a total load of 513 psf including 300-psf live load.

Three load tests to failure conditions were conducted on this waffle slab (Reference 55). Uniform loading over four adjacent interior panels resulted in a sudden failure at an applied load of 895 psf plus the approximately 267-psf actual dead load when the column common to the four panels punched through the slab. The flexural capacity was not

reached in this or the other two tests. The second test consisted of uniform loads on three edge panels. This test ended at a load of 874 psf (607 psf applied load plus dead load) when a punching shear failure developed at an exterior column. This lower-than-expected strength may have been the result of some differential settlement of the structure before testing had started. The third test consisted of uniform loading applied to only one interior panel. A punching shear failure at one corner of the loaded area occurred at a load of 2,239 psf (1,972 psf plus dead load). Results of these load tests again show the importance of adequate shear resistance and the greater strength of a slab when loaded over only one panel.

A portion of a 5-inch-thick flat plate roof slab designed for 50-psf live load was tested (Reference 56) to investigate the use of a reinforcement pattern with top reinforcement only in the column strip. The slab was supported on 24-inch-long, S6 x 12.5 shearheads located atop fairly irregularly located 3-inch-diameter steel pipe columns. The modified reinforcement distribution was used over half of the 2,032-ft² test area, and the normal reinforcement pattern was used over the rest. Both portions of the slab behaved similarly and supported the maximum superimposed load of 104 psf (dead weight = 69 psf) without noticeable distress.

The Nevada nuclear testing program included construction and testing of at least one full-scale reinforced concrete structure designed to serve the dual purpose of an underground parking garage and personnel shelter (Reference 57). A flat slab system with a 30-inch-thick slab and a 29-foot column spacing was used. Three feet of earth covered the flat slab, which was designed to withstand a 40-psi, long-duration load. An average peak-pressure of 31 psi (4,460 psf) on the roof surface resulted from the detonation of the Priscilla shot, an approximately 36-kt, 700-ft-high balloon burst. Peak slab deflections during the test approached 0.60 inch. No cracks were found in the roof slab after the test and the garage shelter interior had been judged to be undamaged.

TABLE C.1 SUMMARY OF RESULTS OF UNIVERSITY OF ILLINOIS 1/4-SCALE SLAB TESTS

Slab	(1) Flat Plate (Reference 38)	(2) Flat Slab (Refer- ence 37)	(3) Flat Slab with Struc- tural Fabric (Refer- ence 40)	(4) Two-Way Slab (Refer- ence 39)	(5) Modified Two-Way Slab (Refer- ence 42)

Design Live
Plus Dead
Load, psf

155 285 285 145 145

Yield Stress
of Reinforce-
ment, ksi

36.7 42.0 70.0 42.0 47.5

Ultimate
Load, psf

360 551 955 537 466

Failure Modes:

- (1) Shear or punching failure around an interior column. The flexural yield-line pattern was fairly well established except at the exterior edge adjacent to the spandrel beams around the perimeter of the slab.
- (2) Yield-line formation and flexural failure of south row of panels; torsional failure of south spandrel beams and failure of south side columns in flexure participated in the slab failure.
- (3) Flexural failure of an exterior row of panels hastened by a flexural failure of the side columns.
- (4) Torsional failures of spandrel beams and accompanying disruption of beam-column connections occurred after most flexural reinforcement except that at the exterior negative flexural reinforcement had yielded; some tensile membrane action had developed in the middle panel.
- (5) Large deflections from flexural yielding (deflections approached the slab thickness). Some torsional distress of edge beams developed.

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