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Structural Performance Evaluation
Of a·Building System

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Structural Performance Evaluation
Of a Building System

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Structural Performance Evaluation of a Building System*

Edward O. Pfrang and Felix Y. Yokel**

A full-scale, first-story portion of a building system was tested in the laboratory in such a manner as to simulate the structural behavior of a three-story building under both service and potential ultimate loading conditions. Additional tests were performed on the system components to provide behavioral data needed for the evaluation of the system.

Performance criteria for the evaluation of the structural safety and adequacy of certain building systems were developed. This report presents the results of the physical tests performed in the evaluation of the safety and structural adequacy of one such system, and discusses their significance. The report also presents data concerning the complex interaction between components which takes place in the building system.

The primary conclusions reached were:

(1) The system, as erected in the laboratory, satisfied the performance criteria which were set for its evaluation with a substantial margin. As a system, it exhibited strength and stiffness in excess of service and ultimate load requirements.

(2) The walls of the system behaved as an integral part of the structure. They provided most of the stiffness of the system with respect to lateral loads, and provided a significant portion of the stiffness against vertical loads.

Key words: Building systems; low-income housing; performance criteria; performance testing.

1. Introduction

It is now recognized that the United States has a severe housing shortage, particularly in the area of low-income housing. This shortage is of such magnitude and urgency as to make questionable its solution through conventional means. It appears that only systems-type solutions taking full advantage of the innovative capabilities of our advanced technology will be capable of coping with this problem economically and within an acceptable time framework.

Traditionally, structural innovations in building construction have been evolutionary rather than revolutionary and have taken place in small, carefully considered increments. Many of these incremental steps have been based upon extensive laboratory and analytical investigation. Progress has usually been based upon component testing and upon simplified and conservative analyses which do not fully account for system interaction. Because of these simplifications, the strengthening effects of so-called nonstructural portions of a building system are, in general, neglected and the complex interaction of components is frequently overlooked. As a result, in those few cases where tests on complete building systems have been performed, results have been obtained which in most cases indicate strength and rigidity far in excess of that predicted either by component testing or by conventional simplified analysis[1].[1] Strict reliance on these conventional concepts tends to inhibit innovative solutions to the building problem.

One solution to this dilemma would be full-scale system tests coupled with mathematical analysis. However, full-scale tests of large building systems are prohibitively expensive and time-consuming and are also difficult to interpret unless they are performed under ideal laboratory control. In addition, the development of mathematical theories generally depends upon a trial-and-error feedback process involving numerous cycles of physical testing. A more reasonable approach appears to be the execution in the laboratory of large-scale subsystem tests which simulate total system behavior. If such subsystems are carefully chosen and are tested in a manner designed to simulate the performance of the total system, and if they are supplemented by critical component tests, then they can be used as a basis for determining the structural adequacy of proposed innovative solutions. This report summarizes the results of such an evaluative study.

2. Objective and Scope

This report presents the results and evaluation of a structural performance test on an innovative building system. Criteria for structural performance and for performance testing are developed and are subsequently applied to system evaluation.

[1] Figures in brackets indicate the literature references on p. 25.

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The report summarizes the results obtained from a comprehensive series of load tests on a full-scale portion of the building system. The test portion of the structure was constructed in the laboratory under provisions of plans and specifications prepared for field erection of the system. The test structure was one story in height and was part of a three-story-high building, chosen and loaded in a manner that simulated the structural response of the complete building subjected to dead, live, and wind loads. The report also presents results obtained from tests on components and materials used in the test structure.

In addition to the evaluation of structural performance, significant information on structural interaction was derived from the data.

3. Notation and Conversion Units

The following notation is adopted for use throughout this report:

3.1. Service Loads

D = service dead load
L = service live load
H = service wind load

3.2. Simulated Loads on Test Structure (see fig. 6.1)

P = 2d story column loads
Hw = West wind load
Hs = South wind load at fire walls
$H's$ = South wind load between fire walls
w = floor load between columns
w' = floor load—cantilever section

3.3. Deflections

Dv = vertical gross deflection
Dvr = residual vertical gross deflection
dv = vertical net deflection
dvr = residual vertical net deflection
dvc = vertical net deflection at column support
$dvcr$ = residual vertical net deflection at column support
Dh = horizontal gross deflection
Dhr = residual horizontal gross deflection
dh = horizontal net deflection

3.4. Lengths

h = height above grade (ground level outside the building)
l = length of member
t = depth of member

3.5. SI Conversion Units

In view of present accepted practice in this country in this technological area, common U.S. units of measurement have been used throughout this paper. In recognition of the position of the United States as a signatory to the General Conference on Weights and Measures, which gave official status to the metric SI system of units in 1960, we assist readers interested in making use of the coherent system of SI units by giving conversion factors applicable to U.S. units used in this paper.

Length
    1 in = 0.0254* meter
    1 ft = 0.0348* meter

Area
    1 in² = 6.4516* × 10⁻⁴ meter²
    1 ft² = 0.09290 meter²

Force
    1 lb (lbf) = 4.448 newton
    1 kip = 4448 newton

Pressure, Stress
    1 psi = 6895. newton/meter²
    1 ksi = 6.895×10⁶ newton/meter²
    1 psf = 47.88 newton/meter²

Mass Volume
    1 lb/ft³ (lbm/ft³) = 16.02 kilogram/meter³

    Moment
    1 kip-in = 113.0 newton·meter

4. Performance Criteria

4.1. Introduction

Some criteria for performance testing have been developed such as ACI 318[2] and N. Y. State Building Code[3]; however, these criteria are not sufficient for the evaluation of comprehensive building systems. The criteria developed for the purpose of this evaluation use some of the existing criteria, supplemented by new criteria where necessary.

The performance criteria used in this report are presented and discussed in this section. First certain necessary definitions are developed; these are followed by Test Criteria and then Performance Criteria. Each Test and Performance Criterion is followed by a commentary. For convenience of reference, these Criteria are summarized in section 4.5.

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* Exactly

4.2. Definitions

4.2.1. Length of Members

The length of horizontal members is taken as the distance between the center lines of their supports or the clear distance between supports plus the member depth, whichever is smaller. In the case of a cantilever beam, the "length" is taken as twice its actual length.

4.2.2. Deflections

Deflection is the displacement of a point in a structure caused by the application of superimposed loads.

The magnitude of the deflection at a point is the component of the displacement of the point in the specified direction, measured from its position before the application of the superimposed loads which caused the deflection.

Horizontal deflections are measured in a direction parallel to the direction of the applied horizontal forces.

Gross deflection is the total deflection of a point.

Net deflection is the part of the gross deflection at a point which is attributable solely to the deformation of a structural member or assembly between its supports.

Residual deflection is the deflection at a point after removal of superimposed loads, measured relative to the position of the point before application of the loads.

4.2.3. Superimposed Loads

Superimposed loads are all loads applied to the test structure to simulate the dead, live, and wind loads acting on the real structure.

4.2.4. Failure

Failure of the structure or any structural component is defined as one of the following:

(a) An increase in deformation of an order of magnitude as defined in sections 4.4.1 and 4.4.3, occurring within 10 minutes [2] without increase in applied load.

(b) The inability of the structure to resist further load.

(c) Sudden major cracking, major spalling, or structural collapse.

4.3. Test Criteria

In order to satisfy the requirements for a

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[2] This time limit was introduced in order to distinguish between long-term creep and a deformation occurring over a relatively short period of time.

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performance test, the following Test Criteria must be satisfied.

4.3.1. Model Selection

A portion of the structure which is capable of simulating the response of the entire structure, and which will represent conditions providing the least margin of safety, shall be selected for testing. Foundation conditions shall be simulated in a manner representing the most adverse anticipated conditions that may exist in a complete structure in the field.

Commentary on Criterion 4.3.1. A similar criterion is developed in ACI 318[2]; however, the emphasis in this paper is on the requirement of having a section of the structure which (1) will represent the performance of the entire structure, and (2) will represent this performance in a conservative manner.

4.3.2. Loading

"Superimposed loads" shall be applied in a manner which will result in conditions equal to or more adverse or more adverse than the conditions in the full-scale structure and which provide the least margin of safety.

Comentary on Criterion 4.3.2. Criterion 4.3.2 requires a simulation which is conservative. It is recognized that exact duplication of field conditions cannot be achieved and should not be attempted. Instead it is required that the test simulate superimposed loads in a manner which will provide the least margin of safety that may exist under any circumstances.

4.4. Performance Criteria

4.4.1 Horizontal Deflection Under Dead and and Wind Load

At a load level of 0.9 dead + 1.1 wind $(0.9D + 1.1H)$ the Horizontal deflection due to the superimposed load of 1.1 wind $(1.1H)$ shall not exceed the following:

$$Dh \leq 0.002h$$

where:

Dh = horizontal gross deflection
h = height above grade.

Commentary on Criterion 4.4.1. Generally a structure will experience its most severe lateral deflection under a condition of minimum vertical load and maximum lateral load. This criterion is designed to prevent excessive deflection under this condition of loading, and

provides a margin of 10 percent over the maximum lateral loads likely to occur under service conditions.

There has been limited experience with high-rise apartment structures which indicated that when such a structure is designed to permit horizontal deflection in excess of $h/400$ to $h/500$ under maximum service wind loads, discomfort and anxiety are experienced by some of the occupants under severe wind conditions. Although it is extremely conservative for low-rise structures, this deflection limitation is adopted here since a more comprehensive criterion has not been developed.

4.4.2. Horizontal Deflection Under Dead, Live, and Wind Load

At a load level of 1.3 Dead + 1.7 Live + 0.8 Wind $(1.3D + 1.7L + 0.8H)$, the horizontal deflection due to the superimposed load of $0.3D + 1.7L + 0.8H$ shall not exceed the following:

$$Dh \leq 0.002h.$$

Commentary on Criterion 4.4.2. Even though the most critical loading with respect to horizontal deflections of a structure is in many cases a combination of minimum vertical and maximum lateral loads, maximum vertical loads combined with lateral loads may be more critical. This criterion imposes conditions which represent the highest loads which should cause no permanent structural damage. It would be unrealistically conservative to impose maximum lateral loads. A lesser lateral load is therefore adopted for this criterion, accounting for the low probability of simultaneous action of maximum vertical loads, combined with maximum wind forces.

4.4.3. Vertical Deflections Under Service Live Load

At a load of 1 dead and 1 live $(1D + 1L)$, the vertical deflection due to the superimposed load of 1 live $(1L)$ shall not exceed the following:

$$dv \leq \frac{l}{360}$$

where:

dv = vertical net deflection.
l = length of member.

Commentary on Criterion 4.4.3. Criterion 4.4.3 is based on the proposition that $\frac{l}{480}$ rep-

resents a reasonable maximum allowable instantaneous deflection under service loads. Prevailing codes usually set $\frac{l}{360}$ as a deflection limitation; however, studies[4] have indicated that this deflection is excessive in terms of user comfort and causes minor distress to finishes and partitions. The proposed $\frac{l}{480}$ deflection limitation reasonably represents present-day consensus based on limited knowledge in this area.

4.4.4. Sustained Load Deflection

At a load level of 1.3 dead + 1.7 live $(1.3D + 1.7L)$ sustained for 24 hours, deflections due to the superimposed load of 0.3 dead + 1.7 live $(0.3D + 1.7L)$ shall not exceed the following:

(a) $\quad dv \leq \dfrac{l}{360} \times \dfrac{(0.3D + 1.7L)}{L}$

(b) $\quad Dh \leq 0.002h.$

Residual deflection due to the superimposed load of $0.3D + 1.7L$, measured not later than 24 hours after removal of the superimposed load, shall not exceed the following:

(c) If $dv > \dfrac{l^2}{20{,}000t}$, $\qquad dvr \leq 0.25\,dv.$

 If $dv \leq \dfrac{l^2}{20{,}000t}$ $\qquad dvr \leq \dfrac{l^2}{80{,}000t}.$

(d) $\quad Dvr \leq dvr + 0.25\,(Dv - dv)$

where:

Dv = vertical gross deflection
Dvr = residual vertical gross deflection
dv = vertical net deflection
dvr = residual vertical net deflection
l = length of member
t = depth of member.

Commentary on Criterion 4.4.4. Structures should not suffer large irreversible deformations under loads which are lower than their ultimate design loads. It is therefore reasonable to require structures to resist superimposed loads up to 90 percent of their ultimate design loads without suffering significant irreversible deformations.

Under most codes, $1.3D + 1.7L$ is about 90 percent of the ultimate design load. This is therefore the highest load which should be reasonably expected to cause no permanent structural damage. The deflection limitation in

4.4.4(a) represents an extrapolation of the service load deflection limit of $\frac{l}{480}$ with an additional allowance for creep deflection.

Criterion 4.4.4(b) is similar to criteria 4.4.1 and 4.4.2, which have been discussed earlier.

Criterion 4.4.4(c) requires 75 percent recovery of vertical deflections. This guards against structural systems which experience significant permanent sets in each cycle of loading that may lead to progressive incremental collapse. By permitting residual deflections of up to 25 percent of initial deflections, reasonable tolerances are provided for creep and system slack. The 75 percent recovery requirement is relaxed for very stiff structural systems $(dv \leq \frac{l^2}{20,000t})$, since there are invariably some small irrecoverable deformations in all structures.

Criterion 4.4.4(d) requires 75 percent recovery of deflections in excess of "net" deflections. These deflections are primarily due to deformations of columns and/or walls and this criterion provides reasonable tolerances for creep and system slack. Lower limits for recoverable deformations cannot be set in this case until further studies are conducted.

4.4.5. Ultimate Strength

The structure or any portion thereof shall not fail at a load smaller than the following:

(a) 1.25 (1.5D + 1.8L)

(b) 0.9D + 1.4H.

Commentary on Criterion 4.4.5. Criterion 4.4.5(a) is for ultimate vertical loads. It is assumed that a structure may in extreme cases fail under loads which are as much as 20 percent below the average failure loads for similar structures (or of computed "ultimate" loads). In absence of a statistical sample of any size it is necessary to assume that if the laboratory sample has a strength of 1.0, the structure simulated by the sample may have a strength as low as $1 - 0.2 = 0.8$. It is therefore required that the laboratory sample be capable of withstanding a load of $\frac{1}{0.8}$, or 1.25 times the design ultimate load, which was taken as 1.5D + 1.8L.

Criterion 4.4.5(b) is tentatively adopted for ultimate lateral load, following the same philosophy with an ultimate load of 1.1H.

4.5. Summary of Test and Performance Criteria

The preceding Test and Performance Criteria are summarized in this section for ease of reference. The criteria numbers remain unchanged.

4.3.1. Model Selection

A portion of the structure which is capable of simulating the response of the entire structure, and which will represent conditions providing the least margin of safety, shall be selected for testing. Foundation conditions shall be simulated in a manner representing the most adverse anticipated conditions that may exist in a complete structure in the field.

4.3.2. Loading

Superimposed loads shall be applied in a manner which will result in conditions equal to or more adverse than the conditions in the full-scale structure which provide the least margin of safety.

4.4.1. Horizontal Deflection Under Dead and Wind Load

At a load level of 0.9D + 1.1H, the horizontal deflection due to the superimposed load of 1.1H shall not exceed the following:

$$Dh \leq 0.002h.$$

4.4.2. Horizontal Deflection Under Dead, Live, and Wind Load

At a load level of 1.3D + 1.7L + 0.8H, the horizontal deflection due to the superimposed load of 0.3D + 1.7L + 0.8H, shall not exceed the following:

$$Dh \leq 0.002h.$$

4.4.3. Vertical Deflections Under Service Live Load

At a load level of 1D + 1L, the vertical deflection due to the superimposed load of 1L shall not exceed the following:

$$dv \leq \frac{l}{480}.$$

4.4.4. Sustained Load Deflection

At a load level of 1.3D + 1.7L sustained for 24 hours, deflections due to the superimposed

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load of $0.3D + 1.7L$ shall not exceed the following:

(a) $dv \leq \dfrac{l}{360} \times \dfrac{(0.3D + 1.7L)}{L}$

(b) $Dh \leq 0.002h$.

Residual deflection due to the superimposed load of $0.3D + 1.7L$, measured not later than 24 hours after removal of the superimposed load, shall not exceed the following:

(c) If $dv > \dfrac{l^2}{20,000t}$, $\qquad dvr \leq 0.25\, dv$.

If $dv \leq \dfrac{l^2}{20,000t}$ $\qquad dvr \leq \dfrac{l^2}{80,000t}$.

(d) $Dvr \leq dvr + 0.25\,(Dv - dv)$

4.4.5. Ultimate Strength

The structure or any portion thereof shall not fail at a load smaller than the following:

(a) $1.25\,(1.5D + 1.8L)$

(b) $0.9D + 1.4H$.

5. Test Structure

The structure as erected and tested in the laboratory was a full-scale subsection of a modular building system. It was designed and constructed by Neal Mitchell Associates Inc. under the provisions of section 4.3.1 of the performance criteria. The plans and specifications were prepared by Neal Mitchell Associates Inc., and are dated 9-13-67. These should be referred to for detailed information. Some typical drawings from these plans were reproduced in this report and modified for illustrative purposes with the permission of Neal Mitchell Associates, Inc.[a]

This section of the report contains a description of the proposed structure, a description of the test structure as erected in the laboratory, and a discussion of the fidelity with which actual field conditions are simulated in this test.

5.1. Proposed Structure

A typical complete structure is illustrated in figure 5.1.* The proposed structural system consists of:
1. Precast components;
2. Cast-in-place topping slabs;
3. Gypsum walls; and
4. Foundations, grade-beams, and slabs on grade.

5.1.1. Precast Components

The precast components of the proposed structure are: (1) columns, (2) main beams, (3) tie beams, and (4) floor channels. Figure 5.2 illustrates an assembled structural frame which contains all the precast components. The frame is illustrated in more detail in figure 5.3. Figure 5.4 shows the erection of a frame.

Figures 5.5 through 5.16 show detailed drawings of the precast components. Figure 5.5 is an isometric view of the main beam, tie beam, and column reinforcement at a connection. Figures 5.6 and 5.7 show typical column details. Main beam details and sections are shown in figures 5.8 and 5.9. Tie beam details are illustrated by figure 5.10.

In accordance with the plans and specifications, main beams, tie beams, and columns are precast of cellular concrete with lightweight aggregate. The nominal wet density of the concrete is 95 lb/ft³ and specified nominal 28-day strength is 4,500 psi for lower-story columns and 3,500 psi for all other precast components. The wet density of this concrete is controlled by the addition of preformed foam at the time of mixing. Reinforcing bars are ASTM-A61[5] (60 ksi)[4] steel for primary reinforcement and ASTM-A15[6] (40 ksi) steel for stirrups, ties and other reinforcement.

Column-beam connections and end details are illustrated in figures 5.11 through 5.15. Figure 5.11 is an isometric view of a disassembled connection. Tie beam end details are shown in figure 5.12; figures 5.13 and 5.14 show the column end detail, and figure 5.15 shows the details of an assembled connection.

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[a] Written permission has been granted by Neal Mitchell Associates to include figures 5.3, 5.5, through 5.21 and 5.23.

[4] All figures referred to in the text appear in the section beginning on p. 31.

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Joints are connected by bolts, then grouted. The grout mix is not specified. The following grout mix was used in the test structure:

1 part Type I cement

2 parts of masonry sand

3 oz per 1 lb of cement of a polyvinyl acetate emulsion.

The floor channel details are shown in figure 5.16. These elements are standard, commercially available precast concrete roof tile.

Concrete used in channel units consists of $3/8$-in maximum size lightweight aggregate ("block-mix"). Air-dry unit weight of concrete is 103 lb/ft^3 and 28-day strength ranges from 4000 psi to 5,500 psi ($7\frac{1}{2}$ sack mix).

Reinforcement consists of a No. 4 deformed intermediate grade (ASTM-A15) steel bar in each leg of the channel and a 34-1412 wire mesh (ASTM-A185) [7] in the back of the channel with the 14-gage wire in the longitudinal direction.

The top of the channel is very rough to develop resistance to horizontal shear between the supporting channel and the topping slab.

5.1.2. Cast-in-Place Topping Slabs

The topping slabs have a specified nominal thickness of 2 in. Concrete is made of $3/4$-in maximum size lightweight aggregate, with a weight of 110 lb/ft^3 and a nominal 28-day strength of 3,000 psi. Reinforcement is 66-1010 wire mesh (ASTM-A185) set one inch from the top of the slab. Additional reinforcement is provided at the main beams by the shear connectors and by two No. 4 bars on the first floor and two No. 3 bars on all other floors, as shown in figure 5.8. This reinforcement is ASTM-A61 steel.

5.1.3. Gypsum Walls

There are three kinds of gypsum walls:
   (1) Fire walls,
   (2) Exterior walls, and
   (3) Interior walls.
The location of these walls in the structural system is illustrated in figure 5.17.

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[4] Plans and specifications for the Neal Mitchell Housing System permit the option of using 50 ksi or 60 ksi steel. The steel used in the test structure had a nominal yield of 60 ksi. See section 11 for the actual properties.

Figure 5.18 shows a typical cross section of the fire walls. The walls are installed in every second bay in the "short" direction of the building (N-S in the test structure). These walls are continuous in all spaces between columns and have no openings in these spaces. The full width of a building will therefore contain two such uninterrupted firewalls in every second bay. (See figure 5.17.)

The fire walls are standard dry-wall construction. Metal channels are attached to the concrete members with power-actuated fasteners at a 6- to 8-in spacing. Metal studs $2\frac{1}{2}$ in $\times$ 25 gage) are spaced 16-in on center. Wallboards on either side of the metal studs consist of one $1/2$-in gypsum backing board (ASTM-C442) [8] and one $5/8$-in gypsum wallboard (ASTM-C36 [9].

The wallboards are fastened to the studs by screws spaced 8 to 12 in o.c., which is a closer spacing than that used in standard practice. The details of the actual fire wall installation in the test structure are illustrated in figure 5.19.

Figure 5.20 shows a typical section of the exterior walls. "Exterior walls" as defined here are the outer walls in the long direction of the proposed building (E-W direction in the test structure). Each building will thus have two exterior walls. (See fig. 5.17.) Exterior walls are located in the outer rows of columns and fill the 10 ft space between columns. The walls are not continuous and each panel may contain a door or a window.

Exterior walls are standard dry-wall construction. Channels and studs are as in the fire walls. Facing consists of $5/8$-in gypsum wallboards (ASTM-C36 on either side of the stud; screw spacing is as in the fire walls. The wall surface exposed to the atmosphere will be protected by optional siding.

"Interior walls" as defined here extend along the two interior rows of columns in the long direction of the building (E-W direction in the test structure). Each building thus has two interior walls in the long directions. These walls fill the 10-ft panels between columns (see fig. 5.17). A 3-ft door may be expected in every second panel.

Several types of interior walls are used in the Mitchell System; of these the standard 2½-in "structicore" partition wall construction was deemed to have the least resistance to lateral load and was thus chosen for the laboratory structure. Figure 5.21 shows typical sections of an interior wall, and figure 5.22 shows a typical interior wall partially dismantled.

5.1.4. Foundations, Grade Beams, and Slabs on Grade

Foundation plans are shown in figure 5.23. All foundations are specified as ready mix concrete with a 28-day strength of 3,000 psi. Slabs on grade are ready mix concrete with a specified 28-day strength of 2,000 psi.

Reinforcement is ASTM-A15 intermediate grade steel and ASTM-A185 welded wire mesh.

Lower-story columns are encased in the foundations. (See fig. 5.7.)

5.2. The Test Structure

5.2.1. Structural Simulation

The test structure before and after installation of the walls is illustrated in figures 5.24 and 5.25, respectively. It comprises a part of the complete structure, made up of full-scale components and erected in the laboratory. The test structure as part of the complete structure is illustrated in figure 5.2.

The performance of the complete structure is simulated in the test structure by:

(1) applying to the test structure all live loads which under field conditions would act directly on the test structure;

(2) simulating all forces caused by dead, live, and wind loads which would be exerted on the test structure by the rest of the structure under field conditions.

The test structure is thus treated in the laboratory as a "free body." The test structure was so chosen that all aspects of structural performance in the field could be simulated under laboratory test conditions. The test structure corresponds to a part of the total structure which is cut off below the slab on grade.

5.2.2. Description of the Test Structure

The test structure was constructed under the provisions of the plans and specifications of the Neal Mitchell System; however, properties of materials and structural details did not always agree with these plans. Detailed information about materials used in the test structure is presented in section 11. Deviations from plans in structural details are noted in this section.

The test structure consised of:
(a) Precast components,
(b) A cast-in-place topping slab,
(c) Walls,
(d) A cast-in-place floor slab.

5.2.2.1. Precast Components

The precast components consisted of:
(a) Six precast columns, which were similar in dimensions to the lower-story columns in the proposed structure (see figs. 5.6 and 5.7) except that they were shortened to a length of 8 ft-5 in, since no embedding in foundations was included.

(b) Three main beams (see figs. 5.8 and 5.9).

(c) Four tie beams (see fig. 5.10).

(d) Eight 2-ft-wide and two 1-ft-wide floor channels. (See fig. 5.16.) The narrow channels were placed along the north and south edge of the structure. Reinforcement in the channel legs consisted of No. 5 deformed intermediate grade (ASTM-A15) bars, instead of the No. 4 bars specified in the plans.

5.2.2.2. Cast-in-Place Topping Slab

The specified thickness of the cast-in-place topping slab is 2 in. The average slab thickness as built in the test structure was 2½ in ± ⅛ in measured from the top of the main beam. The top of the floor channels was irregular and tended to be somewhat higher than the top of the main beam, producing a somewhat lesser average thickness than the measured 2½ in. But even after allowing for a thickness reduction due to floor channel irregularity, the as-built thickness was still in excess of the specified 2-in thickness.

5.2.2.3. Walls

The test structure had the following walls:
(a) East and west walls were "fire walls" as described in section 5.1.3.a, except that ⅜-in gypsum backing boards and ½-in. wallboards were used instead of the thicker sizes called for in the plans.

(b) The south wall was an exterior wall as described in section 5.1.3.b except that the exterior siding was omitted and ½-in thick wallboards were used instead of the ⅝-in thickness shown in the plans.

(c) The north wall was an "interior wall" as described in section 5.1.3(3).

All channels for the wall system were attached by power-actuated fasteners to the floor slab and the structural frame. Vapor seals and insulation between walls were omitted since these materials do not add to the strength of the structure. The omission of exterior siding on the south wall may have slightly decreased the stiffness of that wall, which would cause the test results to be lower than they might otherwise have been.

Each panel in the south wall contained a 5 ft × 7 ft aluminum doorframe on its west side. This represents the least stiff condition that may be encountered in the field.

The western panel of the north wall contained a 3 ft × 7 ft wooden doorframe on its east side. This simulates field conditions.

5.2.2.4. Floor Slab

The cast-in-place floor slab was poured on top of a vinyl sheet which was spread on the laboratory floor. The floor slab was subsequently post-tensioned against the laboratory floor by four 1½-in-diameter bolts in order to prevent sliding due to lateral test forces applied to the structure. Tests indicated that the floor-slab concrete had a 17-day compressive strength of 5,600 psi. Slab reinforcement consisted of a 66–1010 mesh (ASTM-A185).

The slab was poured around the columns which were lined by ½-in asphalt-impregnated fiberboard, thus forming full-depth pockets at the column seats to permit column rotation at the base. A ⅛-in-thick neoprene sheet was inserted between the column base and the laboratory floor.

5.2.2.5. Materials

Standard compression tests (ASTM C33–66) were carried out on cylinders of concrete from the "cast-in-place" slabs and the precast members with the exception of the floor channels. In all cases concrete strength exceeded the strength specified in the plans.

Reinforcing bars were ASTM–A61 (60 ksi) steel wherever the plans permit the option of using 50 or 60 ksi steel.

5.3. Fidelity of Simulation of Field Conditions by the Test Structure

Complete full-scale structures can be and have been tested in the field. While such field tests provide a means for the observation of the performance of a complete structure, it should also be noted that when compared with laboratory tests, field tests have many disadvantages. Some of the more obvious disadvantages are: cost; the time required to erect and test a full-scale structure in the field; changing conditions of temperature and wind; and the difficulty of precise application of loads and measurement of deformations. The major advantages of field testing are the ability to test an entire structure and a better simulation of foundation conditions.

For the case reported here, the entire test was performed inside the laboratory facilities of the National Bureau of Standards. Since it was impractical to erect a complete structure in the laboratory, it was decided to construct a portion of the structure and to test it in a manner that simulated the performance of the complete structural system. A lower-story section was selected, since lower-story components are subjected to the most critical loading conditions.

The load program to which the test structure was subjected is discussed in section 6. The fidelity of the simulation is discussed in the following sections.

5.3.1. Interaction Between the Test Structure and the Complete Structure

Figure 5.2 illustrates the test structure as part of a complete structure. The test structure with the testing equipment installed is shown in figures 5.26 and 5.27. In an actual building, the test structure would be connected to the remainder by:

(a) Columns,
(b) Abutting tie beams and main beams,
(c) A continuous topping slab, and
(d) Walls.

At all of these connections forces are exerted on the test structure, either by direct transmission of loads carried by the connected members or by restraining effects on motion of connected members. It is neither feasible nor necessary to simulate all these effects. Simulation of the most adverse conditions will generally lead to simplified approximations which are on the conservative side. Simulation of structural interaction at these four points of continuity is discussed in the following:

5.3.1.1. Columns

Upper-story columns will transmit to the beam-column connection most of the dead loads generated by the stories above and the live loads acting on these stories.

For the laboratory model it was assumed that the upper-story columns would transmit the following loads to the joint at their base:

(1) Dead loads of the upper stories.

(2) Vertical live loads on the upper stories.

In reality the columns between the fire walls will also transmit a certain amount of horizontal wind-induced shear load. However, as will be noted later, in the presence of the partition walls, only a negligible amount of the total wind shear was carried by the columns. The wind shear from the upper stories was assumed to be carried by the walls to the top slab, which in turn will transmit the shear to the partition walls below.

It will also be noted later that some of the vertical loads are carried by the wall system directly into the foundations. The assumption that the entire vertical load is carried by the columns is a conservative assumption with respect to columns. The fact that the walls could potentially be more highly stressed in the complete structure than in laboratory simulation does not appear to be of significance, since a wall failure by vertical loads would not occur without a simultaneous column failure. Column loads were applied vertically by rams at the center line of the lower-story columns as illustrated in figure 5.28. Rollers were inserted to roll in the direction of racking and to minimize frictional forces which might resist racking while vertical loads were applied.

It is recognized that upper-story columns would transmit moments as well as vertical loads, while the rams applied only axial vertical loads. It is demonstrated in Appendix B that this application of column axial loads is conservative.

5.3.1.2. Abutting Tie Beams and Main Beams

Main beams are discontinuous at both of their ends in the real structure, and this was correctly reflected in the test structure. Tie beams may be either continuous or discontinuous depending on their position in the structure. If tie beams were continuous on either or both sides of the test structure, this would result in increased load-carrying capacity and decreased deflections. Thus, it may be stated that with respect to structural continuity the test structure represents a conservative approximation.

5.3.1.3. Continuity of Topping Slab

In the complete structure, topping slabs may be continuous on three sides of the test section —west, north, and east— or on two adjacent sides of the test section. The severing of this continuity in the test structure represents a conservative approximation with respect to both load-carrying capacity and deflection.

5.3.1.4. Walls

The wind load is imparted to the wall by

(1) shear along its upper connection to the beam above it, and (2) bearing of the windward column against the wall.

Since the floor system is very rigid in relation to beam column joints and walls, the horizontal forces acting above any floor are transmitted into this floor by the walls and in turn essentially equally distributed among the walls below this floor by a uniform displacement of the entire floor.

In the test structure, simulated wind loads equal to one-half the wind loads generated by the entire contributory portion of the three-story building were imparted at the end of each main or tie beam by a ram load, as illustrated by figure 5.26. In the case of the north direction, a wind load was also applied at the main beam on top of the column between the two fire walls. Due to the stiffness of the floor system, these wind loads have a net effect equal to the effect that may be expected on a structure in the field. The reason for applying only one-half of the wind force to each wall is the above-discussed assumption of great floor stiffness, which would distribute the wind load to two wall panels in the north direction and to more than two wall panels in the east direction.

Test results also indicate that the walls participate in the support of vertical loads. This was demonstrated by the fact that deflection of main beams connected to fire walls increased almost fivefold when these walls were removed. As will be noted later, the loading applied in Test No. 9 more than compensated for any adverse effect of vertical loads on the walls under service load conditions. Column loads were computed without regard to possible wall participation in load support. It is therefore concluded that the simulation of wall action adequately represented the most adverse conditions that may be expected in a complete structure.

5.3.2. Simulation of Foundation Conditions

Column foundations in the proposed building extend to a 6-ft depth below grade for exterior columns, and 3-ft depth below the top of the floor slab for interior columns (see fig. 5.23). Exterior column footings are also tied into the perimeter wall for added fixity. This configuration provides some degree of fixity at the column base, the degree depending on prevailing soil conditions.

In the test structure, the columns were "cut off" at the bottom of the floor slab. The lower ends of the columns were provided on all sides with a 1/2-in-thick asphalt-impregnated fiberboard expansion joint against which the floor slab was cast, thus providing a detail similar to that of the real structure where a 3/8-in premolded filler is placed around the

column. The base of the column was set on a 1/8-in-thick neoprene bearing pad which rested on the laboratory test floor. The resulting column connection permitted the column base to rotate, and therefore was a conservative simulation of the real structure, where the foundations provided partial column base fixity.

5.3.3. Simulation of Live Loads

Vertical live loads on the top slab of the specimen were simulated by air-bags which were held down by a suitable reaction system (see fig. 5.26). This created a uniformly distributed load which was able to follow the deflections of the slab. Air bags were made of 20-mil polyethylene and were designed to withstand 300 psf (7 times live load). The live loads applied represented a valid simulation of live load conditions as used in structural design.

Horizontal live loads were applied by horizontal 10-ton rams as illustrated in figures 5.26 and 5.29.

The validity of wind load simulation has been discussed in section 5.3.1.d.

6. Load Program

6.1. Introduction

The load program in this test had three objectives:

(1) Evaluation of the structural adequacy of the proposed system and determination of its ability to satisfy the performance criteria established in section 4.

(2) The acquisition of additional information about the behavior of complex structural systems and the interaction of their components.

(3) The development of suitable methods of performance testing for complex structural systems.

Section 6.2 explains the assumptions which were made with regard to the magnitude of applied live and wind loads and section 6.3 explains the load schedule. Load computations and the detailed sequence of loading used in each test, are presented in Appendix C.

6.2. Applied Loads

All applied loads were determined in accordance with "Minimum Design Loads in Buildings and Other Structures," USASI A58–1955 [11].[5] The following unit service loads were used:

Occupancy loads (floor) —40 psf
Snow loads (roof) —30 psf
Wind loads (walls) —20 psf

6.3. Loading Schedule

Figure 6.1 shows schematically how the test loads were applied to the structure. Table 6.1* explains the symbols used to represent the test loads and the magnitude of these loads. Table 6.2 summarizes the magnitude of test loads which represent the performance criteria.

Tests were conducted on the test structure with walls installed, and subsequently on the same structure after the walls were removed. All load tests were conducted between May 10, 1968, and May 22, 1968, and are listed hereafter.

6.3.1. Tests Conducted on the Structure With Walls Installed

Test 1: Column loads to $0.9D$

Test 2: Column loads of $0.9D$
South wind load to 25 psf
$(0.9D + 1.25H)$

Test 3: Column loads of $0.9D$
West wind load to 25 psf
$(0.9D + 1.25 H)$

Test 4: Column loads to $1.3D + 1.7L$
Major floor load to $1.3D + 1.7L$
$(1.3D + 1.7L)$

Test 5: Column loads of $1.3D + 1.7L$
Major floor load of $1.3D + 1.7L$
Loads sustained for 24 hours
$(1.3D + 1.7L)$

Test 6: Column loads of $1.3D + 1.7L$
Major floor load of $1.3D + 1.7L$
$(1.3D + 1.7L)$

Test 7: Column loads of $1.3D + 1.7L$
Major floor load of $1.3D + 1.7L$
South wind load to 15 psf
$(1.3D + 1.7L + 0.8H)$

Test 8: Column loads of $1.3D + 1.7L$
Major floor load of $1.3D + 1.7L$
West wind load to 15 psf
$1.3D + 1.7L + 0.8H)$

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Test 9:	Column loads of 1D Major floor load to 160 psf (1D + 3.5L)
Test 9-A:	Column loads of 1D Major floor load to 160 psf Minor floor load to 160 psf (1D + 3.5L)
Test 10:	Column loads of 0.9D South wind load to 60 psf (0.9D + 3H)
Test 11:	Column load of 0.9D West wind load to 74 psf (0.9D + 3.7H)

6.3.2. Tests Conducted on the Structure After the Removal of Walls

Test 12:	Column load of 1.3D + 1.7L Major floor load to 1.3D + 1.7L Rollers under column loads oriented to permit sway in the east-west direction (1.3D + 1.7L)
Test 12-A:	Column load of 1.3D + 1.7L Major floor load to 1.3D + 1.7L Rollers under column loads oriented to permit north-south sway (1.3D + 1.7L)
Test 13:	Column load of 1.3D + 1.7L Major floor load to 1.3D + 1.7L Minor floor load of 1.3D + 1.7L Rollers under column loads oriented to permit east-west sway (1.3D + 1.7L)
Test 13-A:	Column load of 1.3D + 1.7L Major floor load to 1.3D + 1.7L Minor floor load of 1.3D + 1.7L Rollers under column loads oriented to permit north-south sway (1.3D + 1.7L)
Test 14:	Column loads of 0.9D South wind load of 10 psf (0.9D + 0.5H)
Test 15:	Column loads of 0.9D West wind load of 16.5 psf (0.9D + 0.8H)

Test 16:	Column loads of 1D Major floor load of 280 psf (1D + 8.4L)
Test 16-A:	Column loads of 1D Major floor load of 370 psf Minor floor load of 280 psf (1D + 6.3L)
Test 17:	Column load of 60 kips on four outer columns (1D + 7L)
Test 18:	Column load of 0.9D South wind load of 10.5 psf (0.9D + 0.5H)

7. Instrumentation

A total of 98 electrical resistance instruments were used to monitor and record structural deformational behavior of the test model. These instruments are schematically located on figures 7.1 through 7.4.

Figure 7.1, an isometric view taken from the southwest of the model, shows the location of load measurement and wall deformation instruments. The instrument numbers correspond to channel designation of automatic data-acquisition equipment. Instrument No. 90, a semiconductor strain-gage pressure transducer, recorded the pressure of the hydraulic system used in simulating column axial loads. Instrument No. 91 recorded the magnitude of horizontal loads. Initially this instrument was a load cell, but was subsequently replaced (after Test No. 5) by a pressure transducer. Instrument No. 91 was interchangeable in location, depending on the direction of horizontal forces. Instrument No. 92 was one of several secondary pressure transducers monitored during the tests to check horizontal force accuracy. Instrument No. 93, a pressure transducer, recorded the magnitude of uniformly distributed floor loads applied by air pressure.

Instruments No. M1 through M7 represent measurement devices employed to check load applications. These instruments were not connected to the automatic scanner, but were manually monitored. M1 and M7 represent pressure transducers located in the associated hydraulic system, while M2 through M6 were load cells attached to the jacking rams. For each test, the pertinent load instrument and deformation linear variable differential transducers (LVDTs) were also recorded by an automatic X-Y plotter.

The LVDTs in figure 7.1 recorded diagonal deformations of dry wall panels over the gage lengths shown. Gages No. 52, 54, 55, 56, and 57 designate LVDT's having readout intervals

of 0.0001 in, while the remaining LVDT, gage No. 53, had an interval of 0.00001 in.

Figure 7.2 illustrates the northwest view of model instrumentation. Diagonal deformations were recorded by LVDT's No. 50, 51, 58, 59, 60, and 61, all with a 0.0001-in. readout interval. Horizontal deflections of the test structure were measured by LVDT's No. 43, 44, 45, 46, and 47 with a readout interval of 0.0001 in.

Figure 7.3 is a plan view section showing vertical deflection transducers located under the second floor of the test structure. In addition, two transducers (No. 48 and 49) were positioned horizontally on the center main beam to record any differential movement relative to the ceiling slab. In general, the vertical transducer readout interval was 0.0001 in, excepting transducers located adjacent to columns read to the nearest 0.00001 in. Transducer readings were also checked by a dial gage deflectometer capable of reading 0.0001 in, which was read visually.

Figure 7.4 shows the location of 40 type A3 electrical resistance strain gages used to measure column concrete strains. The readout increment of these strain gages was 1 μin/in (i.e., 0.000001 in/in).

Calibration of load cells, pressure transducers, and deflection transducers was performed prior to testing of the structure.

Data-acquisition equipment included a 100-channel and a 50-channel automatic electronic scanner and digital recorder. Instrument readings were taken at predetermined load increments. The output data was subsequently key-punched and reduced by electronic computer.

Dial gages were also used to check against possible slip of the test structure floor slab relative to the laboratory floor slab. No such slip was observed.

8. Results

A total of 18 load tests were carried out on the laboratory structure. Of these, 17 involved extensive measurement and recording of loads and structural deformation. The remaining test was run simply as a proof test on column capacity.

Tests No. 1 through No. 11 were performed on the model of the total building system. Tests No. 12 through No. 18 (see section 6.3) were carried out on the system with wall panels removed.

Instrument locations are shown in figures 7.1 through 7.4. The instruments recorded loads and deformations for seventeen tests. Generally each instrument was read immediately after the attainment of the respective

increment of applied load. Reading and recording of data was in general accomplished through the use of an automatic data-acquisition system which recorded results in digital form on printed paper tape. Total acquisition time for each set of readings consisting of all data for one load increment was somewhat less than two minutes. The data was then manually key-punched onto cards, and was automatically reduced, analyzed, and plotted by electronic computer. Approximately 40,000 measurements were thus recorded.

Computer output consisted of a complete tabulation of results, and curves of measured deformations plotted against applied load. In all, more than 2000 curves were plotted. In addition to the data acquired by the automatic digital system, a continuous plot of critical deflection parameter versus applied load was maintained for all tests by an automatic X-Y plotter. This was used along with mechanical dial gages to provide a secondary and independent check on proper functioning of the automatic equipment.

After checking computer output for key-punching errors and malfunction of instrumentation, the results were reviewed to select the more significant information. The most pertinent results are presented and discussed in section 9; additional results are contained in Appendix A as figures. A.1 through A.77.

Each figure of Appendix A is a plot of applied load versus the model deformation as measured by the relevant instrument. The output channel number noted in the figure caption corresponds to the instrument number shown on figures 7.1 through 7.4.

The ordinate of each curve indicates the variable load. Load symbols are defined in section 3. The abscissa indicates deformation, where zero deformation is chosen prior to any load application. Thus in tests where an initial constant load is introduced, the abscissa indicates the deformation due to both the constant load and variable load.

All vertical deformations were measured relative to the structural test floor, thus beam deflection measurements include column shortening, and slab deflection measurements include support movement.

Column concrete strain data have been excluded due to the erratic behavior of these strain gages. Column gages were located 6 in from column ends. Their erratic behavior is attributed to the proximity of joint connections and to the relatively large quantity of steel used in connecting column end hardware to longitudinal reinforcement.

9. Interpretation of Results

9.1. Introduction

The purpose of this section is to discuss the compliance of the structural system with the performance criteria in section 4, the structural behavior under loads, and the interaction of structural components.

It should be noted that all conclusions pertaining to structural performance are based on the structure as built in the laboratory and on erection methods and materials used therein. Variation in materials or erection methods may significantly affect structural behavior.

Data pertinent to the discussion in this section are presented in figures 9.1 thru 9.23.

9.2. Vertical Forces

Vertical forces were applied to the structure in the form of column loads (P), distributed floor loads between the columns (w), and distributed floor loads on the cantilever section of the second floor along the north side of the structure (w'). (For location and magnitude of applied vertical loads, refer to fig. 6.1 and tables 6.1 and 6.2.)

Vertical loads were applied in all tests. In some of the tests they were applied along with horizontal loads in order to evaluate structural response to horizontal loads combined with vertical loads. Other tests were performed for the sole purpose of evaluating structural response to vertical loads. Details of all loading sequences have been discussed in section 6.

9.2.1. Structural Response to Vertical Loads

9.2.1.1. General

Figure 9.1[6] shows the load deflection history of the midspan of the center main beam under the application of a load of $1.3D + 1.7L$ to the columns and main floor section. This figure also shows the effect of sustaining this load for 24 hr and the subsequent recovery of deflections 24 hr after removal of all loads. Deflections at one of the supports of this beam due to the same loading are also shown to permit evaluation of the order of magnitude of the "net deflections" as well as the column deflection. Examination of all test data indicates that from the point of view of magnitude of vertical deflections, this curve illustrates the most critical point in the structure.

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[6] In this figure and several of the others used in this chapter, for the sake of clarity, individual data points and the data obtained during the several intermediate cycles of unloading and reloading have not been shown. However, these results are included on figures A1 through A.77 of Appendix A.

The following observations can be made concerning midspan deflection of the center main beam under the application of a load of $1.3D + 1.7L$ and its subsequent maintenance for 24 hr (fig. 9.1):

(1) The increasing load-deformation curve for the load application portion of the cycle was reasonably linear, indicating elastic behavior;

(2) The 24 hr creep amounted to less than 0.02 in, which is only 7 percent of the permissible deflection set forth as a performance criterion and about 13 percent of the total observed deflection;

(3) Observed recovery was 96 percent (note that most of the creep deflection was recovered).

Figure 9.2 shows the plot of midspan deflection of the center main beam during the application of a 370 psf load $(1D + 8.4L)$ to the main floor span after removal of the walls from the test structure (Test 16). This load was applied after the application of $1D + 1L$ to the columns. This test was designed to be a destructive test of the floor system of the structure; however, the capacity of the loading system (designed for 300 psf) was reached before failure of the test structure. The deflection at one of the beam supports is again plotted on this figure to illustrate the order of magnitude of the "net" deflections. Also shown is the curve for the center beam midspan deflection obtained in Test 9 before the removal of walls. In Test 9 the maximum applied floor load was 160 psf $(1D + 3.5L)$. It is interesting to note the substantial reduction in stiffness against gross vertical deflection resulting from the removal of the walls from the system.

Two definite slope changes are evident in the curve for the midspan deflection of the structure without walls, one at 120 psf and one at 270 psf. A change in slope similar to that taking place at 120 psf is not evident in the curve for the structure with walls. It is felt that this change was probably due to some slippage at the beam column connection and was apparently of minor consequence in terms of structural performance.

The break which is evidenced at 270 psf is more marked. At this load, diagonal tension cracks were observed close to the beam supports (fig. 9.3). Since there are stirrups in the beam (figs. 5.8 and 5.9) and since the curve shows that the structure was capable of carrying substantial additional load, this point may represent a transfer of shear stresses to the stirrups. The structure was subsequently loaded to 370 psf $(1D + 8.4L)$ without additional signs of distress.

Both figs. 9.1 and 9.2 illustrate the case of interior span loading (load "w" acting alone), since this appeared to be the more critical

loading configuration. The relative influence of these two loading patterns is illustrated by figure 9.4, which shows center main-beam midspan deflection for Test 9 with interior loading (w) alone, and Test 9A with interior and cantilever loading ($w + w'$). As would be expected, the "w" loading is more critical in terms of deflection; however, only slightly so.

9.2.1.2. Influence of Walls

The influence of the walls on structural response to vertical loads is illustrated in figures 9.2 and 9.5. Figure 9.2 compares deflections at midspan of the center main beam in Test 9 with walls, and Test 16 with walls removed. These tests had identical loading and the comparison is probably valid, although the structure may have been weakened somewhat before Test 16 by earlier tests. The location at which deflections are compared in this figure reflects the behavior of the entire structure, since most members will make some contribution to the midspan deflection. These figures indicate that the structure with the walls removed had about twice the deflection of the complete structure.

Figure 9.5 compares deflections with and without walls at the position which is likely to be most sensitive to walls; namely, the center of the west main beam which rests on a fire wall. As expected, the influence of the walls is even more marked in this case. The deflection at maximum load without walls is approximately 5 times the deflection with walls. It is thus evident that the walls contribute significantly to the support of vertical loads.

9.2.1.3. Slip Between Main Beams and Topping

Devices which were capable of measuring the slip between the center main beam and the channel slabs were monitored during all tests. These were installed as a means of measuring any differential shear movement between the topping slab (which forms the compression flange of the main beams) and the precast element which forms the tension flange. In none of the tests was there any indication of relative slip between these two components. Neither was there any visual sign of relative slip, even in Test 16 (fig. 9.2) with a load of $1D + 8.4L$.

9.2.1.4. Translation Due to Vertical Loads (Walls Removed)

This aspect of the structural response of the frame was investigated in Tests 12, 12A, 13, and 13A, in which the floor was alternately loaded over its main span alone (w) and its main span plus the cantilever span ($w + w'$), with rollers oriented first to roll in the north-south direction and then in the east-west direction. The results of these tests are shown in figures 9.6 and 9.7. In each case the order of magnitude of lateral displacement under a load of $1.3D + 1.7L$ on the columns and floor was between 0.06 in and 0.08 in and residual displacements were of the order of 0.01 in.

9.2.2. Compliance With Performance Criteria, Vertical Loads

9.2.2.1. Performance Criterion 4.4.3, Vertical Deflections Under Service Live Load

At a load level of $1D + 1L$ the vertical deflections due to the superimposed load of $1L$ shall not exceed the following:

$$dv \leq \frac{l}{480} = \frac{144 \text{ in}}{480} = 0.30 \text{ in.}$$

where: dv = vertical net deflection.

Under vertical loading of $1D + 1L$, the most critical vertical deflection in the test structure occurred at the midspan of the center main beam. Figure 9.4 illustrates test No. 9 plotting total vertical deflection at midspan of the center main beam together with total vertical deflection at one of the column supports of the same beam. The vertical net deflection will be the difference between the midspan deflection and the deflection of the beam support. Figure 9.4 illustrates that at the level of $1D + 1L$ the critical vertical net deflection was 0.04 in, which is considerably less than the permitted 0.30 in net deflection.

Criterion 4.4.3 was therefore satisfied.

9.2.2.2. Performance Criterion 4.4.4, Sustained Load Deflections

At a load level of $1.3D + 1.7L$, sustained for 24 hours, deflections due to the superimposed load of $0.3D + 1.7L$ shall not exceed the following:

(a) $dv \leq \dfrac{l}{360} \times \dfrac{0.3D + 1.7L}{L} =$

$\dfrac{144 \text{ in}}{360} \times \dfrac{100 \text{ lb}^{\dagger}}{43 \text{ lb}} = 0.93 \text{ in}$

(b) $Dh \leq 0.002h = 0.002 \times 94 \text{ in} = 0.19 \text{ in}$

Residual deflections, measured within 24 hours after removal of loads, shall not exceed the following:

(c) If $dv > \dfrac{l^2}{20{,}000t}$, $dvr \leq 0.25\, dv$;

if $dv \leq \dfrac{l^2}{20{,}000t}$, $dvr \leq \dfrac{l^2}{80{,}000t} =$

$\dfrac{144 \text{ in}}{80{,}000 \times 9.5} = 0.03 \text{ in}$

(d) $Dvr \leq dvr + 0.25\,(Dv - dv)$

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where:

Dv = vertical gross deflection
Dvr = residual vertical gross deflection
dv = vertical net deflection
dvr = residual vertical net deflection
l = length of member
t = depth of member

(a) Under the vertical loading of $1.3D + 1.7L$ sustained for 24 hr the midspan of the center main beam exhibited the largest net vertical deflection (dv). This deflection was less than 0.10 in (see fig. 9.1). The net deflection (dv) should be taken as the total deflection (Dv) less the support deflection. Unfortunately, during the sustained-load portion of this test, the instruments measuring the support deflection malfunctioned and there is thus no complete record of the beam support deflection. Thus, only the short-term portion of this deflection has been subtracted to obtain dv. The maximum measured Dv, which was 0.14 in, is considerably less than the 0.93 in allowed by Criterion 4.4.4(a).

(b) The horizontal deflections, which were measured under vertical loads acting alone both with and without walls, were extremely small. In all cases they were less than 0.08 in under $1.3D + 1.7L$ on the floor and columns. (See figs. 9.6 and 9.7.) This is considerably less than the 0.19 in permitted by Criterion 4.4.4(b).

(c) Figure 9.1 shows the residual deflection Dvr to be approximately 0.005 in, which is considerably less than the residual deflection permitted by Criteria 4.4.4.(c) and 4.4.4(d), which is 0.03 in.

Criterion 4.4.4 was therefore satisfied.

9.2.2.3. Performance Criterion 4.4.5, Ultimate Strength

The structure or any portion thereof shall not fail at a load smaller than the following:

(a) $1.25 (1.5D + 1.8L) = w = 145$ psf.

The structure was capable of carrying a load of 370 psf without experiencing failure (fig. 9.2).

Criterion 4.4.5 (a) was therefore satisfied.

9.3. Horizontal Forces

Horizontal forces were applied to the structure in the form of the horizontal loads Hw, Hs and Hs' (see fig. 6.1 and tables 6.1 and 6.2). Racking tests were conducted in the north and the east direction with and without walls. The results of these tests are described and evaluated in the following sections.

9.3.1. Horizontal Loads in the North Direction

In the north direction racking of the structure is resisted by the firewalls.

9.3.1.1. Racking Tests With Minimum Vertical Loads

These racking tests were conducted with a superimposed column load of $0.9D$ acting alone. The results of the racking test in the north direction are illustrated in figure 9.8. This figure shows lateral deflection measured at the level of the second floor of the test structure. Loads were applied to simulate a wind pressure of 25 psf acting from the south. It may be noted from this figure that while overall deflection was small (0.091 in), recovery was also small. Figure 9.9 shows the results of a later racking test which was carried to an equivalent of 60 psf wind load. These two tests are simultaneously plotted in figure 9.10[*] and show good agreement.

Figure 9.11 shows a plot of south wind load versus diagonal compressive deformation measured on one of the fire walls. The diagonal shown in this figure was measured over a gage length of 147 in. The resultant unit strain at a wind load of 25 psf is 0.000073 in/in and at a wind load of 60 psf it is 0.000250 in/in, which is extremely small. It is interesting to note from this figure that the recovery of the walls was good for all levels of load. No signs of distress were observed in the walls or other parts of the structure during Test 2 in which a wind load equivalent to 25 psf was applied. However, at the upper limit of Test 10 at a wind load equivalent to 60 psf, some distress appeared in the form of bowing out (buckling) in compression areas near the corners of the wall panels. These signs of distress disappeared upon removal of the lateral load. After removal of the walls all connections between the walls and the frame were found to be in good condition, showing no dislocation of screws or anchorage devices. During Test 10 there was some opening up of the joints between the columns and the wall panels in regions which would normally be subjected to tension by the development of diaphragm action in the walls. These openings were all less than $\frac{1}{8}$ inch in width and tended to close partially upon removal of the load.

9.3.1.2. Racking Tests at High Vertical Loads

In Test 7 the columns and floor were loaded to $1.3D + 1.7L$ and a 15 psf ($0.8H$) south wind load was applied to the structure. The results

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[*] Only a portion of the test to 60 psf is shown here, since it exceeds the scale of this figure.

of this test are illustrated in figure 9.12. This test is also plotted in figure 9.13 along with Test 2 which had $0.9D$ and 25 psf wind load. The agreement between these two tests is good. The structure experienced a considerably larger lateral drift under the application of the larger vertical load acting alone than it did under the smaller vertical load (0.024 in versus 0.007 in). Under the subsequent wind-load application the structure with the larger vertical load exhibited greater stiffness than it did when more lightly loaded. At the point where the 15 psf wind load was reached, the two deflections were approximately equal (0.050 in).

9.3.1.3. Frame Action Versus Wall Action

The frame was racked after removal of the walls in Test No. 14. Results are illustrated in figure 9.14. This test is compared with an identical racking test performed before removal of the walls in figure 9.10. This comparison clearly indicates that a major portion of the lateral stiffness is provided by the walls rather than by the frame.

Figure 9.15 shows the results obtained in a later racking test (Test 18) on the structure with walls removed. This test was carried to the point where the frame no longer developed increasing resistance to load. In this test the structure had a vertical load on the columns of $0.9D$. In interpreting these results it must be remembered that the wind load reported here is in pounds per square foot of total vertical surface area of the structure. If none of the walls are present, then the surface area upon which the wind forces act is also not present. However, it is also conceivable that a situation could develop in which walls in one direction are present while walls in the opposite direction are absent. In such a case these results would have relevance. Figure 9.15 indicates that the frame acting alone in the north direction cannot be expected to withstand a wind force in excess of 10 psf on the gross area of the structure. By the time this test was performed the structure had been carried through a number of earlier tests which might have somewhat weakened the frame. However, the structure at this point exhibited no obvious signs of distress attributable to earlier testing. It is recognized that the simulation of the column foundation which was used in the test structure was extremely conservative compared to that used in the real structure, particularly with respect to tests without walls. Thus the results of this test possibly fall well below the results which would be obtained from the test of a real structure.

9.3.2. Horizontal Loads in the East Direction

West wind forces in the test structure were resisted by one interior "structicore" wall and one exterior wall. These walls would not normally be expected to be as strong as the fire walls; however, their rigidity in the lateral load tests appeared to be comparable to that of the fire walls. Both of the walls in this direction had openings; however, these walls also had a greater overall length resisting load.

9.3.2.1. Racking Tests With Minimum Vertical Loads

These racking tests were conducted in the same manner as in the south direction (sec. 9.3.1.1). Figure 9.16 illustrates Test 3, which subjected the structure to $0.9D$ plus 25 psf wind load from the west. In this test, measured deformations were extremely small (0.012 in). Recovery characteristics were similar to those observed for the fire walls. There appear to be two breaks in the load-deflection curve (fig. 9.16), one at 10 psf and the other at 24 psf. Neither of these was accompanied by any visual signs of distress in either the concrete frame or in the gypsum walls. Figure 9.17 shows the results obtained from a racking test (Test 11) carried to a wind load in excess of 70 psf. A portion of this test, along with the results of Test 3, is shown in figure 9.18. It is interesting to note that a definite break developed in the load-deflection curve of Test 11 at 6 psf; again, this break was not associated with any visual signs of distress. These breaks in the load deflection curve are not considered to be particularly significant since, for example in Test 11, even at a wind load of 25 psf the lateral drift of the structure is still less than 0.04 in. Figure 9.19 [9] shows a plot of load versus wall diagonal compressive deformation for a wall resisting west wind load during Test 11, which was carried to 74 psf. As was the case for the walls resisting south wind load (fig. 9.10) deformations and correspondingly average strains were extremely small and recovery was good. In this test, distress in the wall was not noted until the very upper range of the loading sequence was reached. At these loads distress was observed in the interior (north) wall in the form of shear cracks (fig. 9.20). However, no noticeable distress was observed in the wall-frame connections. Some progressive opening of the joint between the wall panels and the columns was observed at loads in excess of 35 psf but the separation was not particularly pronounced and was similar to that discussed in section 9.3.1.1.

[9] The erratic behavior noted in the first unload-reload cycle was due to a stuck instrument which was subsequently freed.

9.3.2.2. Racking Test at High Vertical Loads

A racking test was performed in the east direction in Test 8 with $1.3D + 1.7L$ and 15 psf wind load. The results of this test are illustrated in figure 9.21. It should be noted that the application of the vertical loads caused a horizontal deflection in the opposite direction to that in which the wind loads were subsequently applied. This deflection, resulting from the vertical loads, was only partially reversed by the application of the 15 psf wind load. Test 8 ($1.3D + 1.7L$) is plotted along with Test 3 ($0.9D$) in figure 9.22. It can be seen that in either case the deformations due to lateral loads are so small as to make questionable any conclusions concerning the effect of the magnitude of vertical load on lateral stiffness.

9.3.2.3. Frame Action Versus Wall Action

Figure 9.16 (Test 3) shows the response of the structure with walls to racking in the east direction, while figure 9.23 (Test 15) shows the response of the structure after removal of the walls. Both of these curves are plotted together in figure 9.18 [10]. As was the case in the north direction, it is evident that in the east direction the walls provide most of the stiffness against lateral loads.

In designing the test sequences to which the structure was subjected, it was felt that only one meaningful racking test could be carried through to the point at which the structure was approaching collapse. The north direction, which is the narrow direction for this system, was chosen for this test and its results were reported earlier (Test 18) in section 9.3.1.3. In the east direction the maximum load to which the frame without walls was subjected was applied in Test 15, which is shown in figure 9.23. In this test the frame resisted a wind load in excess of 15 psf without collapse, although some minor flexural cracks were observed in the columns at maximum load. This load level was substantially higher than that sustained in the racking test without walls in the north direction. The conservative nature of the foundation simulation in the test structure, which provided a hinge at the lower column connection, undoubtedly affected the results obtained in this test in an adverse manner.

9.3.3. Compliance With Performance Criteria, Horizontal Loads

9.3.3.1. Performance Criterion 4.4.1, Horizontal Deflection Under Dead and Wind Loads

At a load level of $0.9D + 1.1H$ the horizontal deflection due to the superimposed load of $1.1H$ shall not exceed the following:
$$Dh \leq 0.002h = 0.002 \times 94 \text{ in} = 0.19 \text{ in}$$

where:

Dh = horizontal gross deflection
h = height above grade.

In Test 2 (fig. 9.8) under $0.9D$ and a wind load of 22 psf ($1.1H$) from the south, the maximum lateral drift was approximately 0.073 in, and in Test 3 (fig. 9.16) with a west wind load the maximum lateral drift was approximately 0.007 in, while the allowable drift under this criterion is 0.19 in.

Criterion 4.4.1 is therefore satisfied.

9.3.3.2. Performance Criterion 4.4.3, Horizontal Deflection Under Dead, Live, and Wind Load

At a load level of $1.3D + 1.7L + 0.8H$ the horizontal deflection due to the superimposed load of $0.3D + 1.7L + 0.8H$ shall not exceed the following:

$$Dh \leq 0.002h = 0.19 \text{ in}$$

In Test 7 (fig. 9.24) under $1.3D + 1.7L$ and a wind load of 15 psf ($0.8H$) from the south, the maximum lateral drift was 0.045 in and in Test 8 (fig. 9.20) with a west wind load the maximum lateral drift was 0.032 in[11], while the allowable drift under this criterion is 0.19 in.

Criterion 4.4.2 is therefore satisfied.

9.3.3.3. Performance Criterion 4.4.5(b), Ultimate Strength

The structure or any portion thereof shall not fail at a load smaller than the following:

(b) $0.9D + 1.4H$, (28 psf wind load).

The structure was tested under these loading conditions in the north direction and in the east direction in Tests 10 and 11 respectively (see figs. 9.9 and 9.17). No distress was experienced in either test at that load level.

Criterion 4.4.4.(b) is therefore satisfied.

9.4. Summary

(1) All conclusions pertaining to the structural performance of the system in question are based on the structure as built in the laboratory and on the erection methods and materials used therein. Variations in materials and erection methods may influence performance.

(2) The building system satisfied the performance criteria which were set for its evaluation with substantial margin. As a system, it exhibited strength and stiffness in excess of service and ultimate load requirements.

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[10] Only a portion of Test 15 is shown on this figure because of the scale.

[11] In Test 8 the maximum drift was measured upon the application of the vertical load and took place in the opposite direction to the wind-induced deflection. If these two had been in the same direction rather than opposite, the maximum deflection would have been approximately 0.04 in.

(3) The walls of the system behaved as an integral part of the structure. They provided most of the stiffness of the system with respect to lateral loads, and provided a significant portion of the stiffness against vertical loads.

(4) The building system with its walls removed had considerable reserve strength above the required vertical load bearing capacity; however, without the aid of its walls it was not capable of resisting the required service wind loads.

10. Component Tests

10.1. Introduction

Load tests were conducted on the three principal precast load-bearing components of the structure. The components tested were columns, main beams with an appropriate portion of the topping slab connected to them, and a floor-channel slab. These tests were performed to determine the behavior and ultimate strength of the components. Included were tests to determine the effects of creep on the columns and repeated loading on the beams.

10.2. Column Tests

The column specimens tested were typical "short"[12] columns as shown in figure 5.6. Reinforcement was 60 ksi steel. Actual outside dimensions and concrete cover of individual test specimens are shown in table 10.1. Values of concrete strength of the various specimens are reported in section 10.2.1.1. The tests consisted of the following:

(1) Short-term destructive loads were applied parallel to the column axis. Four columns were tested with an eccentric load on the major axis and three with an eccentric load on the minor axis.

(2) Two sustained-load tests were carried out: one with an eccentric load on the major axis, the other with an eccentric load on the minor axis.

The method of applying the eccentric loads to the columns is shown schematically in figures 10.1 and 10.2. The same method was used for both the short-term and the sustained-load tests.

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[12] The term "short" column is applied in the plans to a column with end fixtures at both ends. This column in the structure has the same slenderness ratio as all other columns.

10.2.1. Short-Term Destructive Tests

10.2.1.1. Specimens

Seven columns were tested to destruction, three with a load eccentricity of 0.5 in on the minor axis ($e/t = 0.1$ for columns 1, 5, and 8), three with a load eccentricity of 2.0 in on the major axis ($e/t = 0.33$ for columns 2, 6, and 7), and one with a load eccentricity of 1.5 in on the major axis ($e/t = 0.25$ for column 9).

Columns 1 and 2 were cast at the same time as the test structure components (April 16 and 17, 1968) and from the same concrete, with concrete compressive strength ($f'c$) ranging from 4900 psi to 7200 psi (see table 11.1). These specimens were approximately 20 days old when tested. Columns 5, 6, 7, 8, and 9 were cast from similar concrete at a later date (May 15). Columns 5, 6, 7, and 8 were approximately 30 days old when tested and the concrete had a compressive strength of approximately 5400 psi. Column 9 was 60 days old when tested and the concrete compressive strength was 7000 psi.

The longitudinal reinforcing bars (No. 6 deformed bars) were approximately 3 in shorter than their full required length in Columns 1 and 2, leaving a distance of about 1½ in between reinforcing bars and end fixtures, but in columns 5, 6, 7, 8, and 9 these bars were only ¼ in shorter.

10.2.1.2. Loading

The loads were applied continuously until failure, through a knife-edge loading plate (figs. 10.1 and 10.2) by a 600,000-lb hydraulic testing machine at a rate of 8,000 lb per minute. Deflections were measured with long-throw mechanical dial gages at mid-height. Figure 10.3 illustrates a typical test setup.

10.2.1.3. Results

Test results are shown in figures 10.4 through 10.10 as load-deflection curves. Ultimate loads are tabulated in table 10.2. The average maximum load for columns 1, 5, and 8 (minor axis bending, $e = 0.5$ in) was 78.9 kips. Figure 10.11 shows these columns after testing. Column 1 failed near its end connection, and its mode of failure appeared to be partially due to the short reinforcement used. Columns 5 and 8 (which had longer reinforcement) experienced compression failures in the concrete at mid-height at about 12 percent higher loads than did column 1.

The average maximum load for columns 2, 6, and 7 (major axis bending, $e = 2.0$ in) was 51.2 kip. All three specimens failed in a similar manner, as illustrated in figure 10.12, with excessive bending of the channel-shaped,

top-fixture and some spalling of the concrete near this fixture.

The maximum load for column 9 (major axis bending, $e = 1.5$ in) was 89.5 kips. This column failed by concrete compression at mid-height.

10.2.2. Sustained Loading (Creep) Tests

10.2.2.1. Specimens

Two columns (columns 3 and 4) were tested under a 25 kip sustained load. Both columns were cast with the test-structure components and were about 20 days old when placed under load. Concrete compressive strength ranged from 4900 psi to 7200 psi (see table 11.1).

10.2.1.2. Loading

The loading frames used in these tests are shown in figures 10.13 and 10.14. Column 3 had a load eccentricity of 0.5 in on the minor axis ($e/t = 0.1$), and column had a load eccentricity of 2.0 in ($e/t = 0.33$) on the major axis. A detail of the bottom of the column 4 loading frame is shown as figure 10.15. This figure also shows the heavy spring used to sustain the load on the specimen.

The 25 kip load ($1D + 1L$) was applied by means of a 30-ton hydraulic ram and a load cell inserted between the top two plates of the test frame. The ram load was applied through the column to the spring, causing the spring to compress. Once the required load was applied, nuts on the $\frac{3}{4}$-in tie-bars were tightened against the top knife-edge plate. The deflection of the springs was about $1\frac{1}{2}$ in at the 25 kip load. The loads were checked and adjusted periodically.

Mid-height deflections were measured by means of a taut wire and a mirrored scale. The progression of the deflections with time was measured periodically.

10.2.2.3. Results

Results of a 170-day observation period are presented in figures 10.16 and 10.17 as time-deflection curves. The initial deflections are included in the total deflection for information and comparison purposes.

10.2.3. Interpretation of Column Test Results

10.2.3.1. Short-Term Destructive Tests

Figure 10.18 shows a plot of test results for columns with major axis load eccentricity, together with computed interaction curves for both the column cross section and the overall column slenderness effects. This figure also shows interaction curves derived from performance criteria for lower-story columns.

Curve C is the locus of extreme values of combined axial loads and end moments which the columns must be able to resist in the direction of their major axis. Critical loading conditions for columns were found to be $1.5D + 1.8L$ and $1.25(D + L + H)$. The actual points plotted for these component requirements correspond to $1/0.8$ times the critical loading, where 0.8 represents an understrength factor. The requirement here is similar to that explained in the commentary to section 4.4.5 of this report; namely, that in the absence of a laboratory sample of large size, individual columns are required to exhibit a strength in excess of their ultimate loading requirement. The ultimate moment imposed on the column by wind load was assumed to be $1/7$ of the computed ultimate moment imposed on the frame in the absence of walls. This assumption is based on the test results illustrated in figure 9.10, which compares horizontal wind deflections of the total system to these of the system with walls removed. Curve A of figure 10.18 is a theoretical interaction diagram for cross-sectional capacity, computed for the combinations of vertical load and moment which would cause failure in columns with a concrete compressive strength of 7,000 psi, which was the concrete compressive strength of column 9. Curve B is a similar interaction curve for the specified concrete compressive strength of 4,500 psi.

The total maximum moment acting on a column is the end moment plus an additional moment which equals the product of the applied vertical load times the maximum deflection of the column. To determine the combination of maximum axial load and maximum end moment that can be imposed on a column at its supports, the maximum column moment in interaction curves A and B, which represent the total cross-sectional capacity of the columns, must be reduced by the value of $P \times dh$, which is the product of axial load and maximum column net deflection at column failure. Curve B' has been plotted to account for this moment reduction at the specified concrete compressive strength of 4,500 psi.[13] Curve B' is therefore the interaction curve of ultimate loads and ultimate end moments which a column, constructed in accordance with the plans and specifications for this system, should be theoretically expected to resist. It may be noted by comparing Curves B' and C' that the theoretical column capacity exceeds the required critical loading by a considerable margin.

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[13] Values of $P \times dh$ were computed in accordance with "Proposals for Revision to Sections 915 and 916 of ACI 318–63" by MacGregor, Breen, and Pfrang (unpublished). These computations accounted for concrete cracking.

The actual column tests for loads with major axis eccentricity are also plotted in figure 10.18. It will be noted that each specimen test is plotted twice. The triangular points represent a plot of the axial load at failure against the end moment caused by the axial load times its eccentricity.

The square points represent a plot of the axial load at failure against the maximum moment that actually existed in the specimen at failure. This actual maximum moment is the product of the axial load times the sum of its end eccentricity and the maximum center line deflection at failure. It should be noted that in a slender column such as the specimens tested the maximum center line deflection is relatively large (Refer to figures 10.4 through 10.10).

Only column 9 failed by compression at mid-height. Column 9 had a concrete strength of 7,000 psi, and it can be seen that this column developed strength slightly in excess of the strength predicted by interaction curve A. Columns 2, 6, and 7 failed at their top fixture and therefore did not develop their theoretical ultimate strength. In column 2 the reinforcement was 3 in short. This was corrected in columns 6, 7, and 9, without appreciable effect on columns 6 and 7. To evaluate column strength in terms of component requirements the triangular plots of the test results should be compared with curve C. It may be noted that all the tested columns had considerable excess strength over the component requirements.

Results for columns tested with minor axis eccentricity are plotted in figure 10.19. Interaction curves A, B, and B' are plotted as in figure 10.18, except that in this figure they represent relationships for loads with minor axis eccentricity, and curve A was computed for a concrete compressive strength of 5,400 psi. Columns 5 and 8 experienced a compression failure at mid-height, and show strengths close to the theoretical strength predicted by interaction curve A, which was computed for 5,400 psi concrete (the actual strength of the concrete in these columns). Column 1 failed near its top fixture, and this mode of failure may have been caused by the fact that the reinforcement was 3 in short. In the case of minor axis eccentricity, no sizable end moments are expected to act on the columns, since the tie beams do not participate in the support of vertical loads to an appreciable extent. All columns tested at a minor axis eccentricity of 0.5 in were able to sustain vertical loads in excess of the 51 kip needed to satisfy the component requirements.

In summary, all the tested columns were able to withstand axial loads and moments in excess of required performance. Some of the columns did not develop their full theoretical ultimate strength because of weakness at the end fixture.

10.2.3.2. Sustained Loading (Creep) Tests

Figures 10.16 and 10.17 show the results of creep tests conducted on columns 3 and 4.

Column 4 was loaded with an axial load of 25 kip at an eccentricity of 2 in on its major axis. The test results are illustrated in figure 10.16. This figure also shows a computed value of instantaneous deflection. It should be noted that the vertical load is applied outside the kern of the section. The instantaneous deflection was therefore computed on the basis of a cracked section neglecting concrete tension. This will tend to overestimate the computed deflection, since not every section along the length of the column is cracked. In this case the computed instantaneous deflection is 0.32 in while the measured instantaneous deflection was only 0.22 in. However this measured value is low compared with values measured in tests on columns 2, 6, and 7 which were subjected to similar loading conditions. For the latter three tests the instantaneous deflection at a 25 kip load averaged 0.29 in (see figures 10.7, 10.8, and 10.9).

Figure 10.16 also shows an upper deflection limit for the given conditions, obtained by considering only the steel reinforcement and neglecting the concrete. This limit was computed by assuming that all the load is carried by the reinforcement. The deflection limit thus computed under the conditions of this test is 0.54 in and the steel stress at this deflection limit would be 38.2 ksi, which is well below the specified 60 ksi yield stress of the column reinforcement. Creep buckling under the conditions of this test therefore cannot occur. All computed deflections referred to in this section accounted for an added moment equal to the axial load multiplied by the deflection at each point along the column.

Column 3 was loaded with an axial load of 25 kip at an eccentricity of 0.5 in on the minor axis. This test is illustrated in figure 10.17, together with the computed instantaneous deflection of 0.106 in and the computed deflection limit assuming that no stress is carried by the concrete, which is 0.565 in. In this case the computed instantaneous deflection is based on an uncracked section and is in good agreement with the measured instantaneous deflection of 0.12 in as well as with instantaneous deflections measured in the tests of columns 1, 5, and 8 (figures 10.4, 10.5, and 10.6). The deflection limit under these loading conditions is 0.565 in and the computed steel stress at the deflection limit is 21 ksi, which precludes the possibility of creep buckling under the conditions of this test.

The creep tests on columns 4 and 3 respectively are also plotted in figures 10.20 and 10.21. Curves D in these figures are the interaction curves for maximum axial load and maximum

total moment at which the steel carries the entire load without concrete participation. Curves D' show the same interaction curves for the reduced moments when deflections are taken into account. Thus the curves marked D' represent the combination of axial loads and applied end moments which can be supported by the column reinforcement without concrete participation as a limit condition. It is significant to note that curve D' in figure 10.20 when compared with curve C indicates that creep buckling can not occur in this structure even under the assumed ultimate loading conditions. The creep tests of columns 3 and 5 simulate sustained loading of 1 live + 1 dead load in the structure. It can be seen from the plot of these tests in figures 10.20 and 10.21 that there is a considerable margin of safety against creep buckling in the direction of both the major and the minor column axes.

10.3. Channel Slab Test

One of the channel slabs was picked at random and tested to destruction under centerpoint loading.

The slab was supported at each end and loaded through a 4-in-wide loading beam at midspan. Deflection of the slab was measured at midspan by two 2-in-throw mechanical dial gages.

The test results are shown in figure 10.22. The load at the yield point (2.5 kip) was higher than the predicted load at the yield point of the reinforcement (2.25 kip) using the nominal specified reinforcement yield strength, (40 ksi) and a 4.500 psi concrete strength. The tests on the laboratory structure also indicated satisfactory performance of these components. No material specimens were tested to determine the actual steel and concrete strengths of the floor channels.

10.4. Beam Tests—Repeated Loading

10.4.1. Test Specimens

The test specimens were typical main beam components with a 22-in-wide and 2-in-thick topping slab cast on each beam. Results are presented for seven beams. Preliminary tests on three other beams are not reported because the test conditions (quarter-point loading) were found to be far too severe in relationship to service conditions.

Beams Nos. 6 through 11 were prepared with column stubs passing through the topping slab and with column connection fixtures in place simulating conditions in the structure except that tie beams were not attached and grouted to these connections (a block of wood was used as a spacer to fill the void caused by omission of the tie beams). The connections were not grouted. Beam No. 5 did not have the column stub or column fixtures.

In all of the beams tested, the top surface had not been roughened as required by the plans and specifications of the Neal Mitchell Housing System. All specimen preparation, including the placing of the topping slab, was performed by Neal Mitchell Associates. The top surfaces of these beams had been cast against steel forms and were very smooth.

Two types of shear connectors were used in the seven beams. These shear connectors are illustrated in figure 10.23. Beams Nos. 5, 7, and 9 used Star[14] inserts spaced 19 in. on centers similar to the shear connectors used in the test structure. Beams Nos. 8 and 10 used Richmond (Kohler)[14] inserts spaced 19 in. on centers. Beams Nos. 6 and 11 used the Richmond[14] inserts spaced 9½ in. on centers. All data on shear connector type and spacing in individual specimens are summarized in table 10.3.

10.4.2. Beam Loading

Figure 10.24 is a general view of the test setup. Two 10 kip servo-controlled hydraulic rams applied the load by reacting against a frame bolted to the laboratory tie-down floor. Loading beams under the two rams distributed the test loads. All beams were tested by applying the loads in accordance with the sketch shown in Figure 10.25 and where simply supported by rollers on a clear span of 12.5 ft.

The beams were subjected to 1000 cycles of stress, alternating between intensities corresponding to $1D$ and $1D + 1L$, (for each ram $1D = 2.5$ kip, and $1L = 2.5$ kip; see Appendix C). Subsequently, the beams were tested to failure by 1000-cycle increments with the upper load level being increased at each increment by $0.5L$ (1.25 kip).

The rate of cyclic loading was 1 cycle per second except for a few cycles at the beginning and end of each increment. During these periods when the rate was 0.01 cycle per second, mechanical dial gage readings were made. During these few cycles, center-span deflection measurements were made using a 5-in-throw mechanical dial gage. In addition, continuous center-span deflection measurements were recorded on a strip chart recorder by using a 3-in linear variable differential transducer (LVDT). Both measuring systems can be seen in figure

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[14] Proprietary commercial product.

10.24. In an effort to measure the relative horizontal slip between the beam and its topping slab, 0.001-in dial gages were mounted on the beam ends. One of these gages can also be seen in figure 10.24.

10.4.3. Test Results

Graphs reproduced from strip-chart recordings of the midspan deflections are presented as figures 10.25 through 10.28. Table 10.4 shows the midspan deflections measured at the beginning of each increment of loading for each of the beams. This table also indicates the point at which noticeable slippage between the topping slab and the beam occurred, as measured by the slip dial gages installed at the ends of the beam. After testing, the topping slab was removed from each beam, and spacing and condition of anchorage inserts were determined. The data relative to these tests are presented in table 10.5.

10.4.4. Interpretation of Results

A study of table 10.5 and figures 10.25 through 10.28 indicates that all specimens tested showed a similar pattern of failure First a slip occurred between the precast beam and the topping slab. After this initial slip the beams no longer acted monolithically with the slab, and as a consequence the deflections caused by applied load increased. Deflections also increased moderately with the number of load cycles applied during the application of $1D + 1L$, $1D + 1.5L$ and $1D + 2L$. During the repeated application of the load of $1D + 2.5L$ deflections of all specimens tested increased rapidly and some of the specimens failed. All the remaining specimens failed during the first few cycles of application of the load of $1D + 3L$.

The initial slip that occurs between the precast beam and the topping slab is caused by horizontal shear. In the structure this shear is resisted by the shear connectors (inserts), the column-beam connection (the column base plate and part of the upper story column bear against the topping slab), and friction between the precast beam, the topping slab, and the floor channels. In the separate components that were tested not all these elements were present. Shear resisting devices were varied in the tests to determine their effectiveness in preventing slippage.

Beam 5, which had no column stub or column-beam connection fixtures and had Star inserts at 19 inches on center, experienced slip between the beam and the topping slab during the first cycle of application of the $1D + 1L$ load (figure 10.25). In terms of ultimate

strength it performed considerably better, resisting 1000 cycles of $1D + 2.5L$ without failure.

The results of the test on Beam 9 are shown in figure 10.26. This beam, which was similar to Beam 5 except that it did have a partial column-beam connection, performed approximately equal with Beam 5. Beam 9 exhibited signs of first slip during the first cycle of loading to $1D + 1L$ and failed at the 820th cycle of $1D + 2.5L$, while Beam 5 failed during the first cycle of $1D + 3L$.

In comparison, Beam 7, the companion specimen to Beam 9, performed considerably better than either Beam 5 or Beam 9. Beam 7 was able to sustain 1000 cycles of $1D + 1L$ without any signs of slip. First indications of slip for this beam were observed after the first few cycles of $1D + 1.5L$. Failure occurred at approximately the same point as that of Beam 5.

The results of the test on Beam 8 are shown in figure 10.27. Note that this beam, which had partial column-beam connections and Richmond inserts at 19 inches on center, had about the same initial slip behavior as did Beam 7, which was similar except for type of insert. Beam 8 experienced ultimate failure somewhat earlier in the loading sequence than did Beam 7. Beam 10 (the companion to Beam 8) slipped at about the same point in the loading sequence as did Beam 8, but its ultimate failure took place during the first cycle of $1D + 3L$.

The results of the test of Beam 6 are shown in figure 10.28. Beam 6 had a partial column-beam connection and Richmond inserts spaced at $9\frac{1}{2}$ in on center. This beam was able to sustain the full 1000 cycles of loading from $1D$ to $1D + 1L$ without slippage and went on to sustain about 500 cycles of loading from $1D$ to $1D + 1.5L$ before slip developed. Beam 6 was able to sustain 1000 cycles of $1D$ to $1D + 2.5L$ without failure and finally failed during the seventeenth cycle of $1D + 3L$. Its companion, Beam 11, showed first signs of slip at 300 cycles of $1D$ to $2L$ and ultimately failed at 1230 cycles of $1D + 3L$.

When the repeated load tests were conceived, it was felt that from the standpoint of slip behavior, the beams should be capable of sustaining 1000 cycles of loading from $1D$ to $1D + 1L$, and should be capable of sustaining a loading of at least $1D$ to $1D + 2L$ before ultimate failure. This component requirement was set for this particular test, even though it was realized that the performance of the main beam as a separate component does not necessarily simulate the behavior of the complete system. All of the beams tested which had partial column connections, except for Beam 9, satisfied this requirement. The reason that the presence

of the partial connections had no effect on Beam 9 is not clear.

Beam strength was substantially improved at an insert spacing of 9½ in, as in Beams 6 and 11.

The beams tested as isolated components experienced considerably larger deflections at $1D + 1L$ than did the center main beam of the test structure (figure 9.2), indicating that the component test was conservative in comparison with the system test.

11. Material Tests

11.1. Introduction

Tests were conducted on the concretes used in the various parts of the structure as well as on the reinforcing steel used in the precast components. The objective of these tests was to determine the relationship between minimum specified properties of materials and the properties of the materials used in the test structure, and to determine material properties which might be useful in analyzing the tests on the main structure and structural components.

11.2. Concrete Tests

The concretes tested were: (1) concrete used in the precast components, except the channel slabs, (2) concrete used in the on-grade floor slab, and (3) concrete used for the topping slab. Concrete specimens were tested for the following: (1) compressive strength, (2) tensile splitting strength, (3) unit weight, (4) air content, and (5) modulus of elasticity.

11.2.1. Precast Component Concrete

The precast components were cast in two days (April 16 and 17, 1968) from five batches of lightweight aggregate concrete. This concrete was made from a ⅜-in maximum size expanded shale aggregate, with preformed foam added at the time of mixing. A rather high cement content (about 9 U.S. bags per cubic yard) was used, and water was added to produce a workable mix. The amount of the preformed foam used was adjusted to provide a concrete with a fresh weight of about 96 lb/ft³ at the mixer. The slump was judged to be about 2 in although it was not measured. The workability of the concrete was excellent with no indication of either segregation or bleeding.

Test specimens (6 × 12 in cylinders) were cast in cardboard molds from four of the five batches. These specimens were shipped in the molds to the test site with the structural com-

ponents and were removed from the molds when about 8 days old. They were then stored in the laboratory air until tested.

The components were cast under commercial conditions and no records were available which would permit the association of individual components with particular batches of concrete.

11.2.2. Floor and Topping Slab Concrete

The on-grade floor slab was cast from a 1-in maximum size crushed-stone concrete delivered by a ready-mix truck. The mix proportions and slump are not known. The compressive test specimens were molded in 6 × 12 in cast iron molds which were removed when the concrete was 3 days old. The specimens were then air-dried until tested.

The topping slab was cast from a standard 6-bag, 3000 psi lightweight mix delivered by a ready-mix truck in two batches. The first batch was placed in the west section of the topping slab. The coarse aggregate was a ¾-in maximum size expanded shale and the fine aggregate was a natural sand. The compressive test specimens were molded in 6 × 12 in cast iron molds which were removed at 2 days of age. The specimens were then air-dried until tested.

11.2.3. Concrete Test Results

The results from the strength tests are shown in table 11.1. The unit weight and air-content determinations are presented in table 11.2. Air-content determinations were made by ASTM Method C-457[12]. The values of the modulus of elasticity are shown in table 11.3. By way of comparison, values for an average lightweight aggregate concrete are included in these tables. These values are averages from a total of 46 batches of concrete made from 21 different expanded shale, lightweight aggregates[13]. The average cement content for these concretes was 6.5 bags/yd and the average wet density was 100.3 lb/ft³.

The results indicate that: (1) the compressive strength of the lightweight concrete used in the precast components was well above the design strength of 3500 psi; (2) there was considerable variation in the strengths from batch to batch of the lightweight concrete; (3) there was considerable variation in the unit weights from batch to batch of the lightweight concrete; and (4) there was an apparent increase in the unit weight of the concrete as placed in the precast components when compared to the fresh unit weight at the mixer (about 96 lb/ft³).

These indications justify three conclusions:

(1) The concrete strengths in the test structure were significantly higher than the

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strengths called for in the plans and specifications of the Neal Mitchell System.

(2) Handling and placing techniques of the fresh concrete can affect the unit weight (and therefore the strength) of the concrete. This is especially true in the case of the high-air-content concrete used in the precast components. The unit weight of the concrete at the mixer may not necessarily be equal to the unit weight of the concrete in the form.

(3) When working with lightweight concretes, quality control tests on the fresh concrete should be made at the point of placing in such a manner that handling and placing effects can be evaluated.

11.3. Reinforcing Steel

Specimens of the reinforcing steel used in the precast components were tested to determine their yield and ultimate strengths. The results are presented in table 11.4.

12. Summary and Conclusions

12.1. Summary

A full-scale, first-story portion of a building system was tested in the laboratory in a manner that simulated the structural behavior of a three-story building under both service and potential ultimate loading conditions. Additional tests were carried out on components of this building system to determine their behavior and capacity and to provide data needed for the evaluation of the system. Performance criteria for the evaluation of the structural safety and adequacy of certain building systems were developed.

12.2. Conclusions

This series of tests demonstrated that it is feasible and practical to use structural performance tests as a basis for the evaluation of innovative building systems.

All conclusions pertaining to the structural performance of the system in question are based on the test structure as built in the laboratory and on the erection methods and materials used therein. Variation in materials and erection methods may influence performance.

The following significant deviations of the test structure from the plans and specifications of the Neal Mitchell System have been determined:

(1) The test structure had higher than specified concrete strength.

(2) Topping slab thickness exceeded that shown in the plans.

(3) Floor channel reinforcement size was increased.

(4) Gypsum wallboard thickness was less than that shown in the plans.

(5) Greater than ordinary variations in concrete strength and in the dimensions of precast members were observed.

The following conclusions relative to the performance of the building system have been reached:

(1) The building system satisfied the performance criteria which were set for its evaluation with substantial margins. As a system it exhibited strength and stiffness in excess of service- and ultimate load requirements.

(2) The walls of the system behaved as an integral part of the structure. They provided most of the stiffness of the system with respect to lateral loads, and provided a significant portion of the stiffness against vertical loads.

(3) The building system with its walls removed had considerable reserve strength above the required ultimate vertical load bearing capacity; however, without the aid of its walls it was not capable of resisting the required service wind loads.

(4) All columns tested as separate components satisfied component requirements. Column creep tests indicate that the application of service loads over a long period of time is not likely to result in creep-buckling of columns.

(5) Five of six subassemblages consisting of a precast main beam, a section of the topping slab and a partial column-beam connection were able to resist 1000 cycles of repeated loading from dead to dead-plus-live load without exhibiting signs of deterioration. Two such subassemblages with reduced shear connector spacing satisfied this performance requirement by a considerable margin. The column-beam connections appeared to play a major role in shear transfer between the precast beam and the cast-in-place topping slab.

13. Acknowledgment

EARLE F. CARPENTER was the Research Engineer in charge of electronic data processing and participated in the preparation of this report.

THOMAS W. REICHARD, Research Physicist, was in charge of specimen and material testing and participated in the preparation of Chapters 10 and 11 of the report.

ROBERT G. MATHEY, Assistant Chief of the Structures Section, supervised and coordinated the testing effort.

JOHN E. BREEN, Professor of Civil Engineering at the University of Texas critically reviewed the report and made many helpful suggestions.

The team effort of these persons made the successful completion of this project possible.

14. References

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[4] Federal Housing Administration Research Paper No. 30 (1954).

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[13] Reichard, T. W., Creep and Drying Shrinkage of Lightweight and Normal Weight Concrete. NBS Monograph 74, National Bureau of Standards, Washington, D.C. (March 1964).

15. Tables

TABLE 6.1. *Simulated Loads*
(symbols and magnitude)

(For computations, refer to Appendix C)
Symbols for Loading:
D = Service dead load
L = Service live load
H = Service wind load

Simulated load	Symbol	Magnitude		
		1D	1L	1H
Second story column	P (kip)	10	7	-----
W-E wind	Hw (kip)	-----	-----	2.05
S-N wind (point opposite firewall)	Hs (kip)	-----	-----	4.05
S-N wind (point between firewalls)	Hs' (kip)	-----	-----	0.9
Major floor load[a] (equivalent distributed load between columns (center strip))	w (psf)	[b]46 [c]9.3	43	--
Minor floor load[a] (Equivalent distributed load between north end of structure and northern row of columns (cantilever strip))	w' (psf)	[b]46 [c]9.3	43	--

[a] w and w' were increased to allow for incomplete area coverage by the air bags (see fig. 6.1).

[b] 46 psf is the dead load weight of the floor of the test structure.

[c] 9.3 psf is the additional dead load which would be acting on the real structure but which is not present on the test structure.

TABLE 6.2. *Simulated loads in load schedule*

(For computations, refer to Appendix C)

Loading	P	Hw	Hs	Hs'	w	w'
	(kip)	(kip)	(kip)	(kip)	(psf)	(psf)
1D + 1L	17				52	52
0.9D + 1.1H	9	2.3	4.5	1.0		
1.3D + 1.7L	25				100	100
1.3D + 1.7L + 0.8H	25	1.6	3	0.7	100	100
1.25 (1.5D + 1.8L)	35				144	144
0.9D + 1.4H	9	2.9	5.7	1.3		

Table 10.1. *Column tests (actual dimensions and reinforcement cover of the columns)*

Column No.	Outside dimensions in inches		Cover Measurements in inches							
	"X"	"Y"	A	B	C	D	E	F	G	H
1	4.76	6.05	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{11}{16}$	$\frac{13}{16}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{11}{16}$
2	4.77	6.07	$1\ \frac{1}{8}$	1	$1\ \frac{1}{16}$	$\frac{7}{8}$	1	$1\ \frac{1}{8}$	$1\ \frac{1}{16}$	$1\ \frac{1}{8}$
3	4.76	6.05	-------	-------	-------	-------	-------	-------	-------	-------
4	4.76	6.15								
5	4.79	6.16	$1\ \frac{1}{4}$	$1\ \frac{1}{4}$	$1\ \frac{3}{16}$	$1\ \frac{1}{4}$	$1\ \frac{1}{16}$	$1\ \frac{1}{16}$	$1\ \frac{1}{8}$	$\frac{7}{8}$
6	4.81	6.06	$1\ \frac{1}{8}$	$1\ \frac{1}{16}$	$1\ \frac{7}{8}$	$1\ \frac{1}{16}$	$\frac{7}{8}$	$1\ \frac{1}{8}$	1	$\frac{3}{4}$
7	4.78	6.05	$1\ \frac{1}{4}$	$1\ \frac{3}{16}$	$\frac{7}{8}$	1	$1\ \frac{1}{16}$	$1\ \frac{1}{8}$	$1\ \frac{3}{8}$	$1\ \frac{3}{16}$
8	4.74	6.15	$1\ \frac{1}{4}$	$1\ \frac{1}{8}$	$\frac{15}{16}$	1	$1\ \frac{11}{16}$	$1\ \frac{1}{4}$	$\frac{3}{4}$	$1\ \frac{5}{16}$
9	4.75	6.20	$1\ \frac{1}{8}$	$1\ \frac{1}{8}$	$\frac{7}{8}$	$\frac{7}{8}$	$1\ \frac{1}{16}$	$1\ \frac{1}{8}$	$1\ \frac{1}{8}$	$1\ \frac{1}{8}$
Average	4.77	6.10	1.12	1.07	0.96	0.98	0.92	1.08	1.03	1.01
specified (fig. 5.13)	4.75	6.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
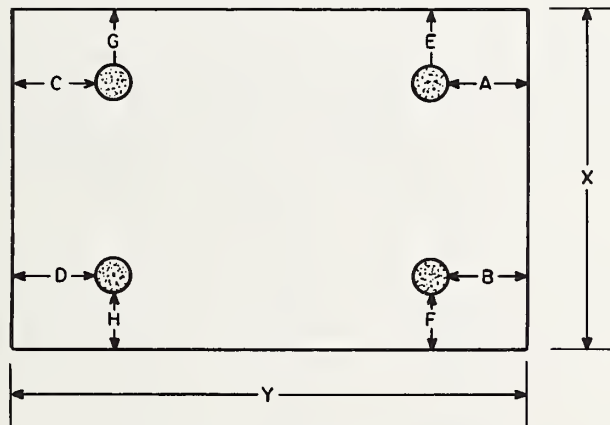
Table 10.1. *Illustrative figures*

TABLE 10.2. *Column tests*
(ultimate loads and failure modes)

Column No.	Major axis eccentricity	Minor axis eccentricity	Ultimate load	Type of failure
	(in)	*(in)*	*(kip)*	
1	0.0	0.5	73.0	Bottom fixture
5	0.0	0.5	82.0	Concrete mid-ht.
8	0.0	0.5	81.8	Concrete mid-ht.
			Avg. = 78.9	
2	2.0	0.0	56.6	Top fixture
6	2.0	0.0	51.0	Top fixture
7	2.0	0.0	45.9	Top fixture
			Avg. = 51.2	
9	1.5	0.0	89.5	Concrete mid-ht.

TABLE 10.3. *Beam tests (insert spacing and concrete strength of specimens)*

Beam No.	Inserts	Date cast		Date beam tested	Concrete strength, psi[a]	
		Beam	Topping		Beam	Topping
5	Star[c] at 19 in	4/17	5/10	5/20	7000	3400
7	Star at 19 in	5/15	[b]6/7	6/13	6740	4100
9	Star at 19 in	5/15	[b]6/7	6/15	6740	4100
8	Richmond at 19 in	6/5	[b]6/7	6/14	6400	4100
10	Richmond at 19 in	6/5	[b]6/7	6/17	6400	4100
6	Richmond[d] at 9½ in	6/5	[b]6/7	6/12	6400	4100
11	Richmond at 9½ in	6/5	[b]6/7	6/18	6400	4100

[a] Approximate strength when beam was tested. All concretes were made from expanded shale and were similar to concretes used in the test structure.

[b] Column stubs cast 6/5 from concrete used in beams cast on that date.

[c] All beams with 19 in insert spacing between supports had one insert outside each support at 18 in from the centerline of the support.

[d] All beams with 9½ in insert spacing between supports had three inserts outside each support at 6, 18 and 30 in from the centerline of the support.

NOTES ON INSERTS:

1. Star inserts were $\frac{3}{8}$ in zinc base, die-casting alloy.
2. Richmond inserts were $\frac{3}{8}$ in grey cast-iron "Kohler".
3. Jam nuts were used with Star inserts.
4. Cross-bars on shear-studs were 8 in long when inserts were spaced at 9½ in between columns.

TABLE 10.4. *Beam tests (repeated load test results)*

Beam No.	Inserts used	Beam mid-span deflections, inch[a]										Results	
		Increment 1		Increment 2		Increment 3		Increment 4		Increment 5		Slippage observed	Collapse occurred
		D	$D+L$	D	$D+1.5\,L$	D	$D+2\,L$	D	$D+2.5\,L$	D	$D+3\,L$		
5	Star at 19 in	0.32	0.72	0.68	1.31	0.82	1.80	0.92	2.45	1.01	----------	1 cycle of 1 L	1 cycle of 3 L
7	Star[b] at 19 in	0.20	0.68	0.56	0.97	0.78	1.48	0.99	1.89	------	----------	Few cycles of 1.5 L	3 cycles of 3 L
9	Star[b] at 19 in	0.17	0.76	0.61	1.07	0.81	1.43	0.96	2.06	------	----------	1 cycle of 1 L	820 cycles of 2.5 L
8	Richmond[b] at 19 in	0.18	0.48	0.30	0.65	0.80	1.36	1.00	2.23	------	----------	Few cycles of 1.5 L	4 cycles of 2.5 L
10	Richmond[b] at 19 in	0.15	0.50	0.38	0.74	0.46	1.07	0.72	1.45	0.86	----------	100 cycles of 1.5 L	1 cycle of 3 L
6	Richmond[b] at 9½ in	0.17	0.54	0.42	0.82	0.70	1.28	0.82	1.63	1.28	3.05	500 cycles of 1.5 L	17 cycles of 3 L
11	Richmond[b] at 9½ in	0.21	0.55	0.40	0.79	0.50	1.02	0.59	1.21	0.70	1.52	300 cycles of 2 L	1230 cycles of 3 L

[a] Deflections reported were measured at beginning of each increment and are based on initial zero-load condition.
[b] These beams had partial column connections.

TABLE 10.5. *Beam tests (modes of insert failures)*

Beam No.	Inserts	Mode of Insert Failure
5	Star at 19 in	Inserts broken 1 in below interface and pulled out
7	Star at 19 in	Inserts broken 1 in below interface and pulled out
9	Star at 19 in	Bolts broken just above jam nuts
8	Richmond at 19 in	Bolts sheared off at slab-beam interface
10	Richmond at 19 in	Bolts sheared off at slab-beam interface
6	Richmond at 9 ½ in	Bolts sheared off at slab-beam interface
11	Richmond at 9 ½ in	Bolts sheared off at slab-beam interface

TABLE 11.1. *Concrete tests*[a]

Concrete	Batch	Date tested	Age at test	Compressive strength		Splitting strength
				Specified	Actual	
			Days	psi	psi	psi
Precast component[b]	4/16—B	May 10	24	3500	5100	[c]346
Precast component[b]	4/16—C	May 10	24	3500	4930	[c]287
Precast component[b]	4/17—A	May 10	23	3500	7160	[c]369
Precast component[b]	4/17—B	May 10	23	3500	7090	
Floor slab		May 10	17	2000	5600	
Topping slab	A	May 21	26	3000	3840	
Topping slab	B	May 21	26	3000	2560	
Avg. value for typical lightweight aggregate concrete[d]			28		5800	

[a] The tests on the laboratory structure were carried out during the period of May 10–22. See Appendix C for actual dates of each test.
[b] Concrete for the long columns (first story) was specified to be 4500 psi strength.
[c] Tested May 22 at an age of 36 days after 25 days air drying.
[d] Derived from NBS Monograph 74, March, 1964.

TABLE 11.2. *Concrete tests (unit weights and air contents of concrete)*

Concrete	Batch	Date tested	Age	Air drying period	Unit weight	Air content
			Days	Days	lb/ft³	%
Precast component	4/16—B	May 16	30	21	98.5	11.3
Precast component	4/16—C	May 16	30	21	96.4	
Precast component	4/17—A	May 16	29	21	103.7	
Precast component	4/17—B	May 16	29	21	103.4	8.2
Floor slab		May 14	21	20	147.7	
Topping slab	A	May 16	21	20	117.7	
Topping slab	B	May 16	21	20	108.3	
Average values for typical lightweight aggregate concrete[a]			28	27	97.0	4.7

[a] Derived from NBS Monograph 74, March, 1964.

TABLE 11.3. *Concrete tests (modulus of elasticity of precast component concrete*

Batch	Age at test	Date tested	Compressive strength ($f'c$)	Secant modulus[a]
	days		*psi*	10^6 *psi*
4/16—B	41	May 27	6400	2.1
4/16—C	41	May 27	6030	2.2
4/17—B	40	May 27	7380	2.4
Avg. value for light-weight aggregate concrete[b]	28		5850	2.2

[a] Secant Modulus at 0.4 $f'c$ after prior loading to 0.5 $f'c$ several times.
[b] Derived from NBS Monograph 74, March 1964.

TABLE 11.4. *Reinforcing steel test results*

Specimen	Where used	Yield strength		Ultimate strength
		Specified	Actual	
		psi	*psi*	*psi*
No. 5 deformed bar	Beams	60,000	65,000	96,000
No. 4 deformed bar	Cantilever beams	60,000	73,000	117,000
No. 2 plain bar	Tie bars in columns	40,000	47,000	77,000

16. Figures

FIGURE 5.1. *Typical complete structure.*

FIGURE 5.2. *Structural system model.*
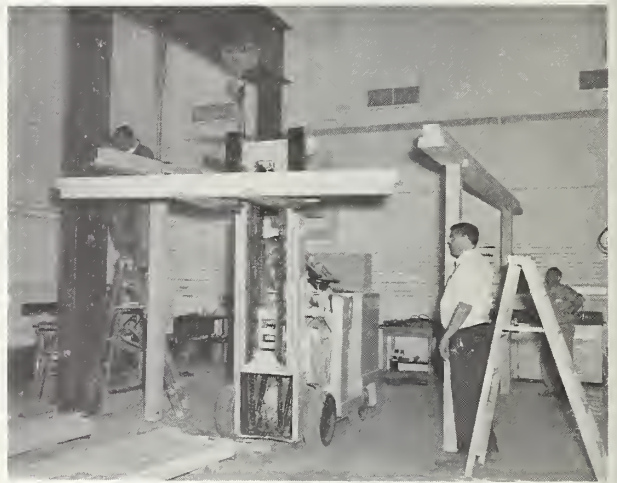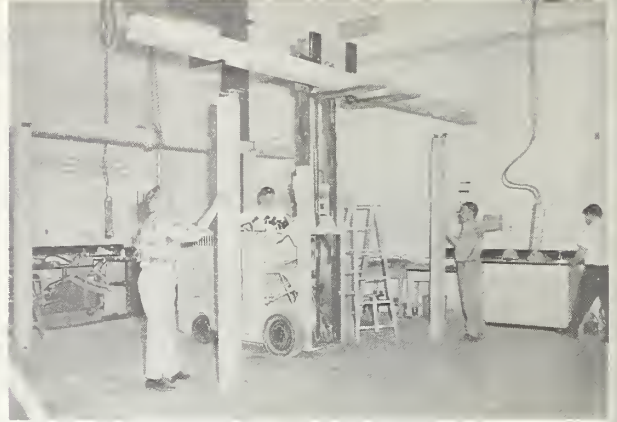
FIGURE 5.4. *Assembling structural frame.*
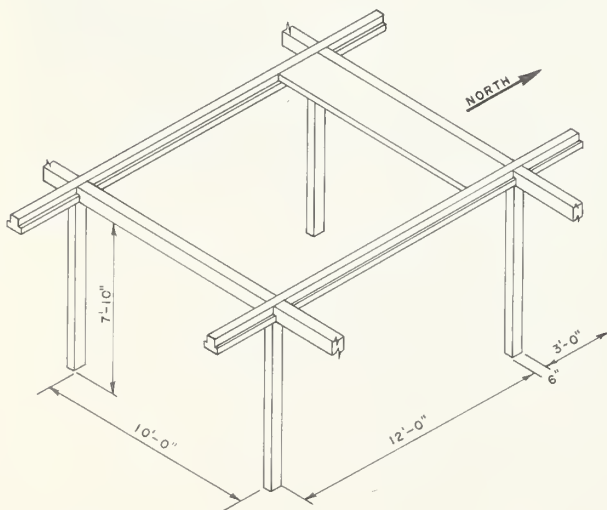
FIGURE 5.3. *Typical bay of two-bay frame test.*
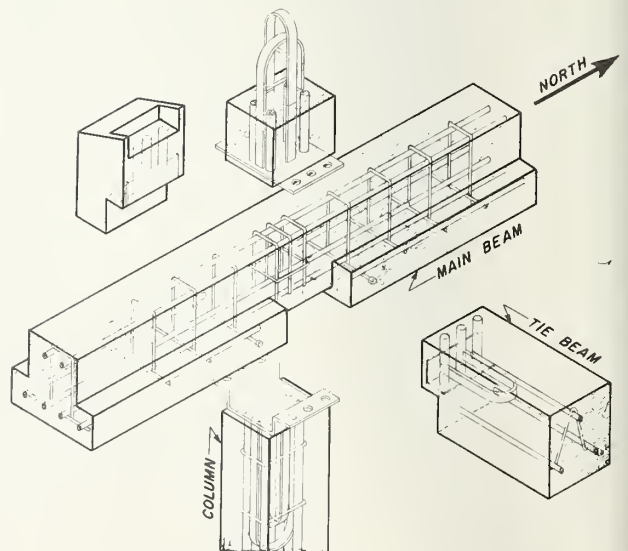
FIGURE 5.5 *Beam and column reinforcement.*
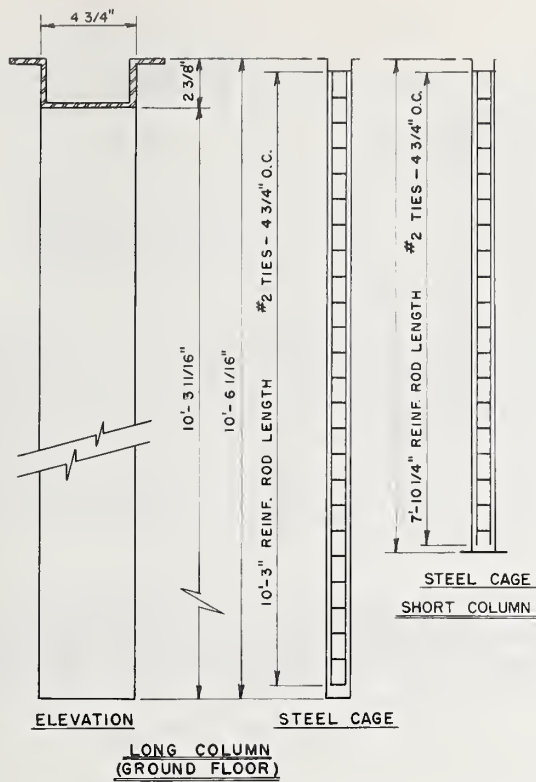
FIGURE 5.6. *Elevation of column reinforcement.*
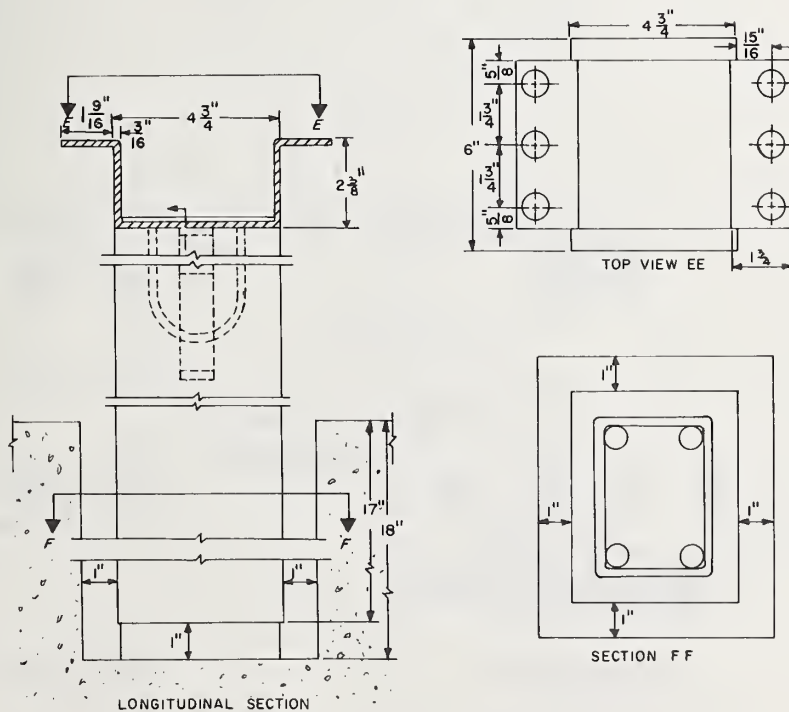
FIGURE 5.7. *Long column details.*

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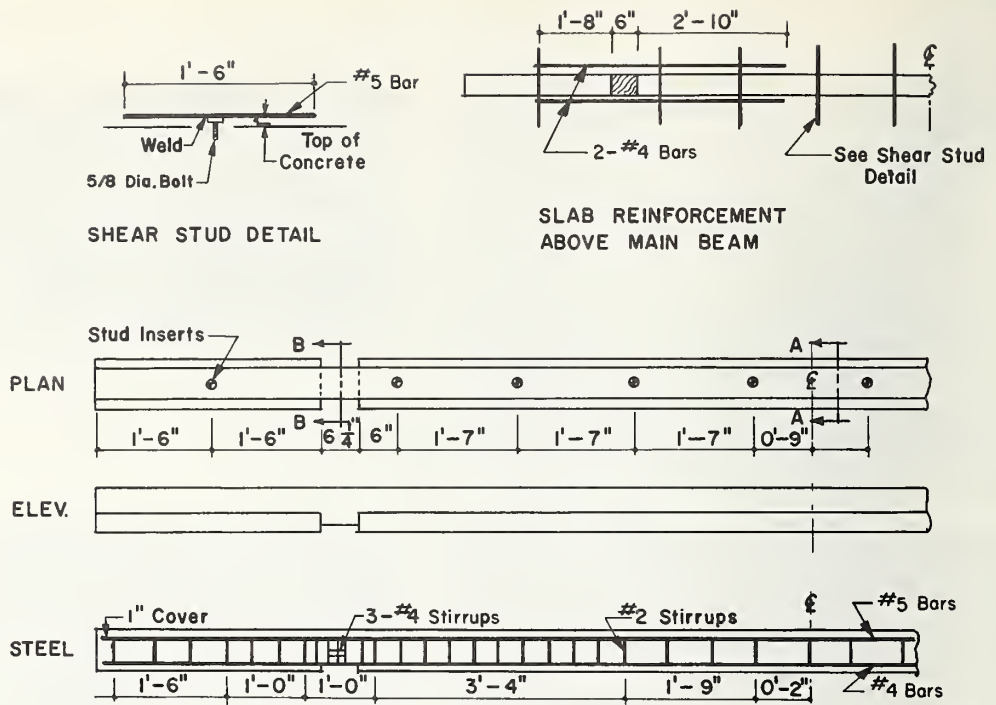
FIGURE 5.8. *Main beam reinforcement.*
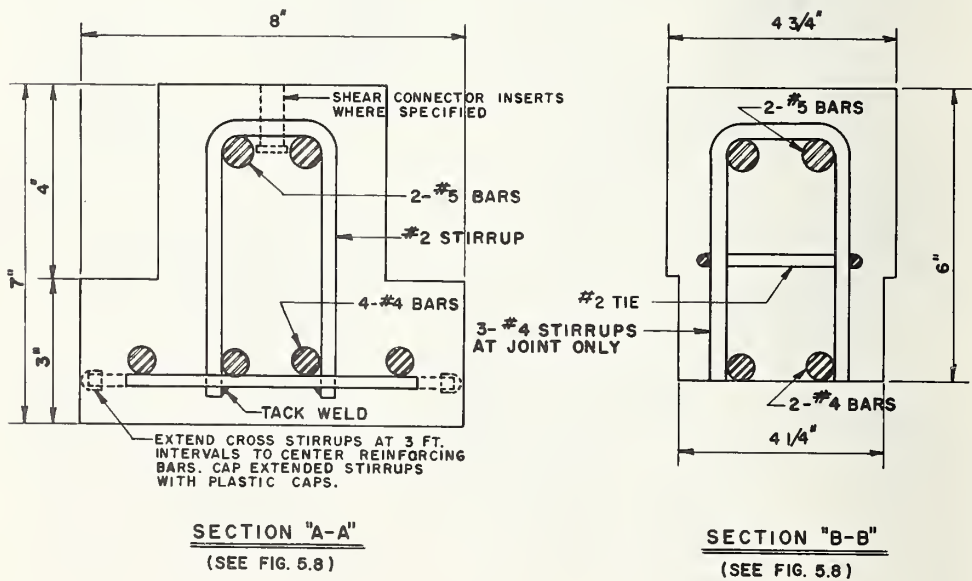
FIGURE 5.9. *Main beam sections.*
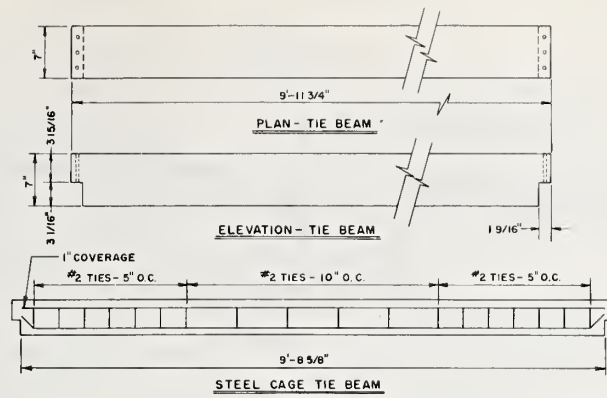
FIGURE 5.10. *Tie-beam plan and elevation.*
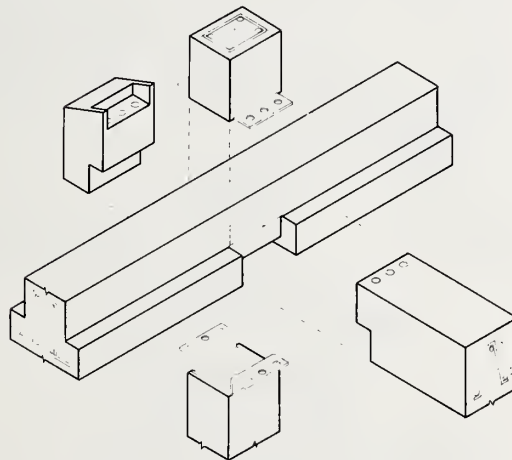
FIGURE 5.11. *Beam and column connection.*

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SECTION A-A END CONDITION

SECTION B-B END CONDITION

SECTION - TYPICAL

END ELEVATION

FIGURE 5.12. *Tie-beam end details.*

BASE CONNECTION ℞ PLAN

HEAD CONNECTION ℞ PLAN

PLAN SECTION @ COL. BASE
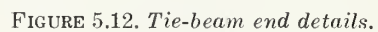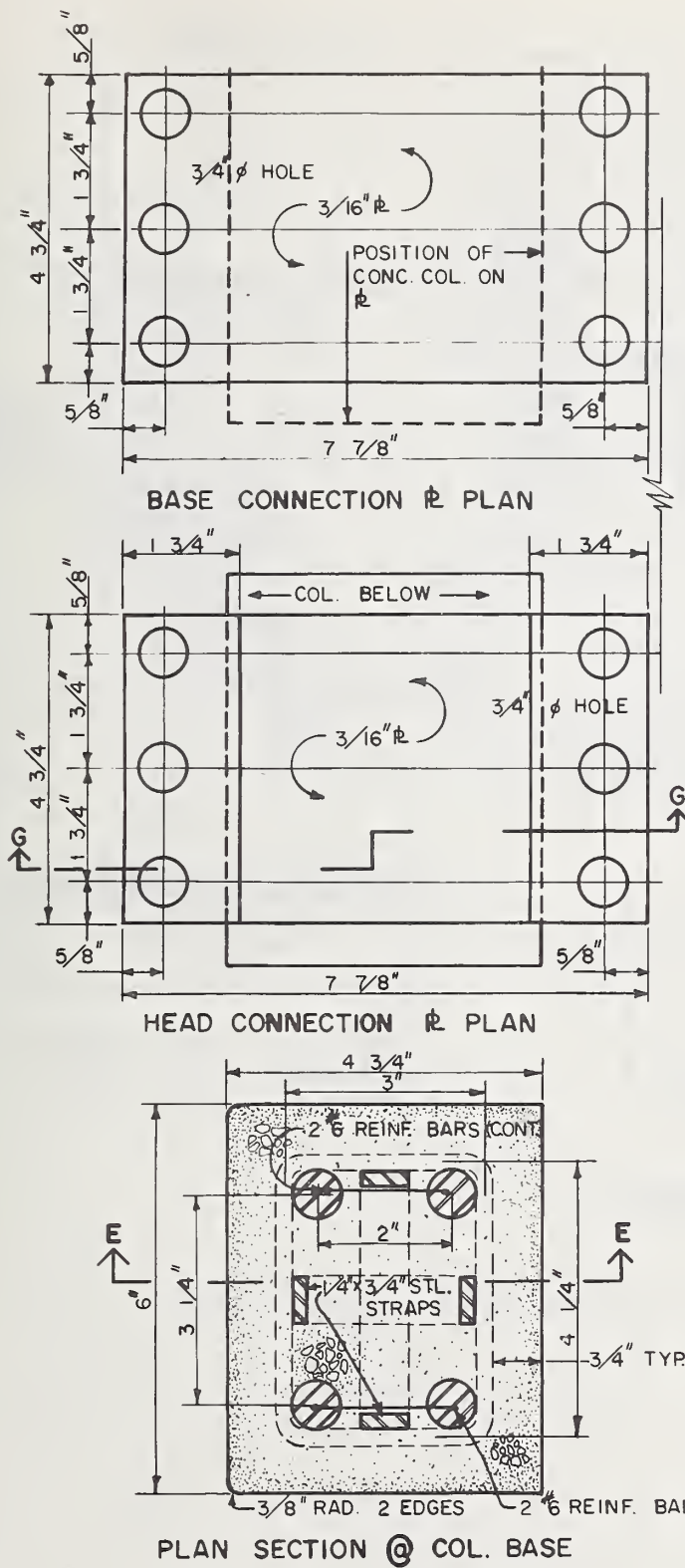
FIGURE 5.13. *Plan sections of column connections.*

BASE CONN. ⅊
SEE SECT. F-F
THIS SH'T.

NOTE:
FOR DETAILS OF
TIE BEAM FASTENING
TO CONN. ⅊ SEE
SH'T. C-D-S-14

1/8" TH'K.
NEOPRENE PAD

2 3/8"

5"

4"

10"

1'-0"

#3 STIFFENER
RODS WELD TO
INSIDE FACE
OF CONN.
⅊ STRAPS

1/4" x 3/4" ST'L.
STRAP BEYOND
WELD LEGS TO
CONN. ⅊

4"

4 3/4"
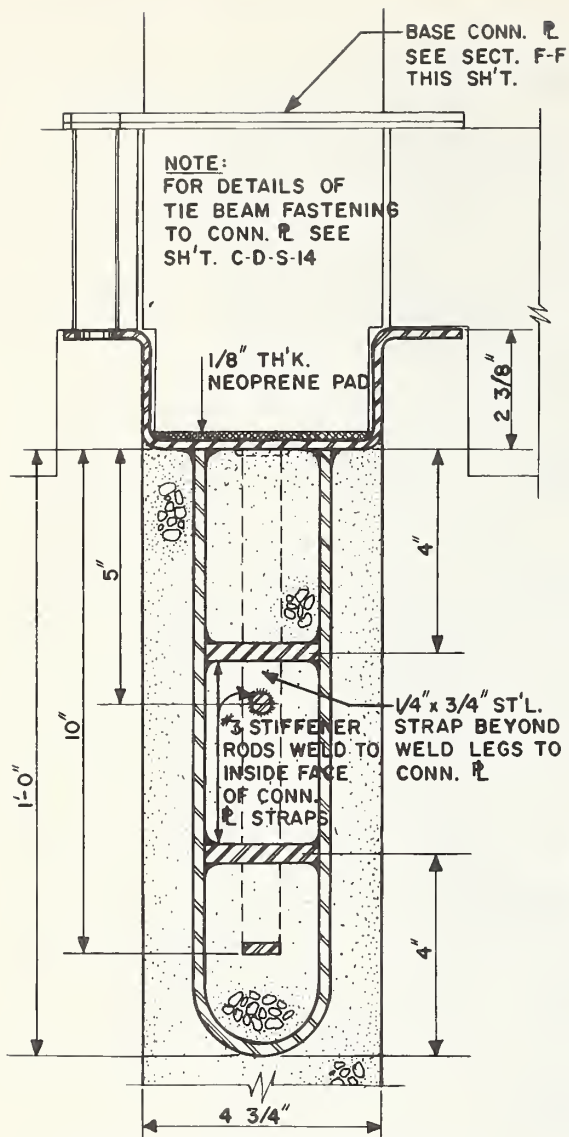
SECTION G-G HEAD CONNECTOR
⅊ SHOWING STEEL STRAPS

4 3/4"

4"

5"

1'-0"

10"

1/4" x 3/4" ST'L.
STRAP BEYOND
WELD LEGS TO
CONN. ⅊

#3 STIFFENER
RODS WELD TO
INSIDE FACE
OF CONN.
⅊ STRAPS

4"

1/8" TH'K. NEOPRENE PAD

2 3/4" INSIDE
FACE OF TIE ROD

7 7/8"
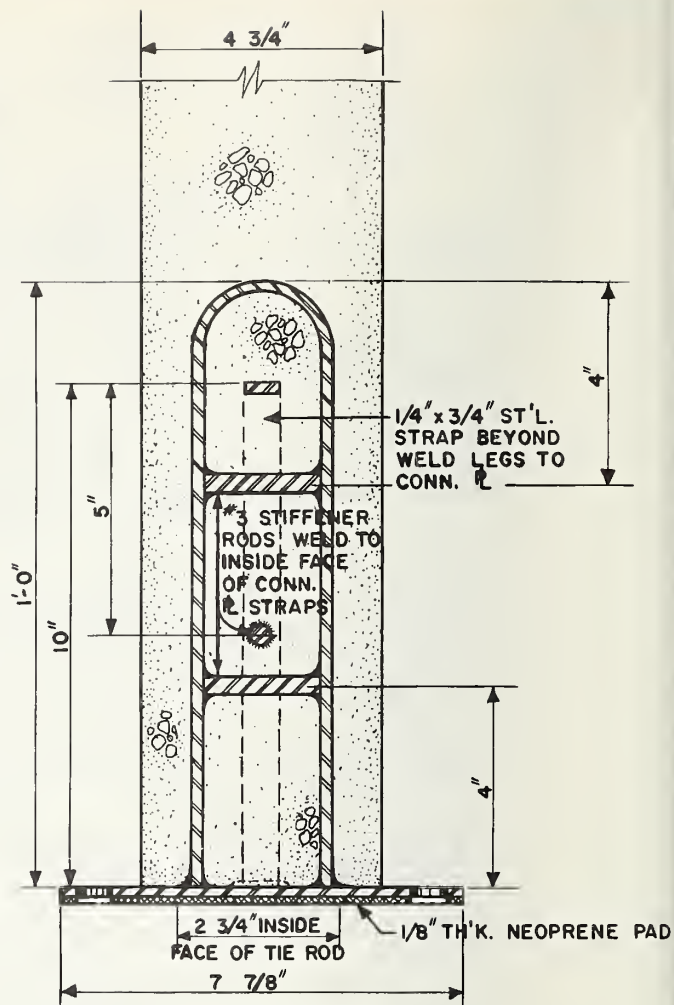
SECTION E-E BASE CONNECTOR
⅊ SHOWING STEEL STRAPS

FIGURE 5.14. *Elevation sections of column connections.*

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SECTION A-A

3/16″ PLATE
1/8″ BEARING PAD
THINWALL METAL TUBING 3/4″ I.D.
3/4″ HIGH STRENGTH BOLT
1/8″ PLATE
℄ OF 4″ NELSON STUD
1/8″ CLEARANCE EDGE OF ℔ TO FACE OF TIE BEAM
3/16″ PLATE
1/8″ BEARING PAD
EDGE OF LEDGE ON CANTILEVER BEAM
KEY FOR GROUT
1/16″ CLEARANCE
GROUT FROM TOP FLUSH WITH BOTTOM OF BEAM

1/8″ CLEARANCE

Figure 5.15. *Connection detail.*

39
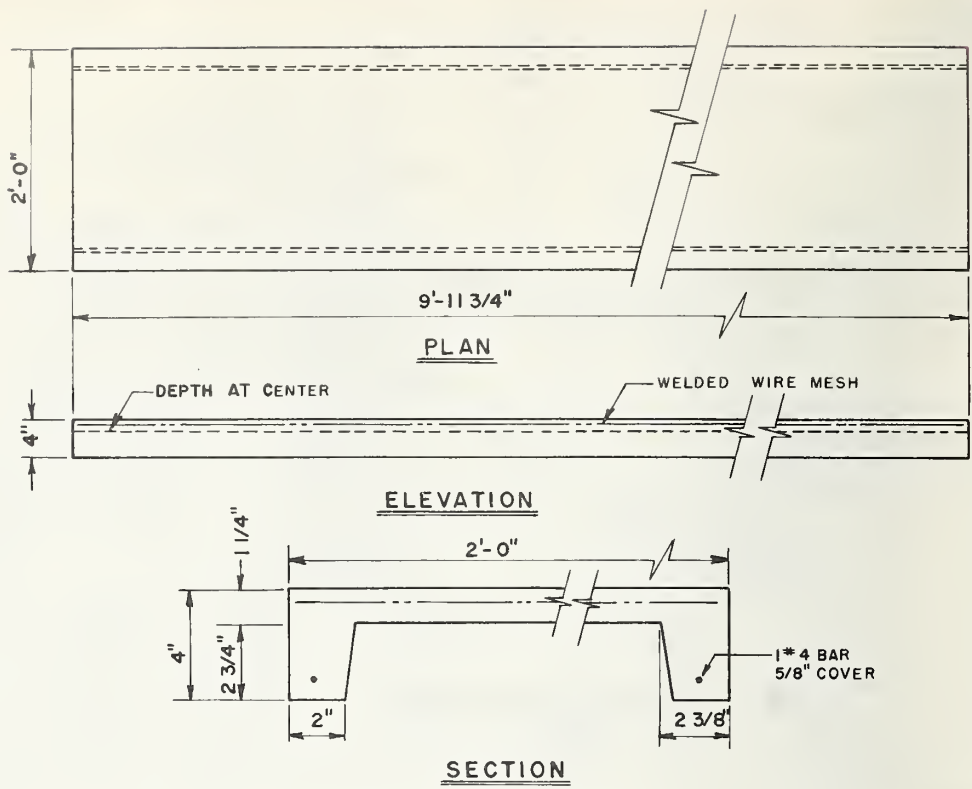
FIGURE 5.16. *Floor channels.*
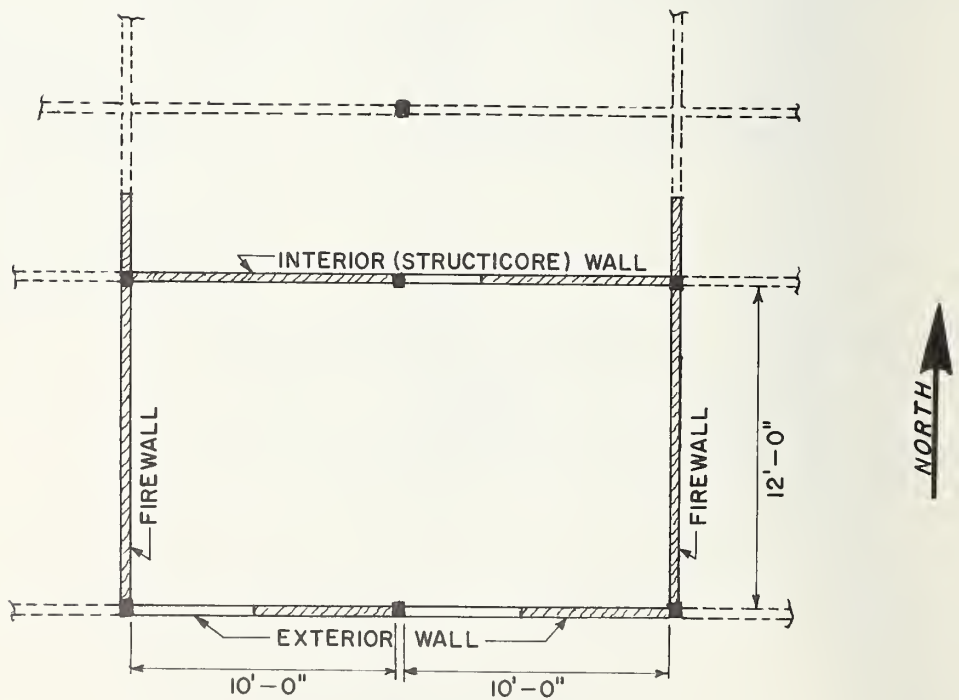
FIGURE 5.17. *Plan section through walls.*

INTERIOR FACE OF COLUMN

1/4" X 3/8" URETHANE ADHESIVE BOTH SIDES

1/2" X 1 1/8" ANGLE CEMENT TO SHEETROCK

2" X 1/4" URETHANE

5/8" SHEETROCK

1/2" SHEETROCK

25 GAUGE GALV CHANNEL RUNNER (2 1/2") NOTE: DO NOT FASTEN VERTICAL STUDS TO THE RUNNER THEY RIDE UP & DOWN

COLUMN FACE
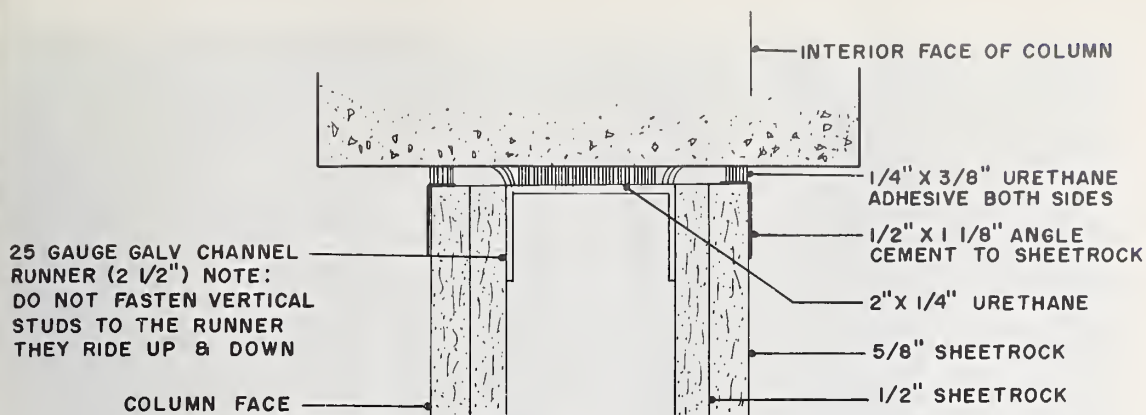
SECTION
DETAIL AT HEAD
2 HOUR RATING

COLUMN

1/4" URETHANE

CAULK

1/4" X 3/8" URETHANE ADHESIVE BOTH SIDES

1/2" X 1 1/8" ANGLE CEMENT TO SHEETROCK

25 GAUGE 2 1/2" GALV STEEL STUD

5/8" SHEETROCK

1/2" SHEETROCK
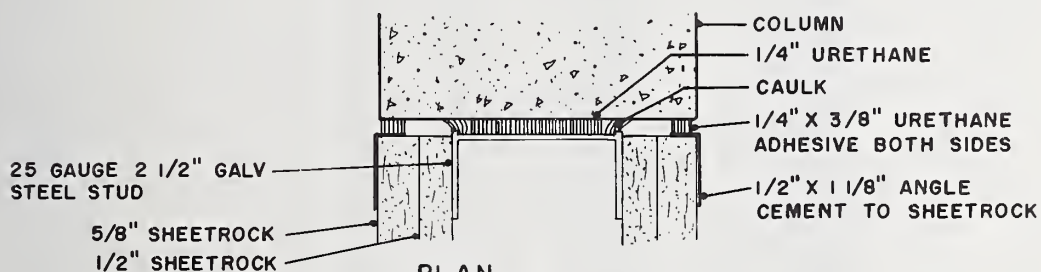
PLAN
DETAIL AT COLUMN
2 HOUR RATING

5/8" GYP SHEETROCK

1/2" GYP SHEETROCK

2" TOPPING (CONC) RESILIENT FLOORING
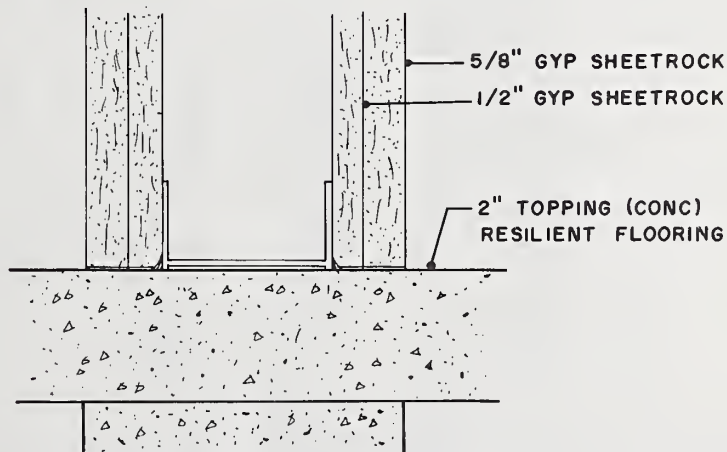
SECTION
DETAIL @ SILL
2 HOUR RATING

FIGURE 5.18. *Fire wall.*
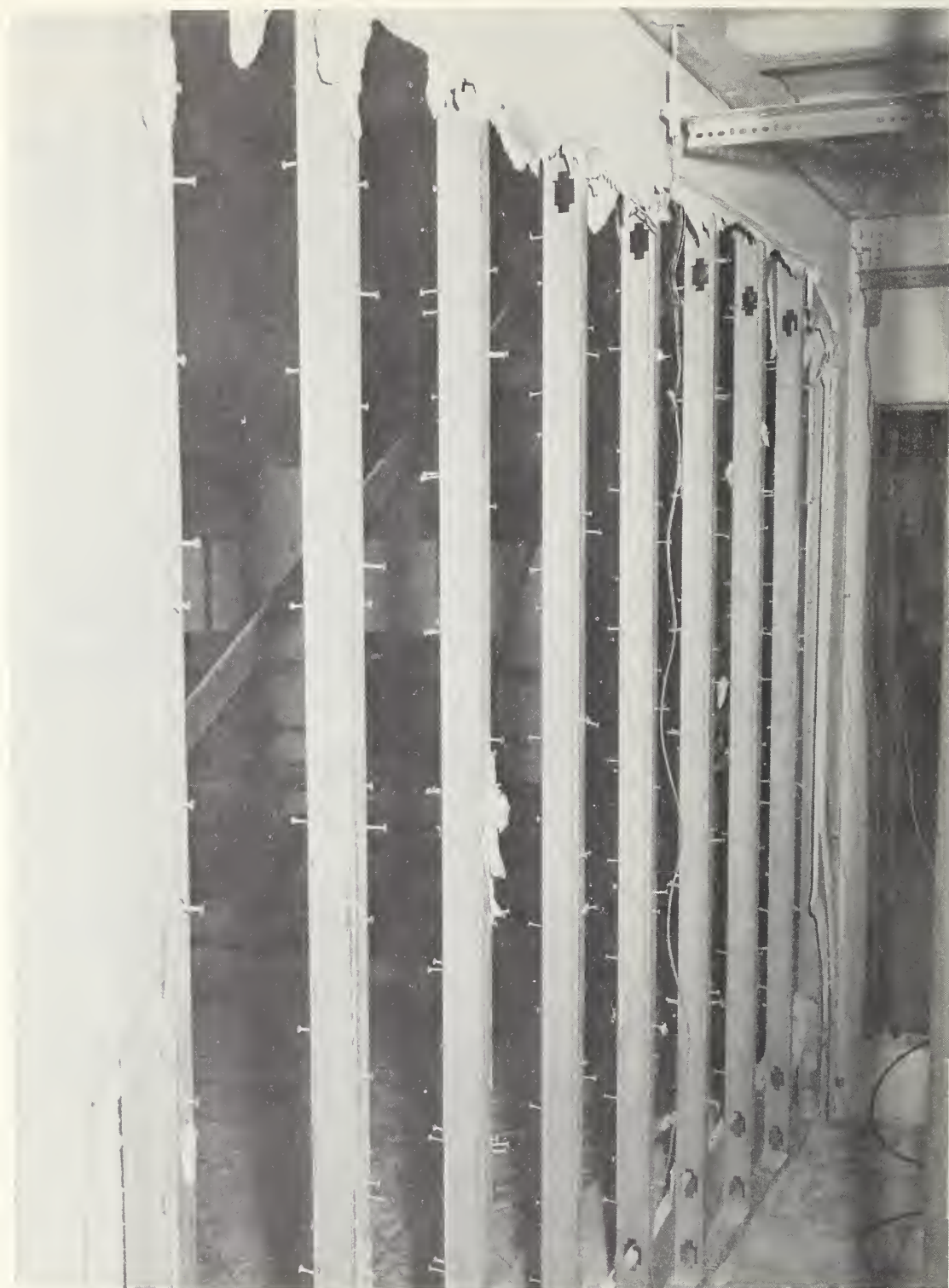
FIGURE 5.19. *Fire wall (partially dismantled).*

COMPRESSIBLE FILLER
CAULKING

EDGES SEALED
FACE STAINED

2"x 1/4" URETHANE DRAWN 1/4"
(NOT COMPRESSED)

TEXTURE I-II SELECT GRADE
CEDAR GROOVES 4" O.C.

25 GAUGE GALV. CHANNEL
RUNNER

1/4" X 3/8" URETHANE
ADHESIVE BOTH SIDES

1/2" X 1 1/8" ANGLE
CEMENT TO SHEETROCK

5/8" SHEETROCK FASTEN TO
STUD W/ 1 1/4" SELF TAPPING
SCREW-TAPE AND COMPOUND
JOINTS

TOP OF VERT. STUD

SECTION OR PLAN
DETAIL AT HEAD

TEXTURE I-II SELECT GRADE
CEDAR-USE TRIM HEAD SCREW-
SELF COUNTERSINKING SEE
ELEVATION FOR SCREW
A LOCATION THIS SHEET

STAGGER JOINTS

2 1/2" 25 GAUGE STEEL
STUD 16" O.C.

5/8" GYPSUM SHEETROCK
OUTSIDE FOILFACE

PLAN

SECTION
DETAIL AT SILL

TEXTURE I-II SELECT
GRADE CEDAR USE TRIM
HEAD SCREW SELF
COUNTERSINKING SEE
ELEVATION

1 5/8"

COMPRESSIBLE FILLER

CAULKING

25 GAUGE STEEL STUD GALV. 16" O.C.

5/8" SHEETROCK FASTEN TO STUD
W/ 1 1/4" SELF TAPPING SCREW TAPE
AND COMPOUND JOINTS
FOIL FACE OUTSIDE

25 GAUGE GALV. CHANNEL RUNNER

FACE OF COLUMN

CUT 1 5/8"X 2 5/8" CONTINUOUS
(PRESSURE TREATED) WOOD
MEMBER-SET IN MASTIC AT SILL

RESILIENT FLOOR

FIGURE 5.20. *Exterior wall details.*

43
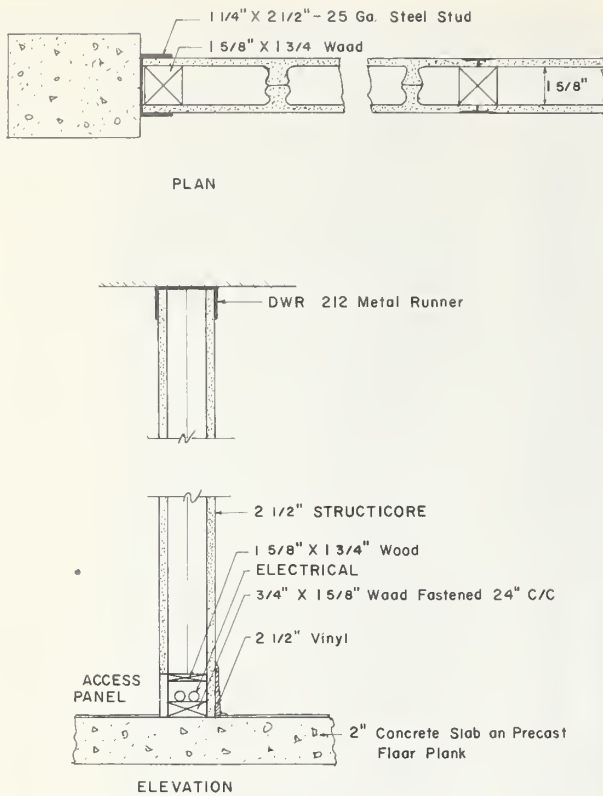
FIGURE 5.21. *Interior wall sections.*

FIGURE 5.22. *Interior wall (partially dismantled)*

FIGURE 5.23. *Foundation detail.*

FIGURE 5.24. *Test structure before the installation of walls.*
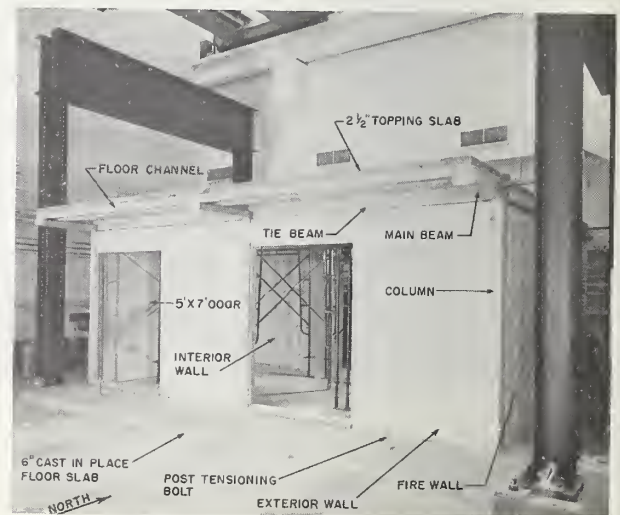
FIGURE 5.25. *Test structure after the installation of walls.*

44
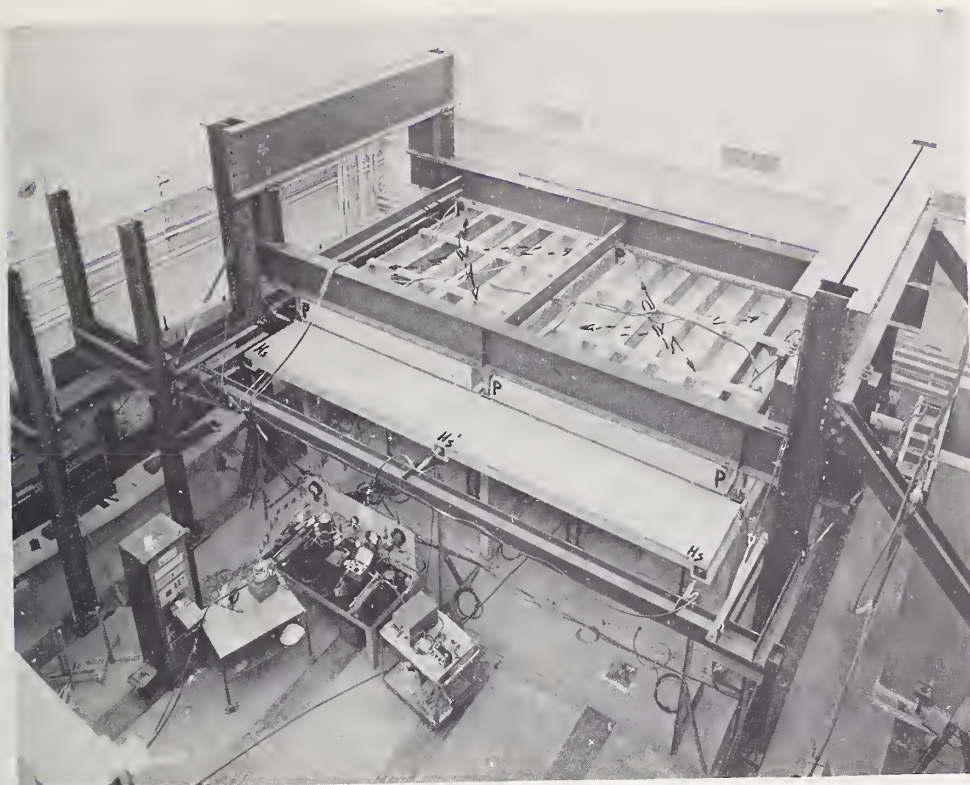
FIGURE 5.26. *Test structure with testing equipment installed (top view).*
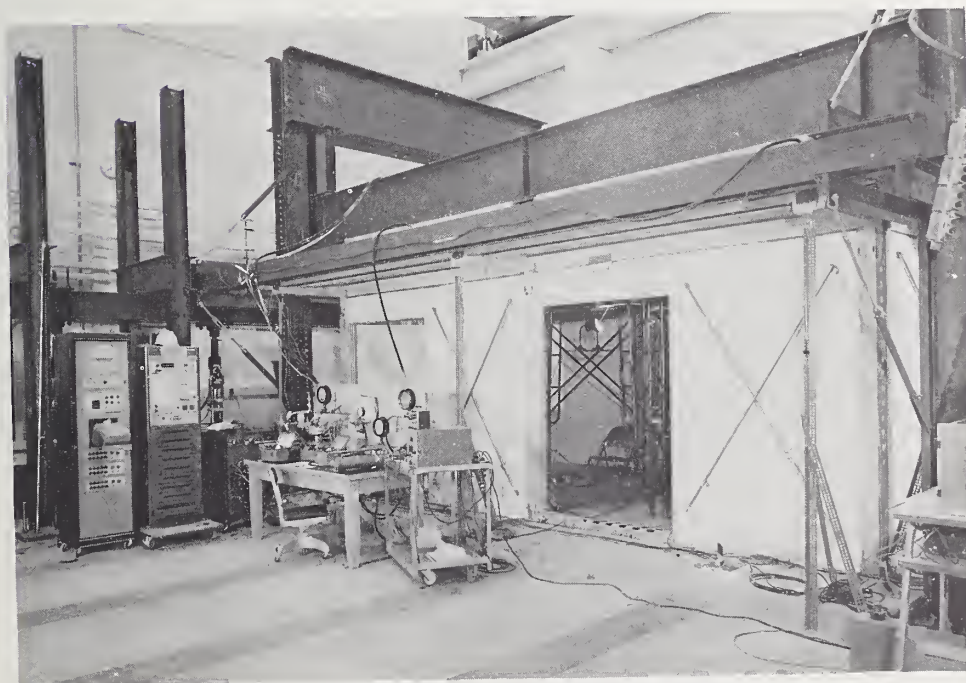
FIGURE 5.27. *Test structure with testing equipment installed (front view).*

45
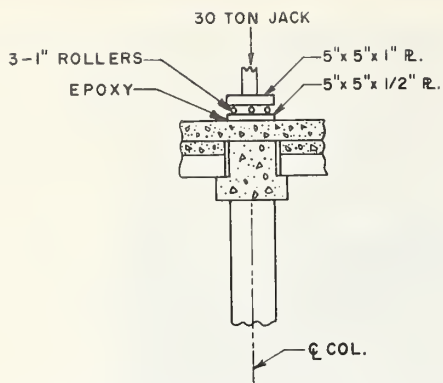
FIGURE 5.28. *Column load equipment connection.*
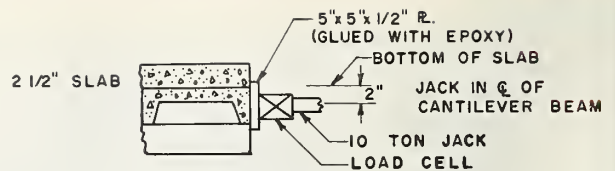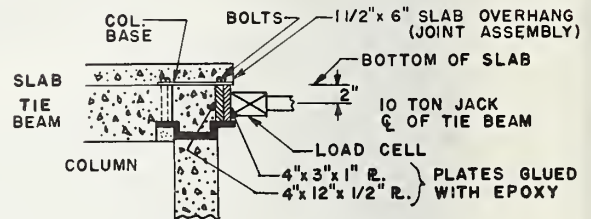
FIGURE 5.29. *Racking load equipment connection.*
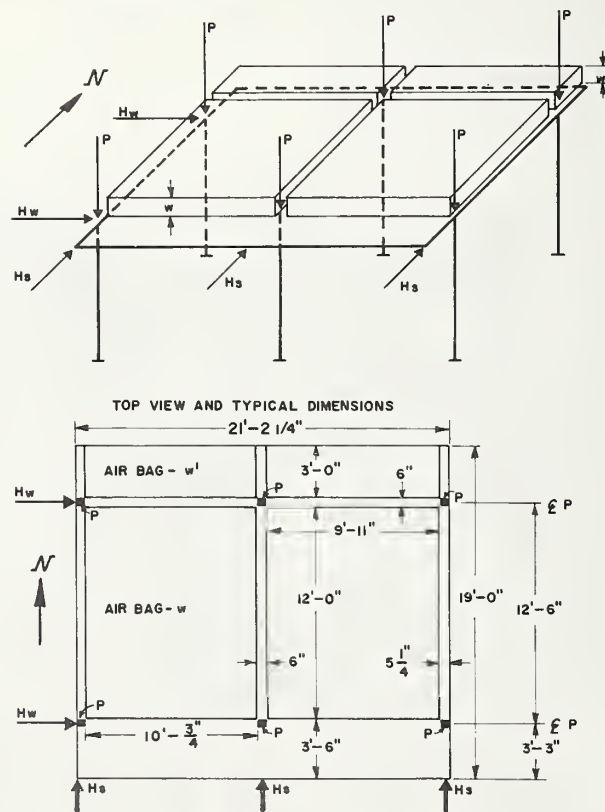
FIGURE 6.1. *Loading of test structure.*
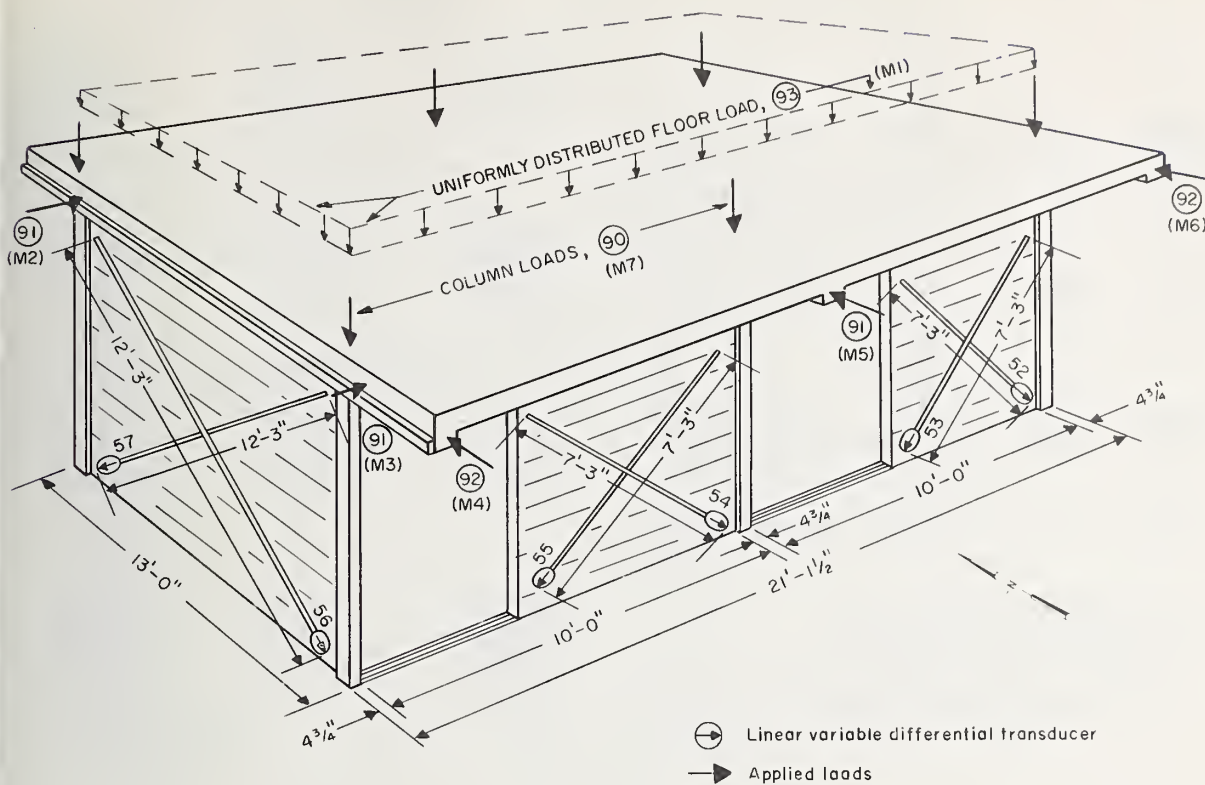
FIGURE 7.1. *Instrumentation location. Northeast schematic.*
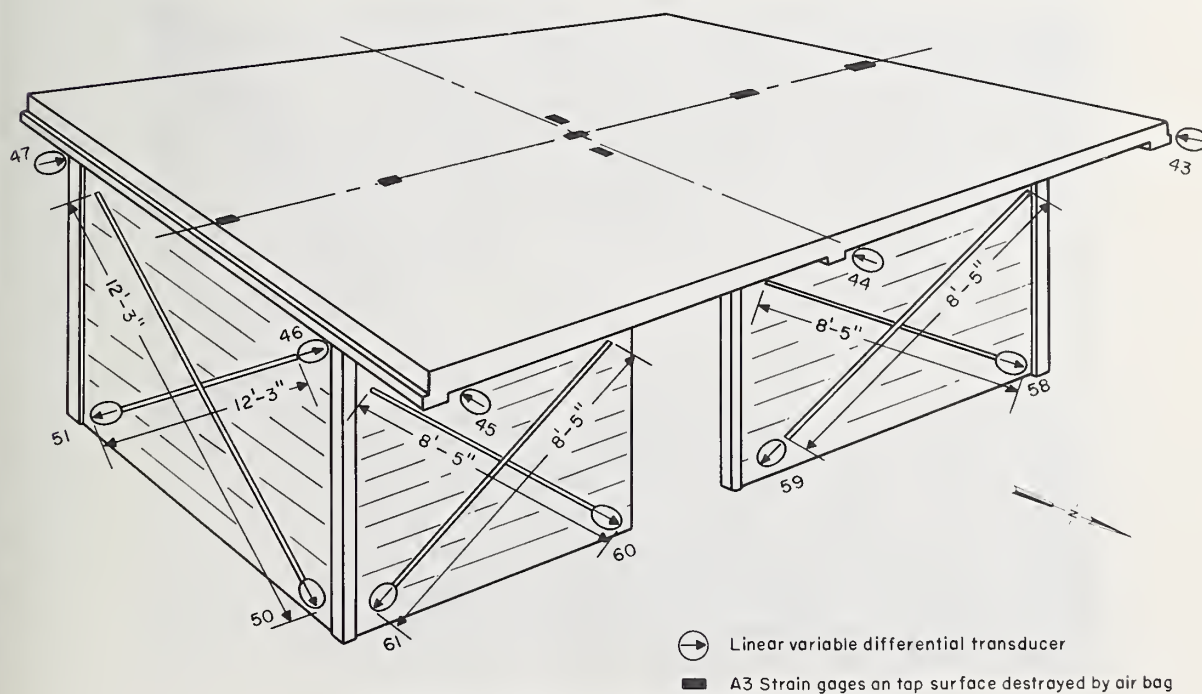
FIGURE 7.2. *Instrumentation location. Southwest schematic.*

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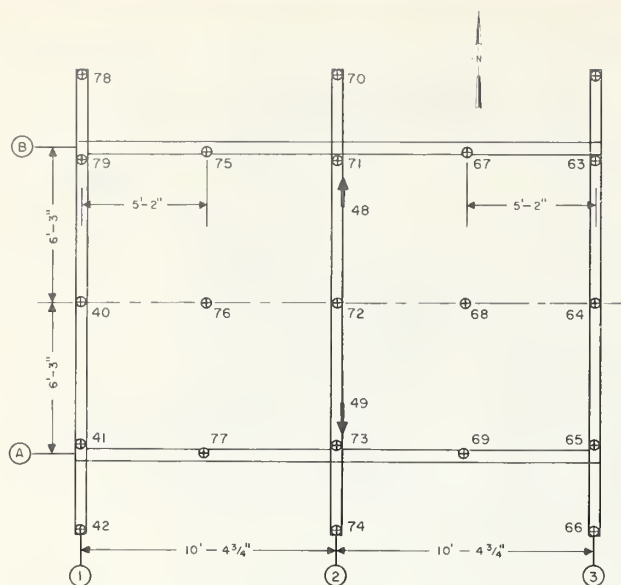
Linear variable differential transducer – vertical
Linear variable differential transducer – horizantal slip
Numbers define channel location digital recorder tape

FIGURE 7.3. Vertical deflection gages.

PLAN SECTION 6" BELOW CEILING BEAMS

PLAN SECTION 6" ABOVE FLOOR
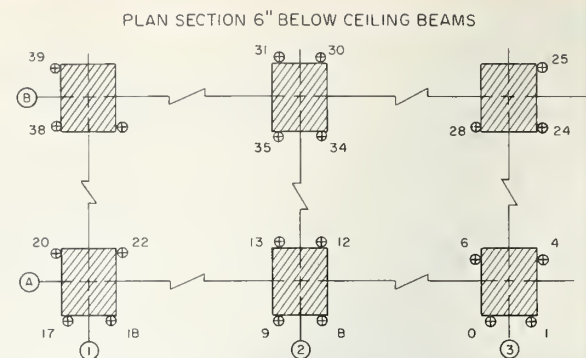
Type A–3 strain gage located 1/2"
from corner of column

FIGURE 7.4. Column strain gage location.

CHANNELS 72 AND 73, SEE FIGURES A–11 AND A–16
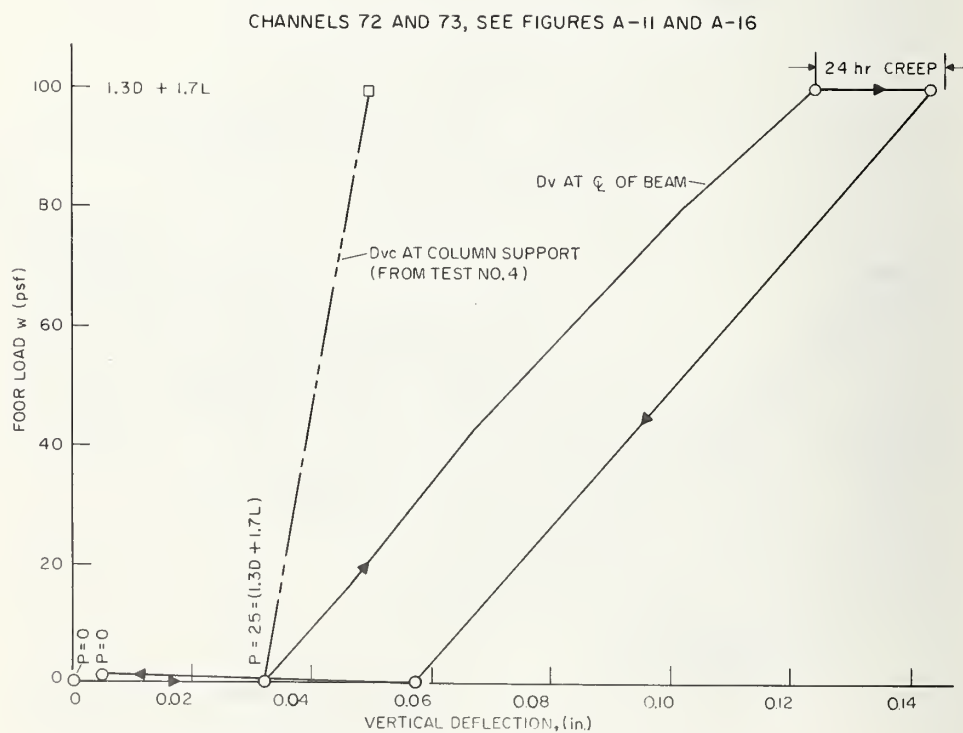
FIGURE 9.1. Test No. 5, sustained vertical load versus
midspan deflection of center main beam.
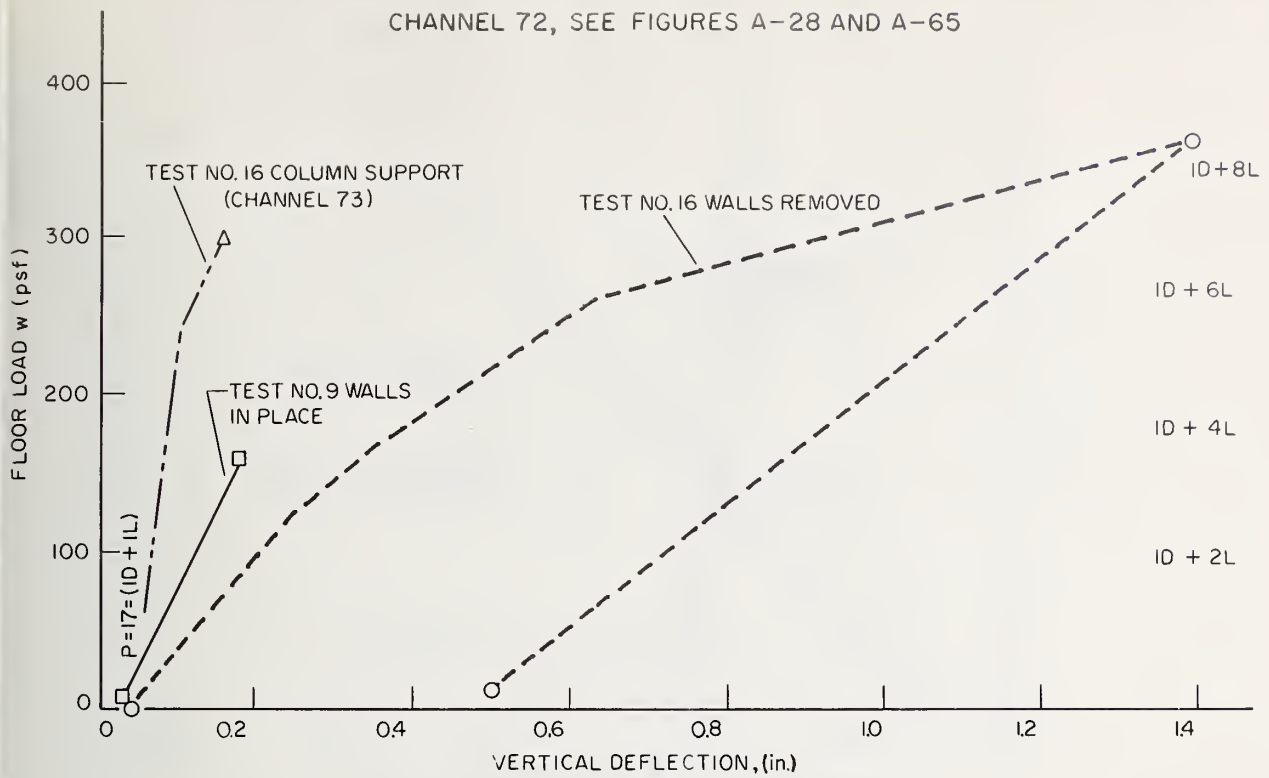
FIGURE 9.2. *Test No. 16, major floor load* (w) *versus midspan deflection of center main beam.*
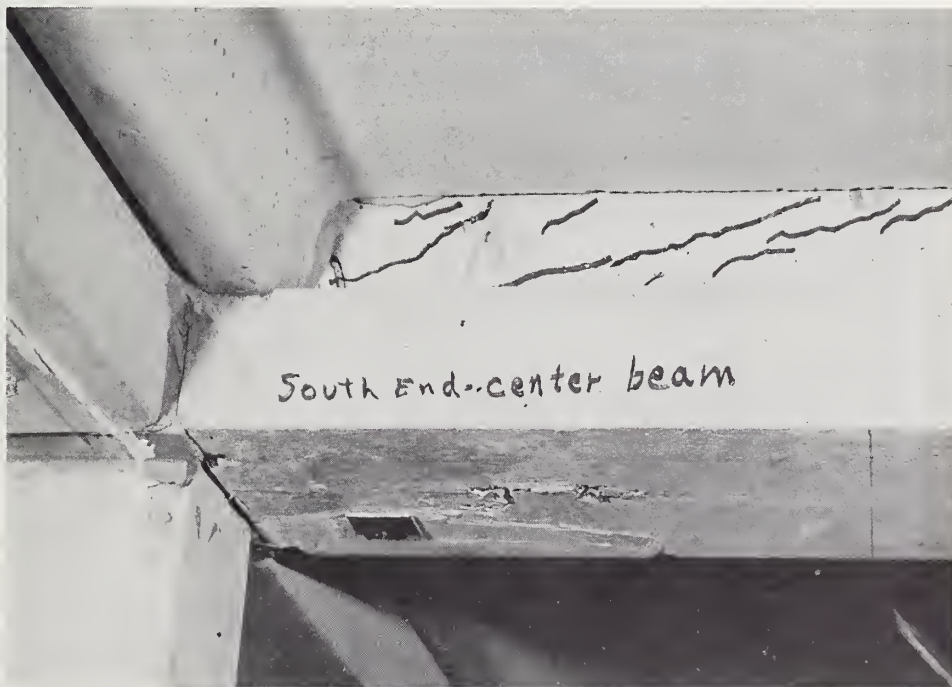
FIGURE 9.3. *Diagonal tension cracking at* w = 270 psf *(Test No. 16).*
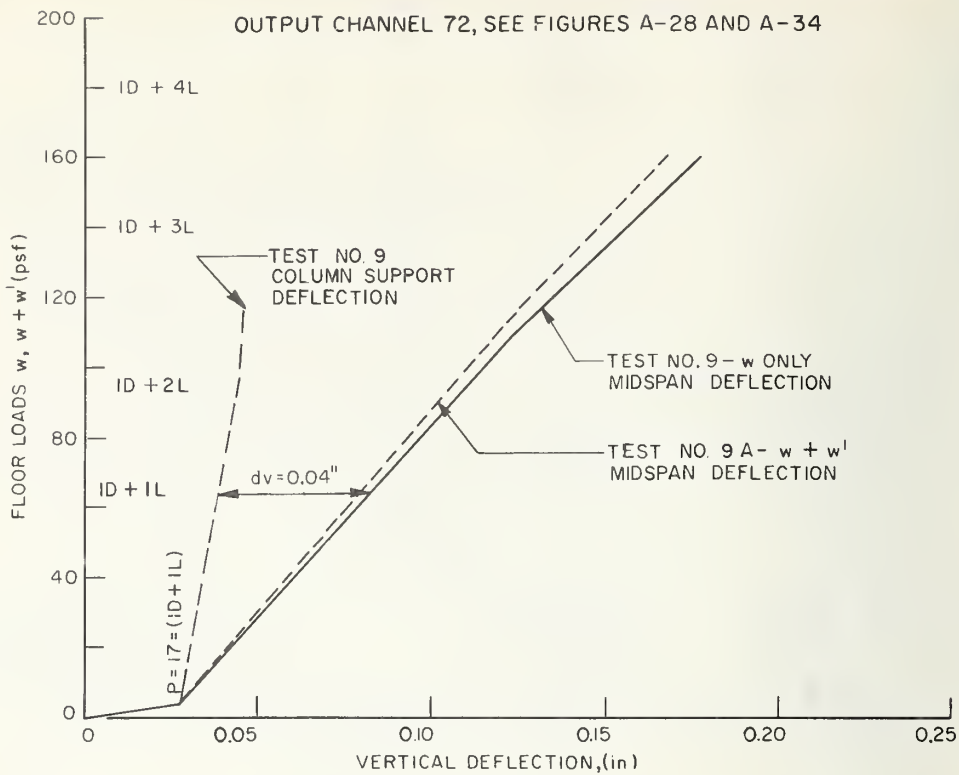
FIGURE 9.4. *Interior floor load* (w) *and total floor load* (w + w') *versus midspan and column support deflection of center main beam.*
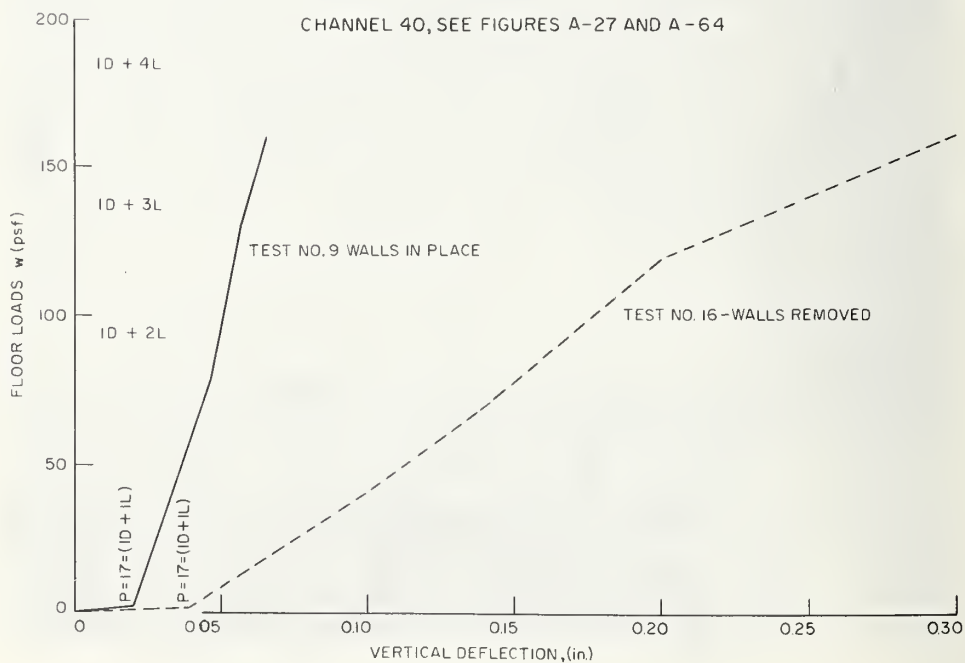
FIGURE 9.5. *Midspan deflection of west main beam with and without walls.*
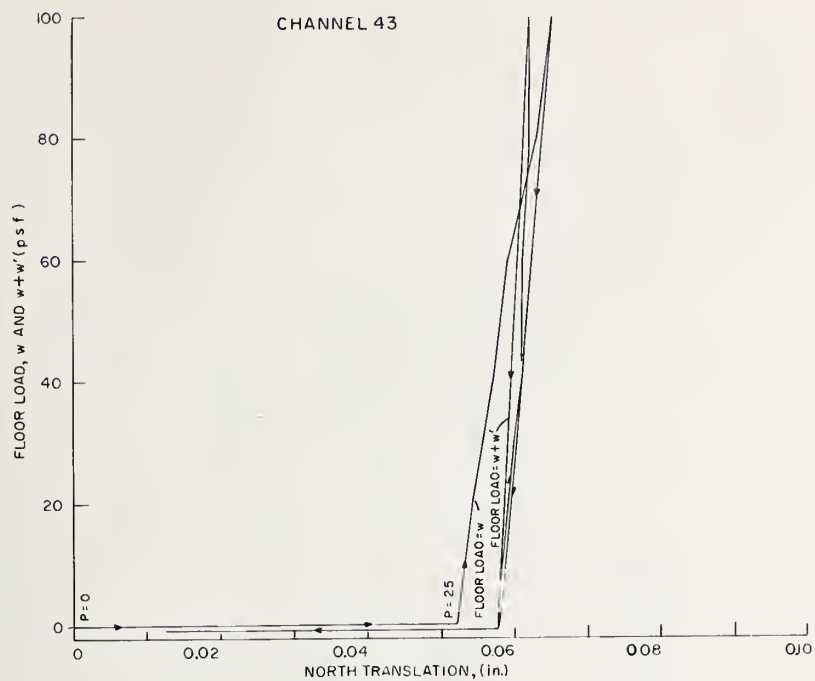
FIGURE 9.6. *Tests No. 12A and 13A, vertical load versus north translation.*
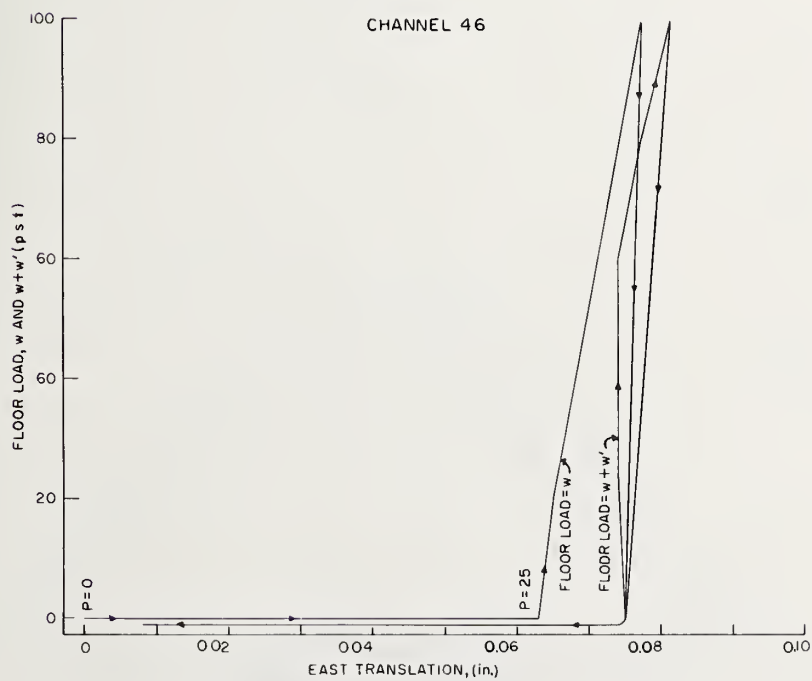
FIGUGE 9.7. *Tests No. 12 and 13, vertical load versus east translation.*
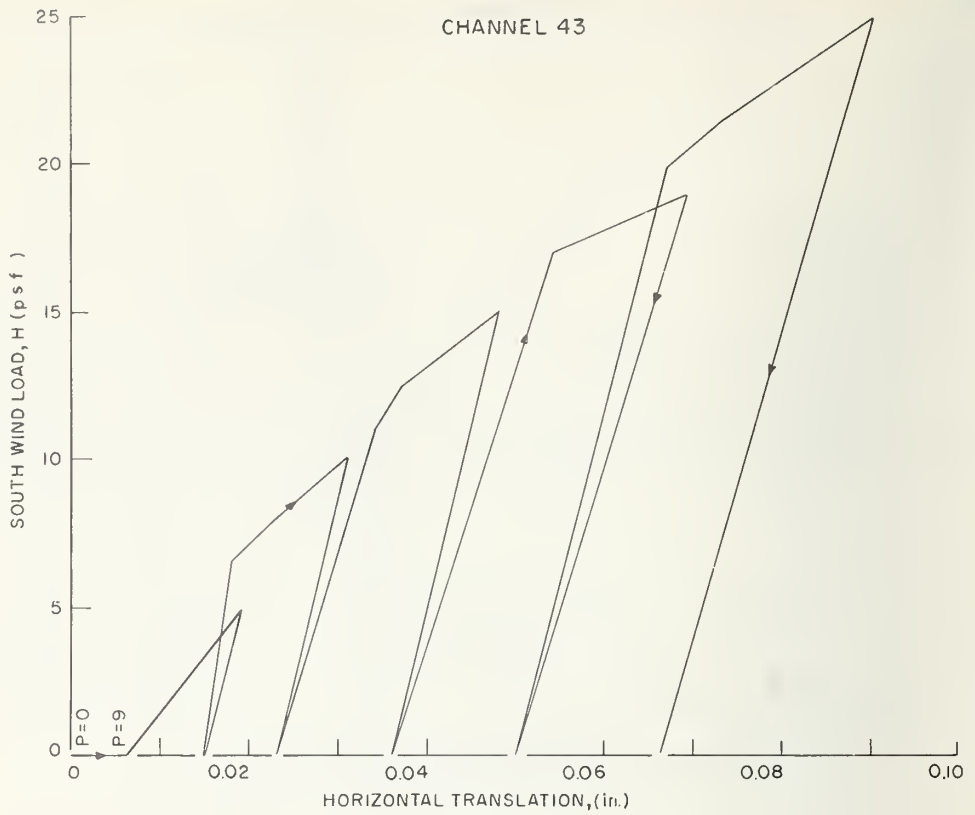
FIGURE 9.8. *Test No. 2, south wind load versus translation.*
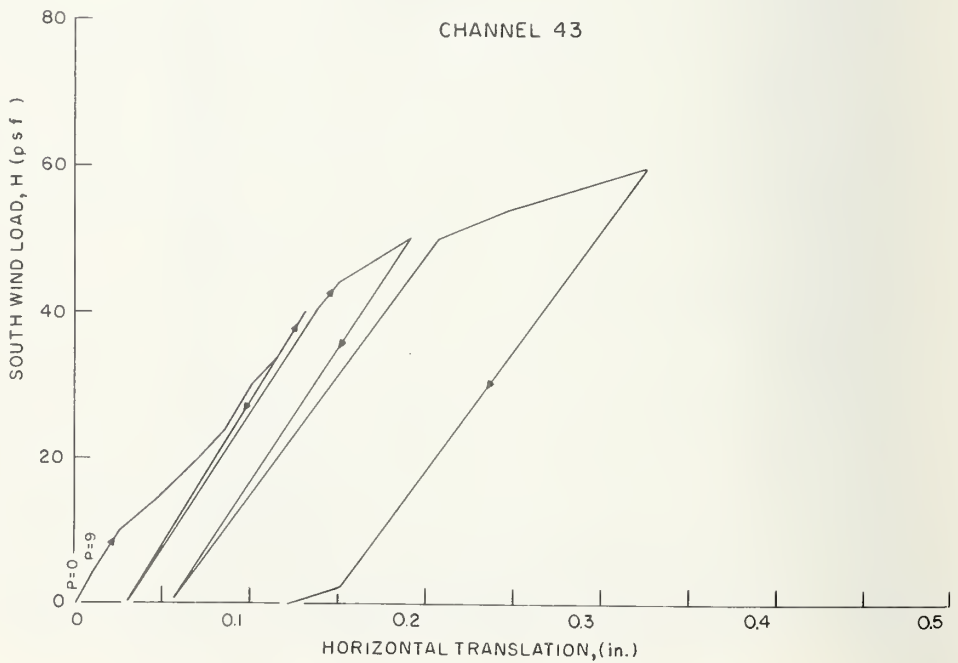
FIGURE 9.9. *Test No. 10, south wind load versus translation.*
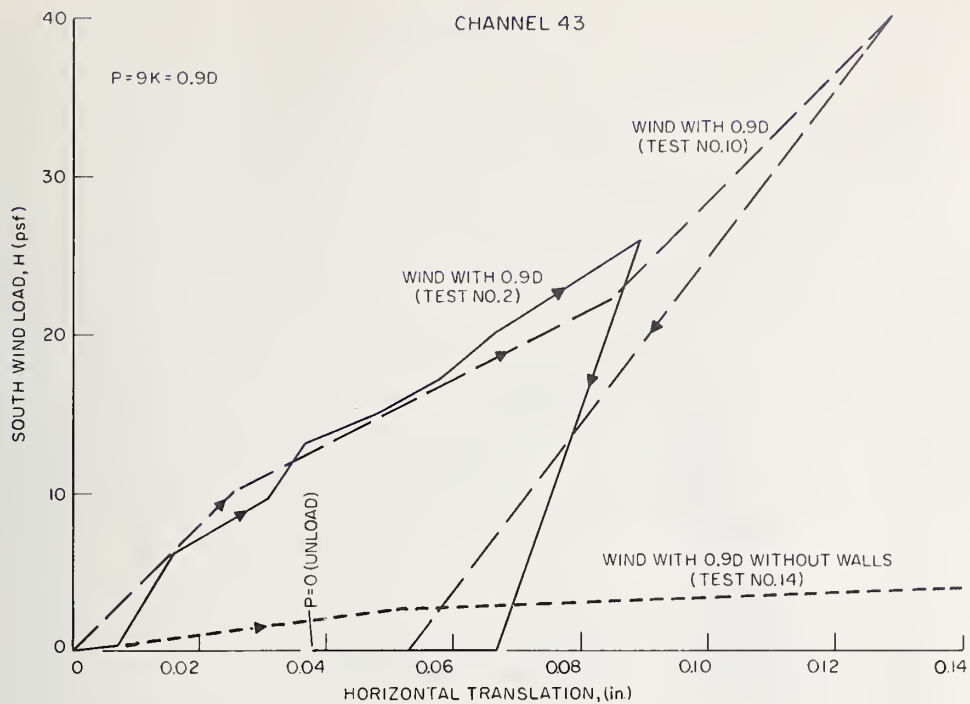
FIGURE 9.10. *North-south horizontal translation of structure with and without walls.*
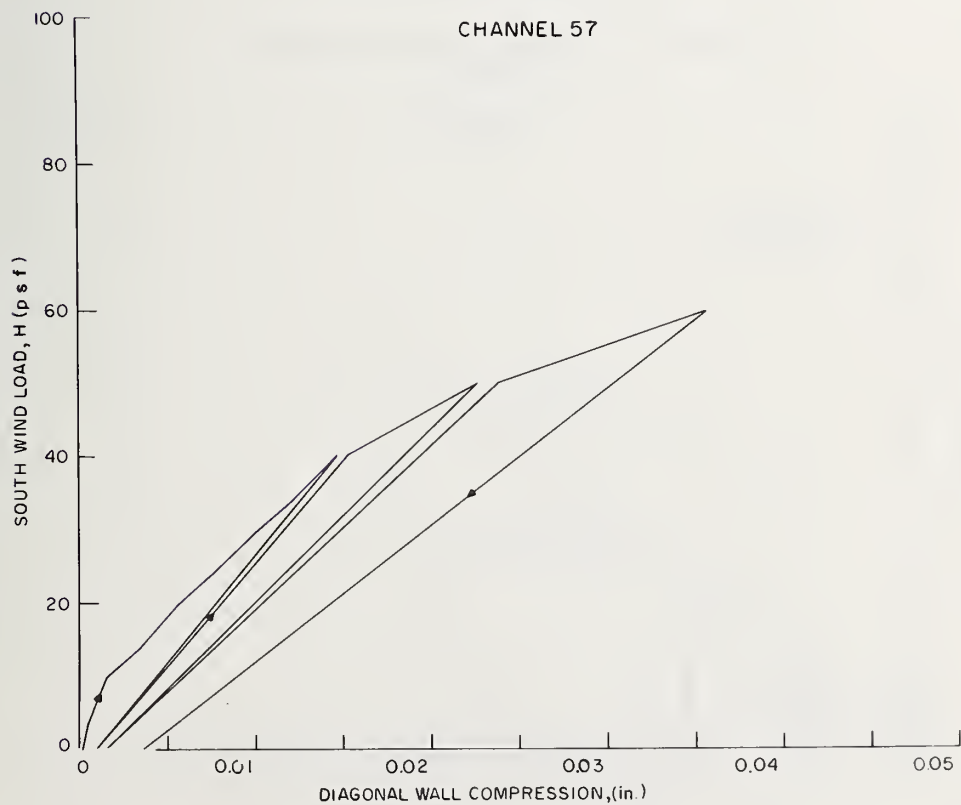
FIGURE 9.11. *Test No. 11. south wind load versus wall compression.*

53
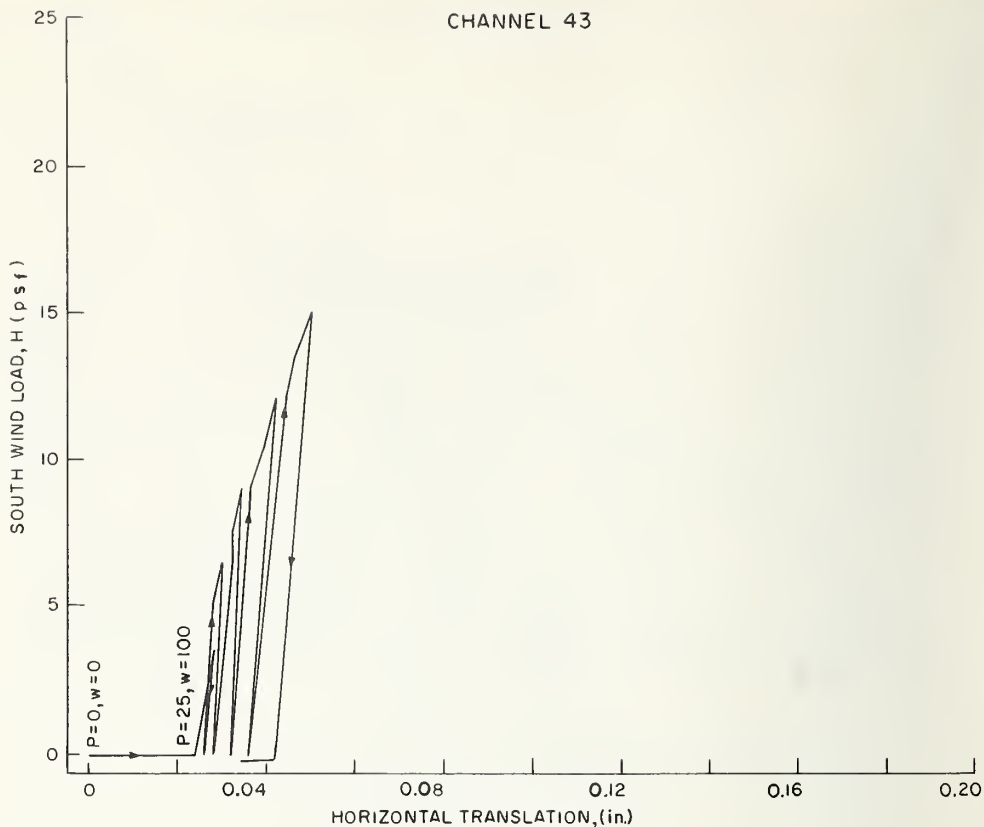
FIGURE 9.12. *Test No. 7, south wind load versus translation.*
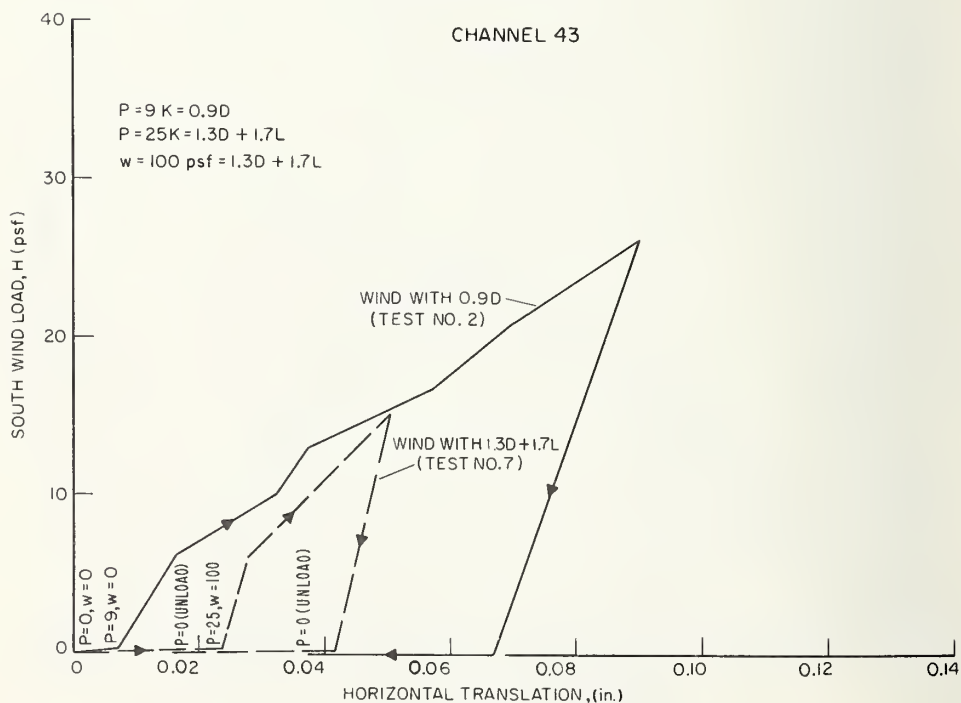
FIGURE 9.13. *Effect of vertical loads on north-south horizontal translation.*
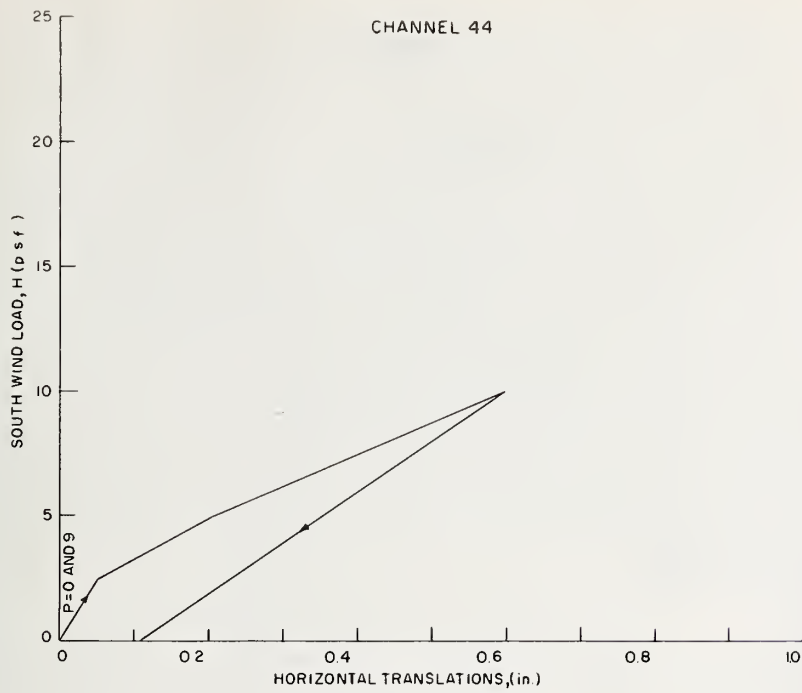
FIGURE 9.14. *Test No. 14, south wind load versus translation.*
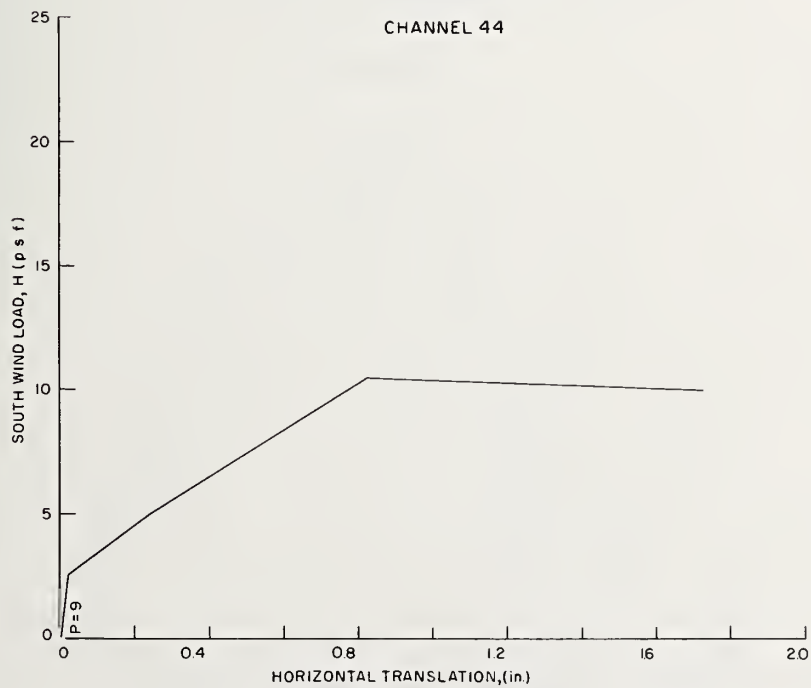
FIGURE 9.15. *Test No. 18, south wind load versus translation.*
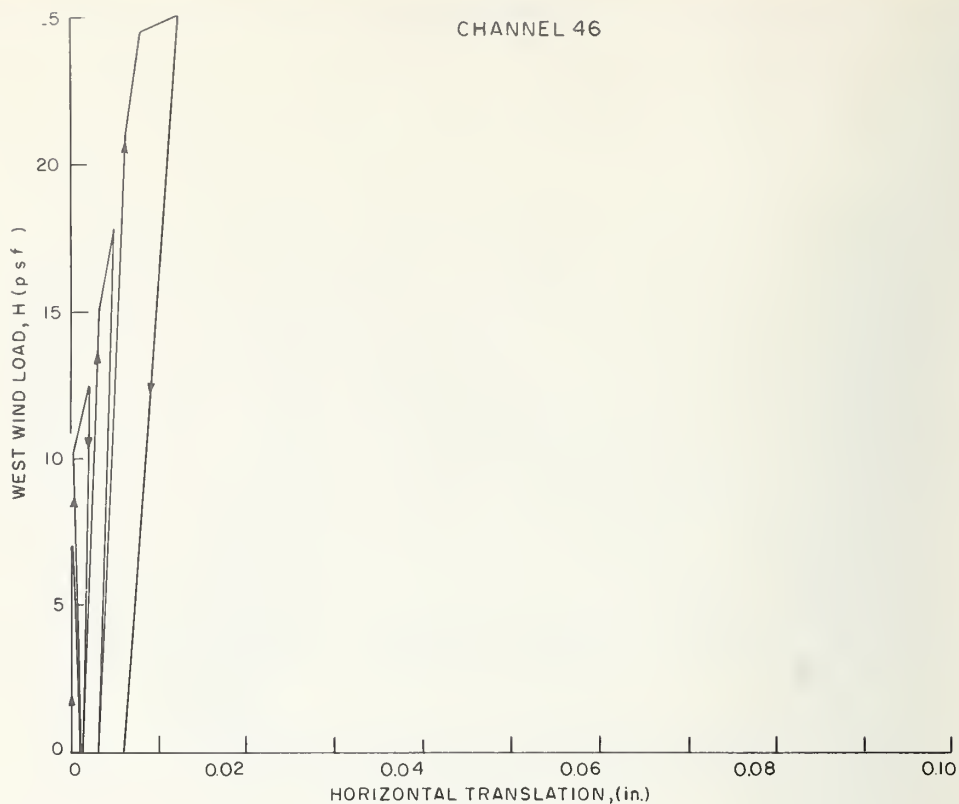
FIGURE 9.16. *Test No. 3, west wind load versus translation.*
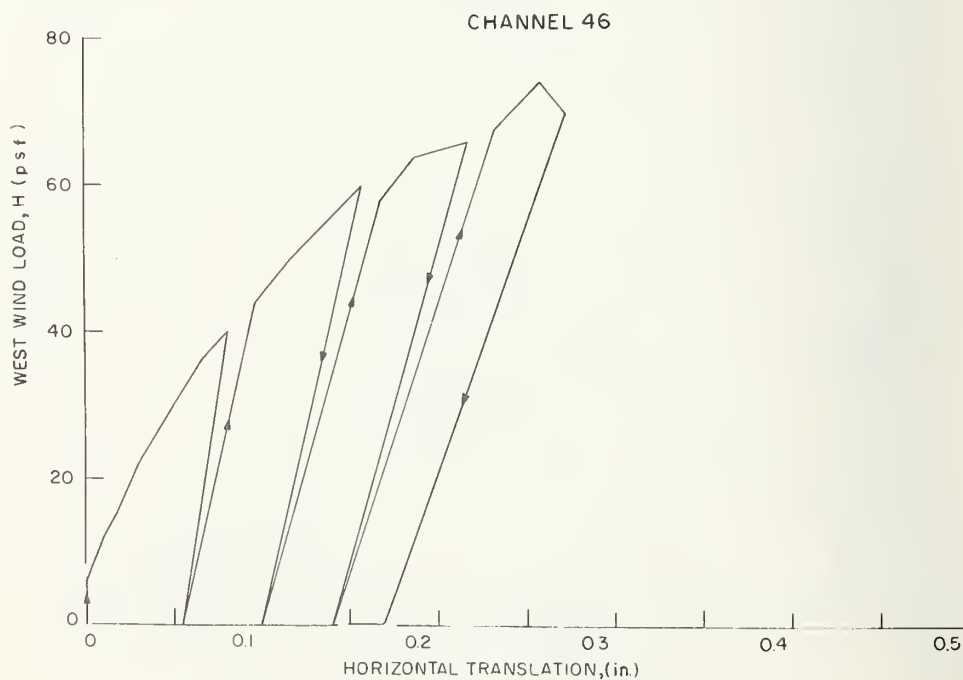
FIGURE 9.17. *Test No. 11, west wind load versus translation.*
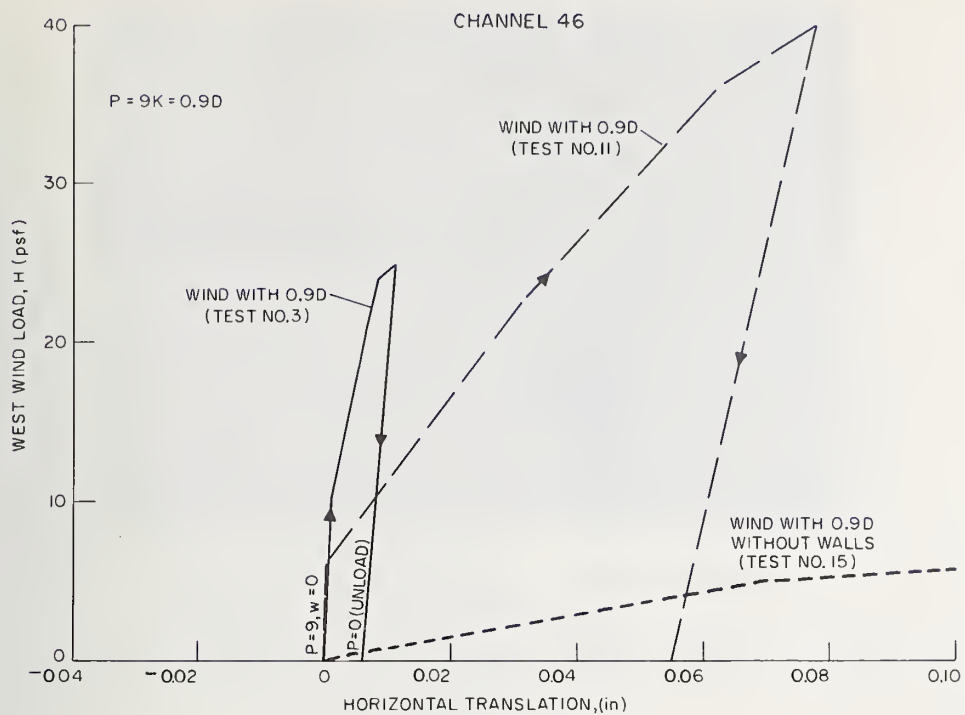
FIGURE 9.18. *East-west horizontal translation of structure with and without walls.*
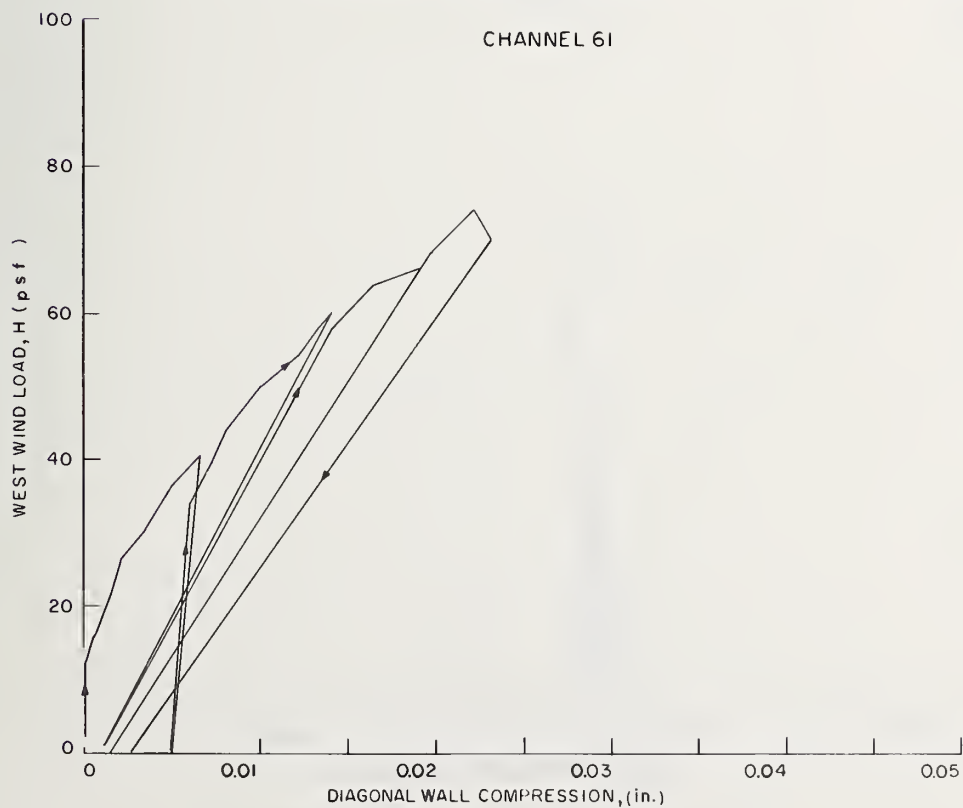
FIGURE 9.19. *Test No. 11, west wind load versus wall compression.*
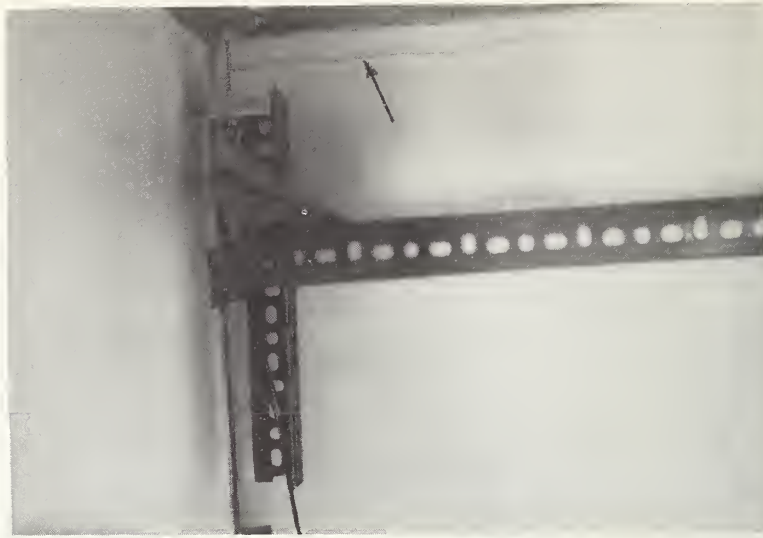
FIGURE 9.20. *Drywall crack near ceiling on interior side of each wall.*
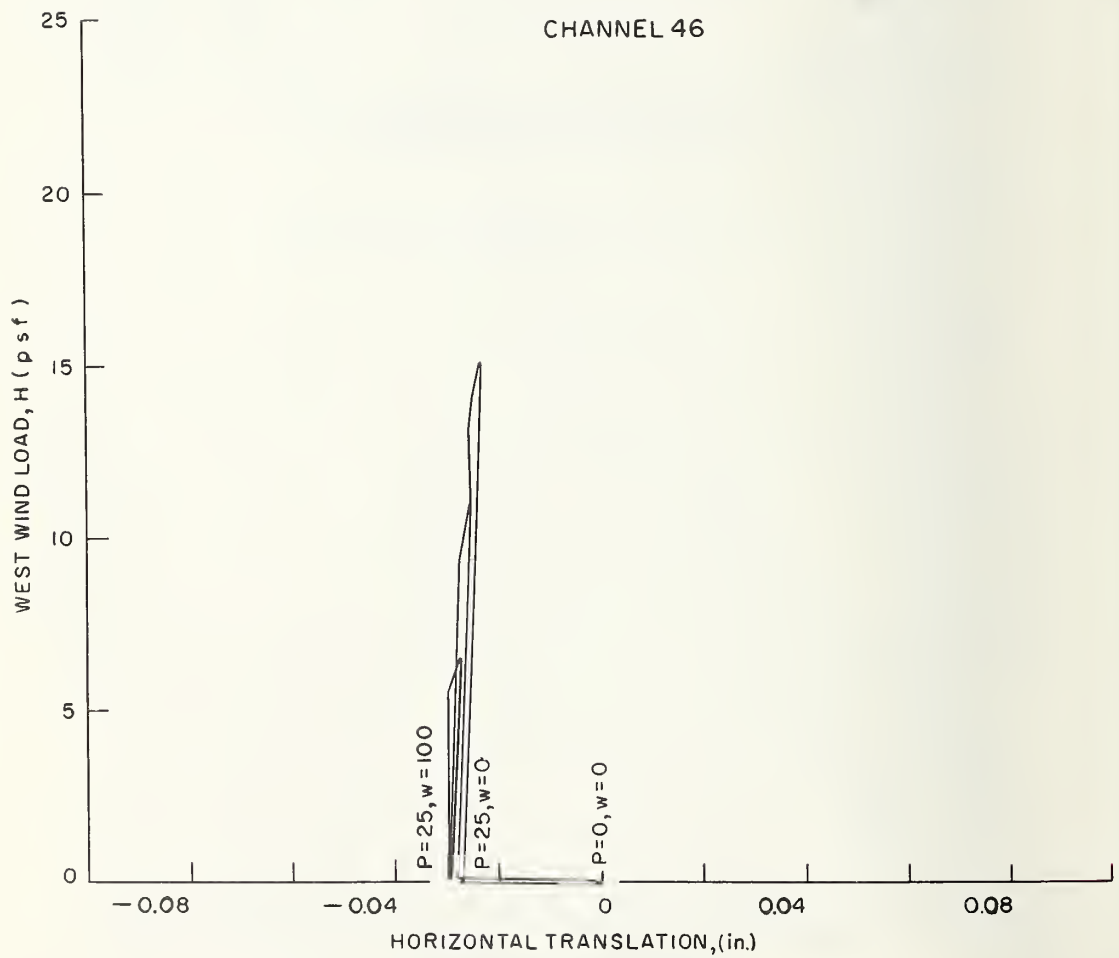
FIGURE 9.21. *Test No. 8, west wind load versus translation.*
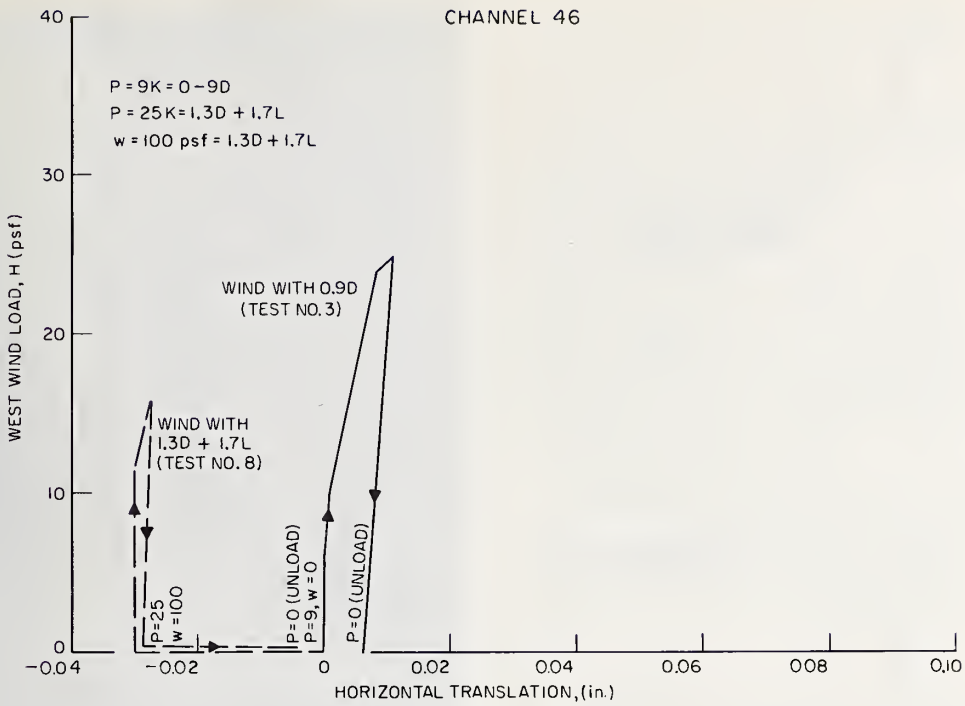
FIGURE 9.22. *Effect of vertical loads on east-west horizontal translation.*
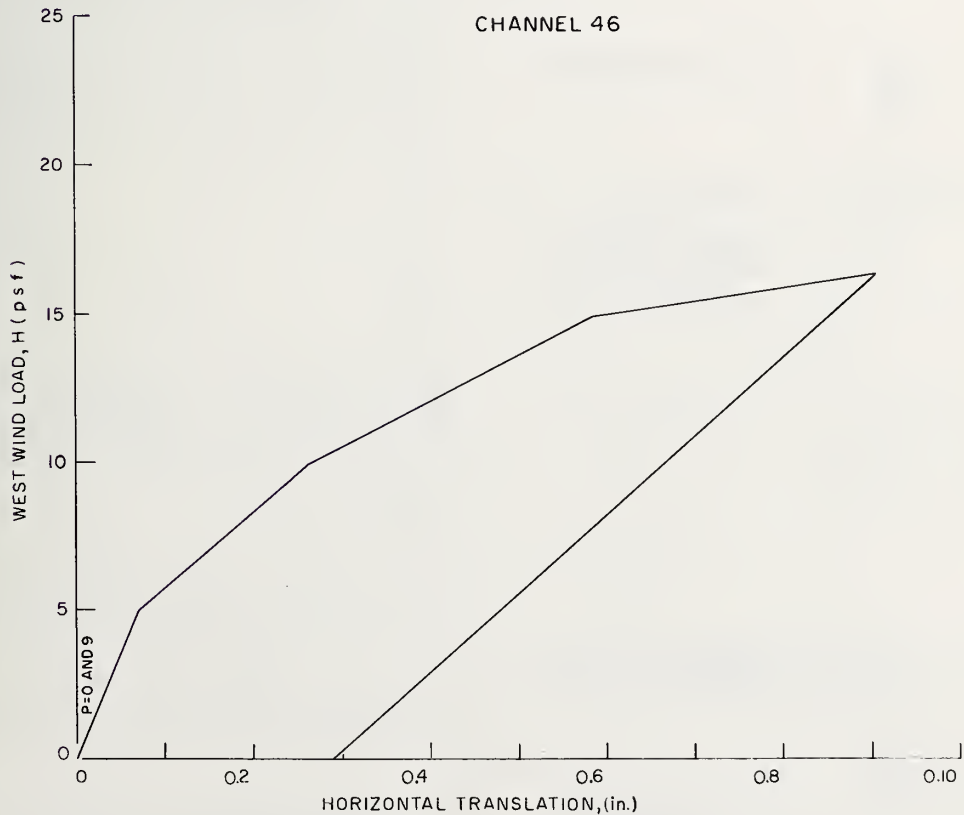
FIGURE 9.23. *Test No. 15, west wind load versus translation.*
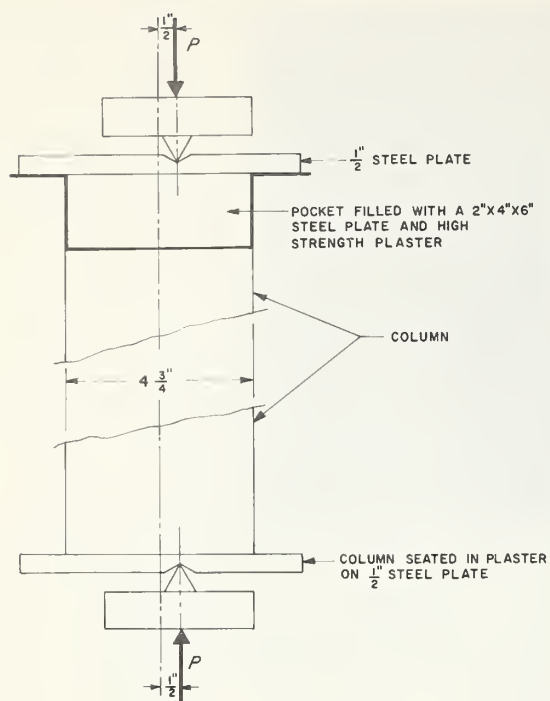
FIGURE 10.1. *Test method for column tests (minor axis).*
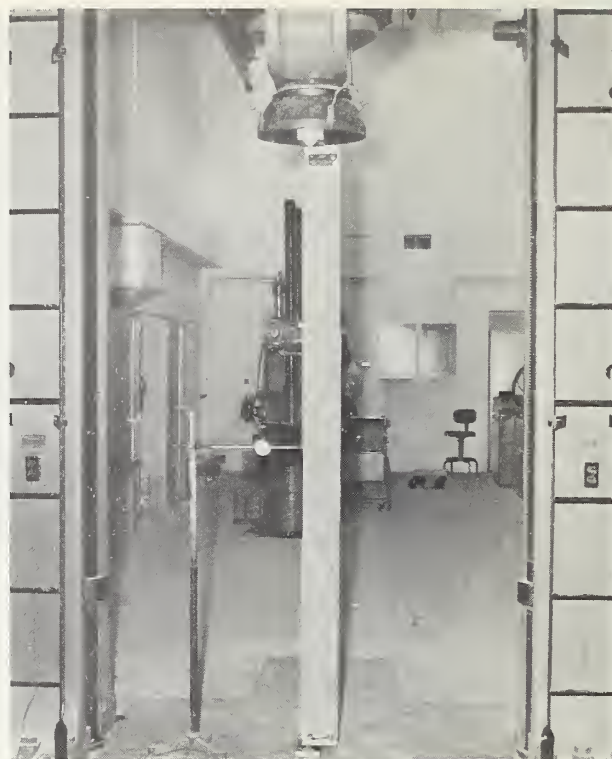
FIGURE 10.3. *Column under test.*
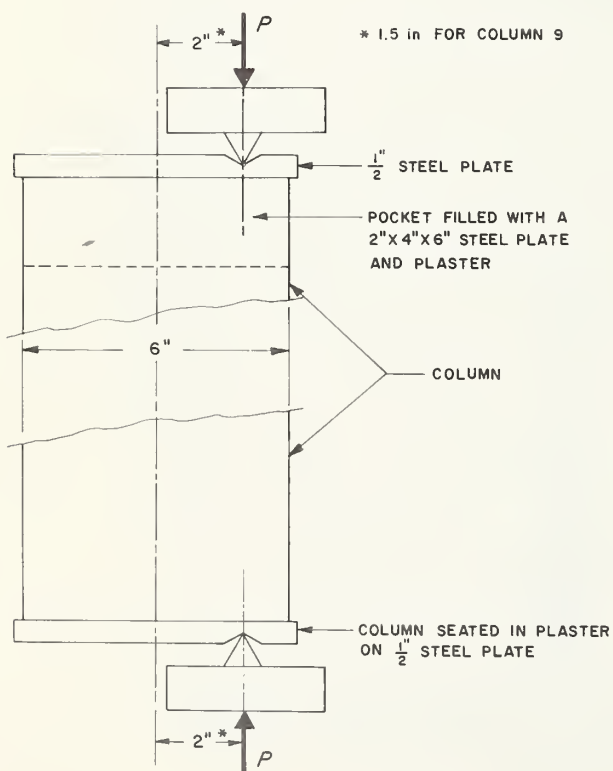
FIGURE 10.2. *Test method for column tests (major axis).*
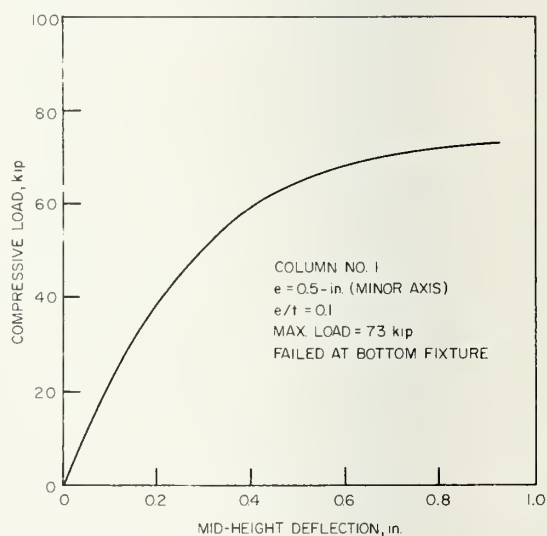
FIGURE 10.4. *Short-term test on column No. 1.*
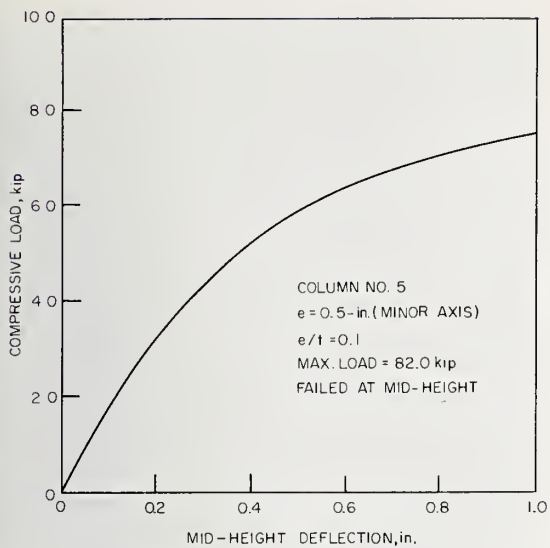
FIGURE 10.5. *Short-term test on column No. 5.*
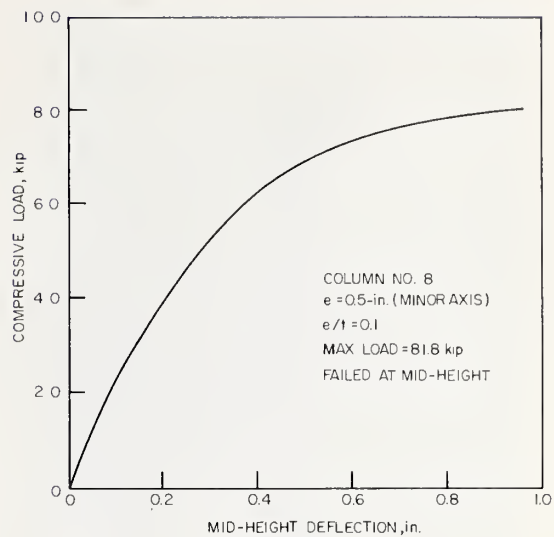
FIGURE 10.6. *Shirt-term test on column No. 8.*
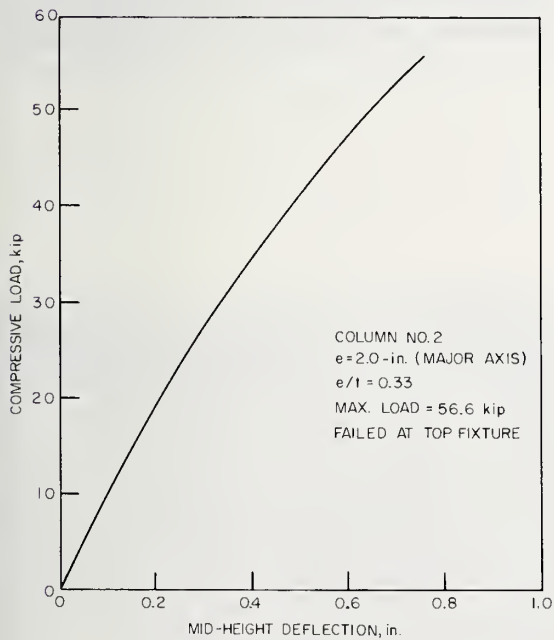
FIGURE 10.7. *Short-term test on column No. 2.*
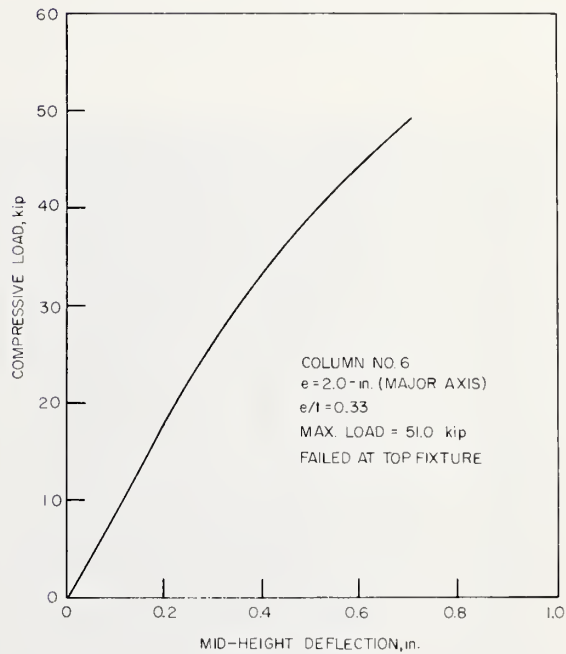
FIGURE 10.8. *Short-term test on column No. 6.*
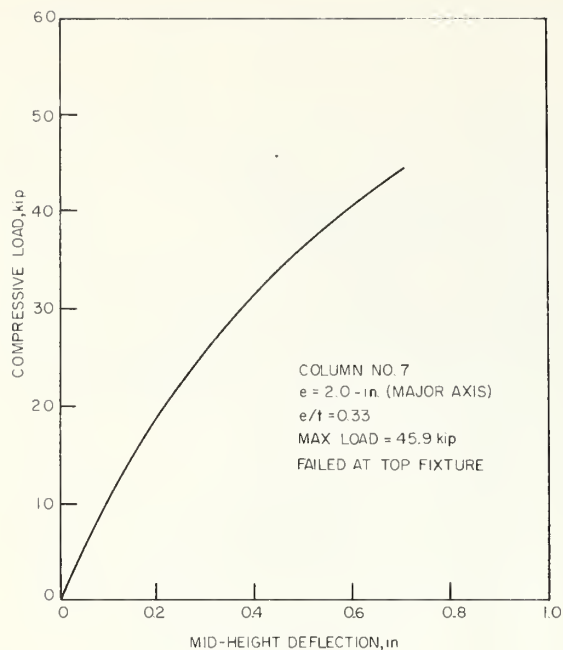
FIGURE 10.9. *Short-term test on column No. 7.*
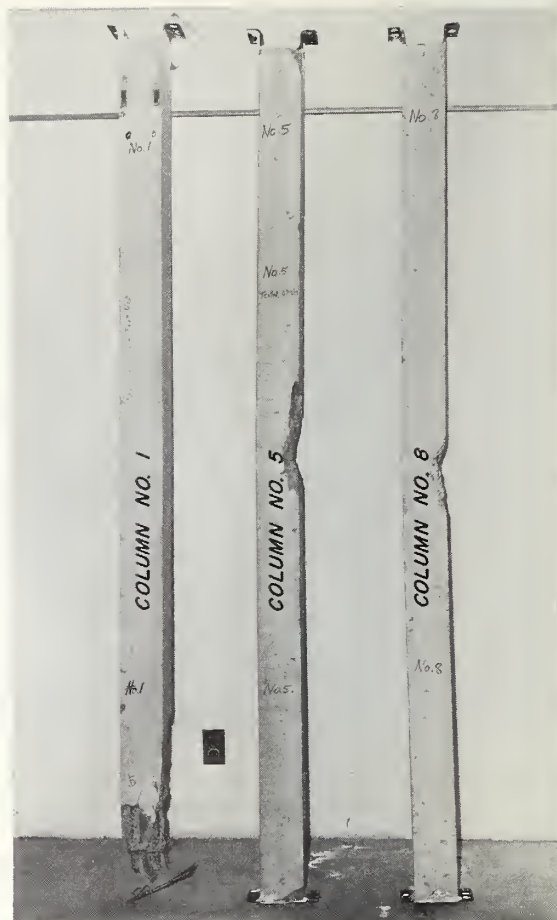
FIGURE 10.11. *Columns after testing.*
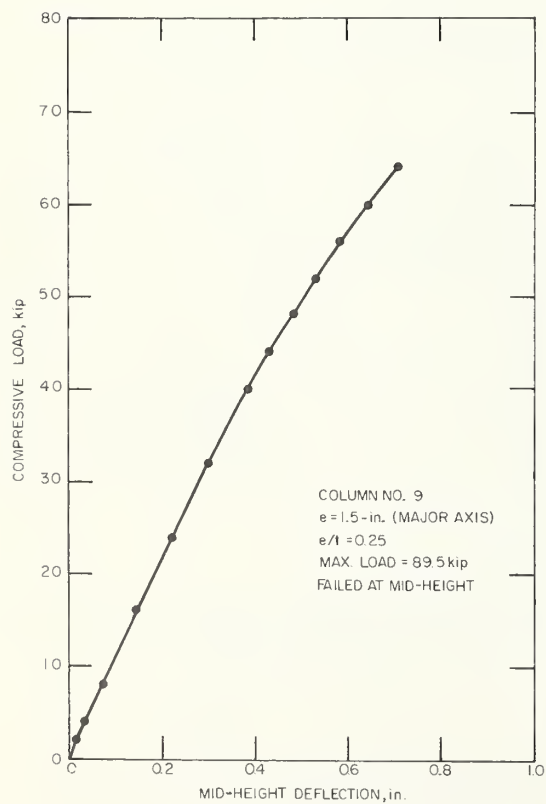
FIGURE 10.10. *Short-term test on column No. 9.*
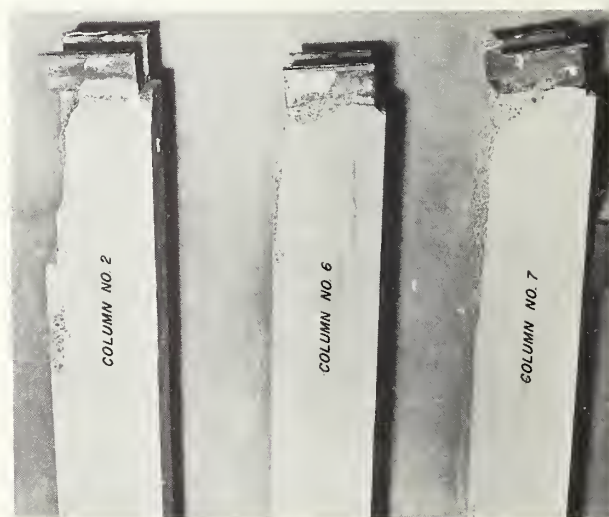
FIGURE 10.12. *Column ends after testing.*
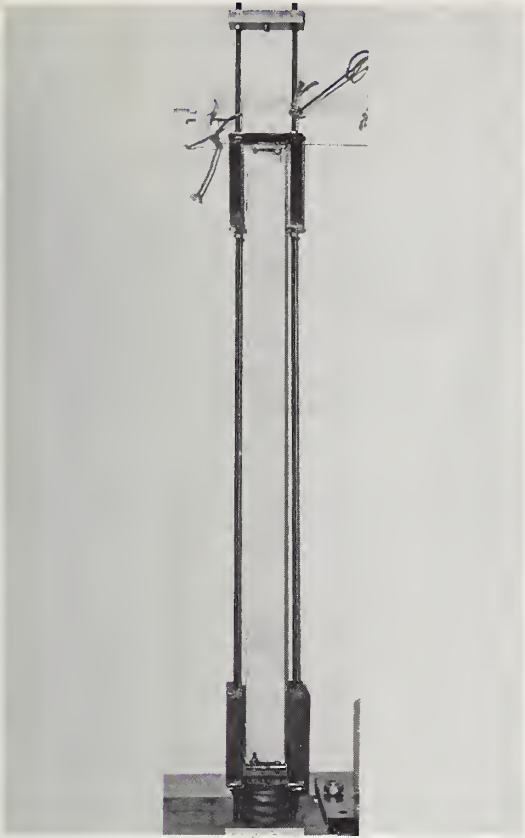
FIGURE 10.13. *Creep loading fame for column No. 3 minor axis bending.*
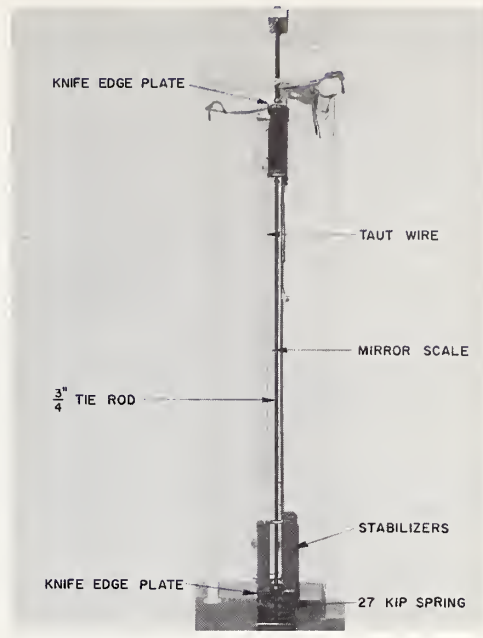
FIGURE 10.14. *Creep loading frame for column No. 4 major axis bending.*

KNIFE EDGE PLATE

TAUT WIRE

MIRROR SCALE

$\frac{3}{4}$" TIE ROD

STABILIZERS

KNIFE EDGE PLATE

27 KIP SPRING

FIGURE 10.15. *Detail of base for column No. 4 creep frame.*
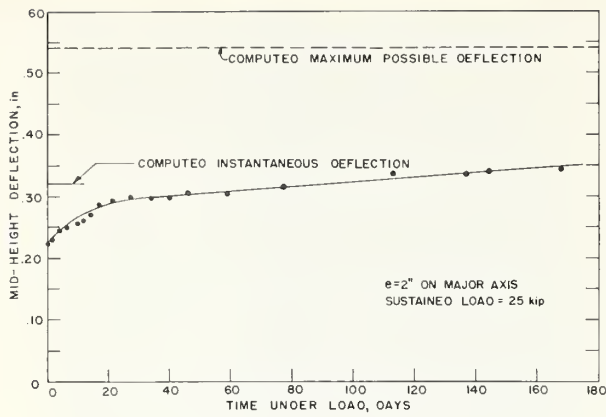
FIGURE 10.16. *Sustained load test, column No. 4.*
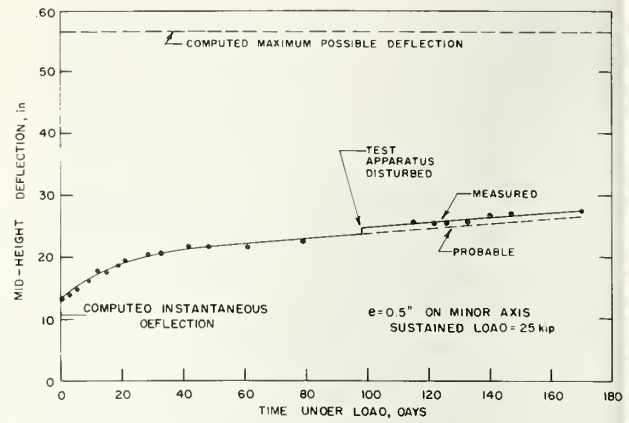
FIGURE 10.17. *Sustained load test, column No. 3.*
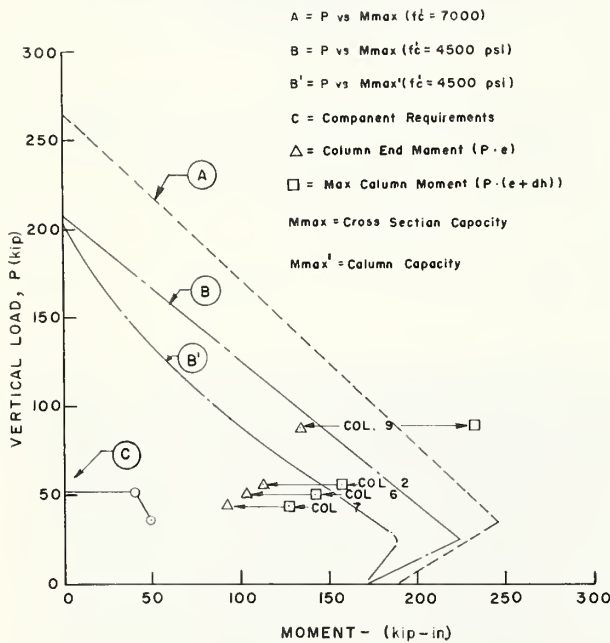
FIGURE 10.18. *Columns with major axis eccentricity.*
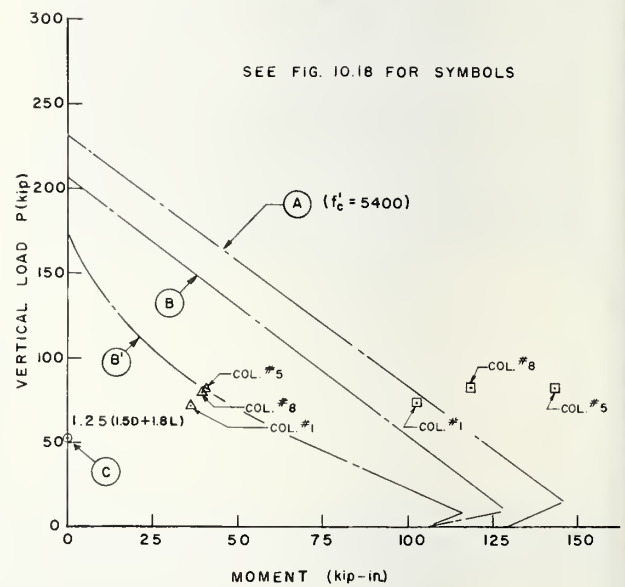
FIGURE 10.19. *Columns with minor axis eccentricity.*
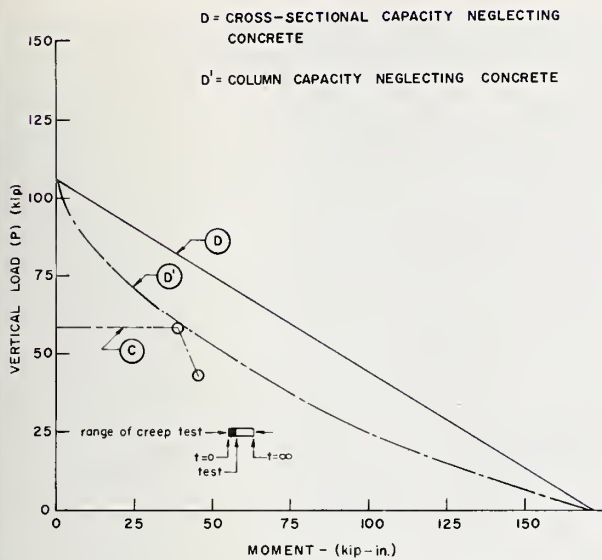
FIGURE 10.20. *Column creep with major axis eccentricity.*
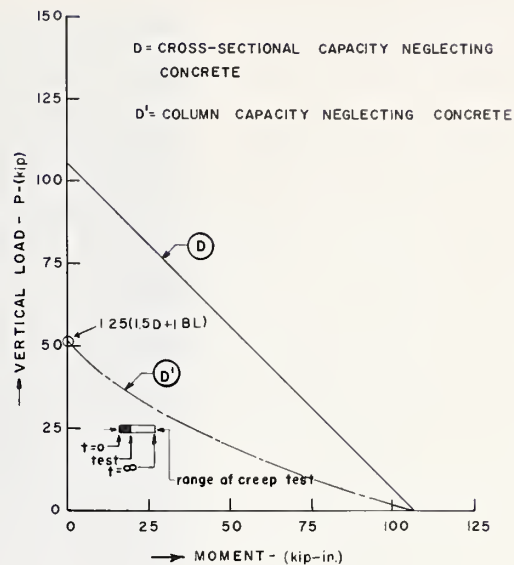
FIGURE 10.21. *Column creep with minor axis eccentricity.*
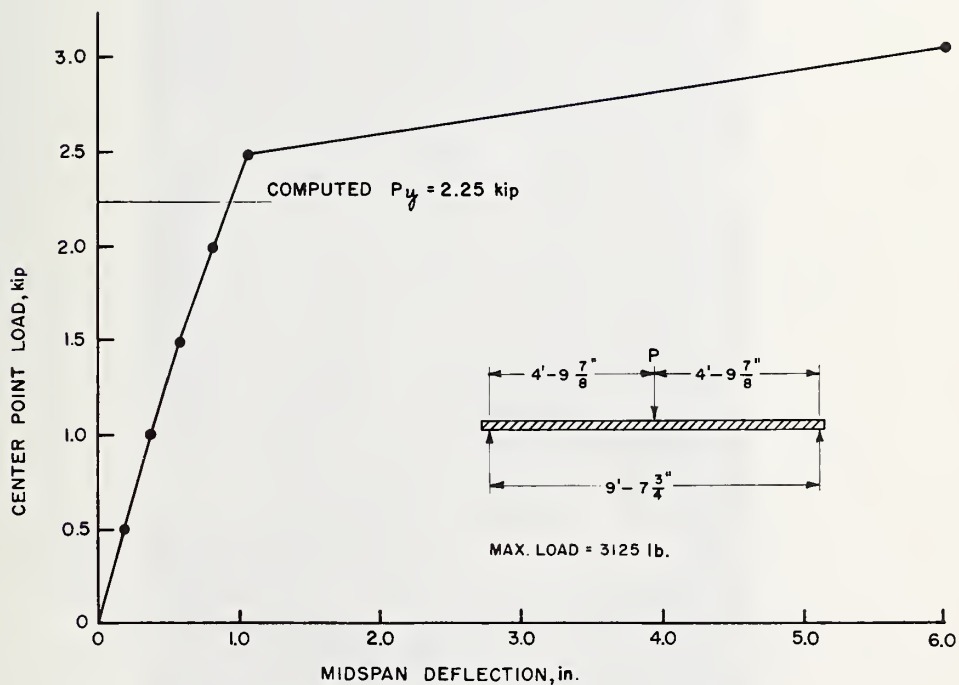
FIGURE 10.22. *Channel roof slab test.*
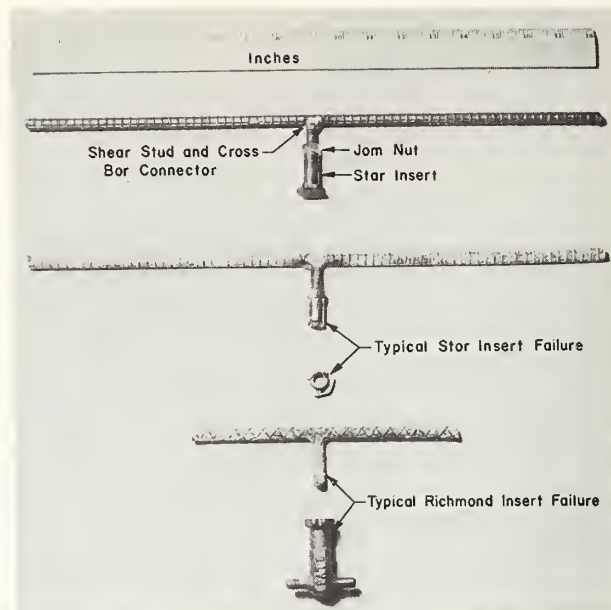
FIGURE 10.23. *Beam to slab shear connectors.*
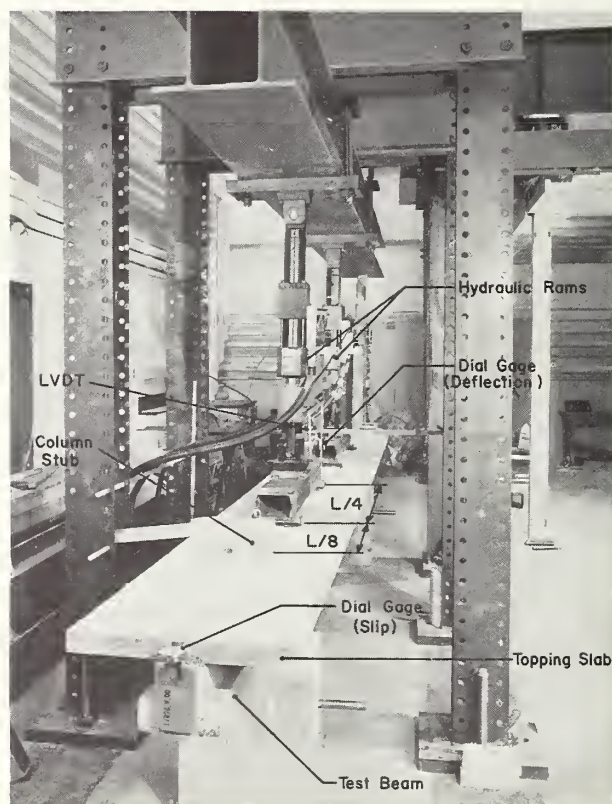
FIGURE 10.24. *Fatigue test on main beam.*

FIGURE 10.25. *Fatigue loading, beam No. 5.*

FIGURE 10.26. *Fatigue loading, beam No. 9.*

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BEAM NO. 8
RICHMOND INSERTS AT 19"
PARTIAL BEAM COLUMN CONNECTION
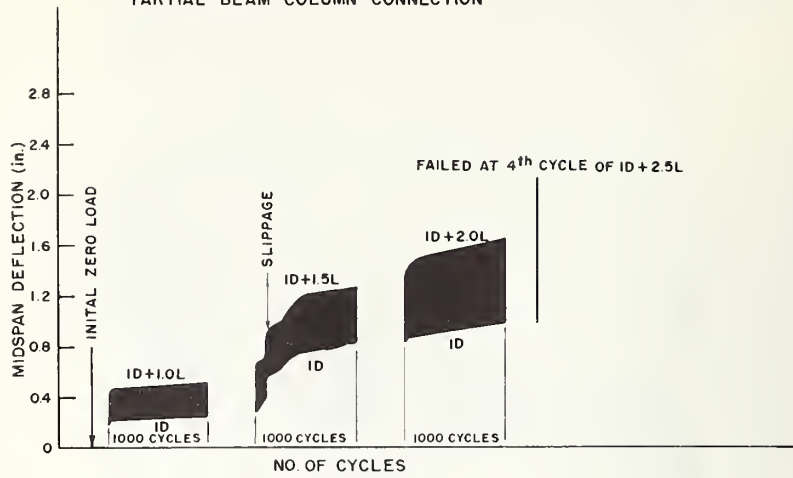
FIGURE 10.27. *Fatigue loading, beam No. 8.*

BEAM NO. 6
RICHMOND INSERTS AT 9½"
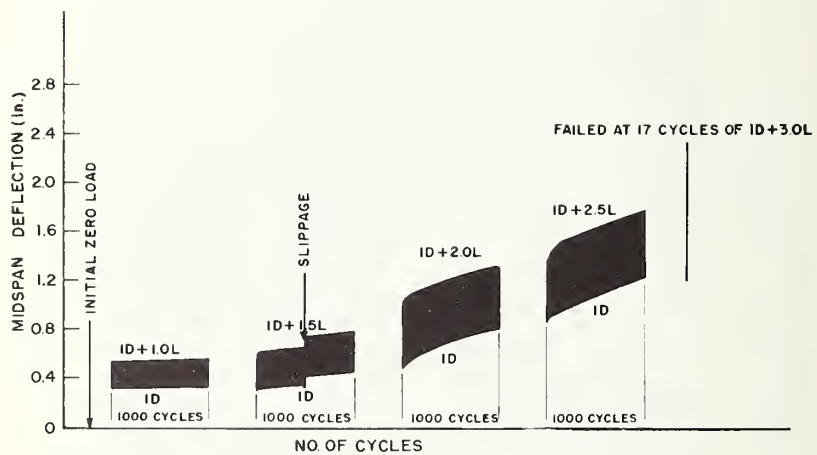
FIGURE 10.28. *Fatigue loading, beam No. 6.*

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FORM PR-221
(7-67)

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
Bureau of Public Roads

REPORT ON SAMPLE OF CONCRETE CYLINDER(6 x 12 inch)

LABORATORY NUMBER	NAME	DATE REPORTED
N-5592	Mr Tom Reichard	5/29/68

IDENTIFICATION MARKS	SUBMITTED BY	
NBS B4-16 No. 4	National Bureau of Standards	

DATE SAMPLED	DATE RECEIVED 5 p.m.	SAMPLED FROM
--	5/24/68	

QUANTITY REPRESENTED	SOURCE OF MATERIAL
--	National Bureau of Standards

LOCATION USED OR TO BE USED	EXAMINED FOR Air (void) content
--	by Linear Traverse (Rosiwal Method)

TEST RESULTS

The concrete specimen, a 6 x 12-inch cylinder, NBS B4-16 No. 4, was received 5/24/68. An inch thick slice was diamond sawed through the vertical axis of the cylinder. One side was ground and lapped for 30 minutes (on a Lapmaster "24" with No. 1950 Lap-master Lapping Compound).   The prepared surface was examined under 112X magnification and tested by the Linear Traverse (Rosiwal) Method (ASTM Designation C 457-66 T).

The data recorded were:

        Number of voids smaller than .02 inches = 2114
        Number of voids larger than .02 inches = 151

        Total number of voids (N) ----------- = 2265
        Total length of traverse (T) -------- = 124.76 inches
        Traverse in inches over voids smaller than .02 inches = 7.83 inches
         "   "   "   "   "   larger than .02   " = 6.22   "
         "   "   "   "   all voids ------------------ =14.05   "

Calculated values:

        Air void content (voids smaller than .02 inches) = 6.3 percent
         "   "   "   "   larger than .02 inches) = 5.0   "
                  Total air void content = 11.3   "

Voids in porous aggregate were not counted.

<u>Note</u> information phoned to Mr. Reichard 4 p.m. 5/29/68

                                    M. C. Gleason

~~Chief, Division of Physical Research~~ Chief, Materials Division	PER M C Gleason

EDITION OF 6-59 OF THIS FORM MAY BE USED

FIGURE 11.1. *Sample report form.*

Appendix A. Plotted Test Results

List of Figures

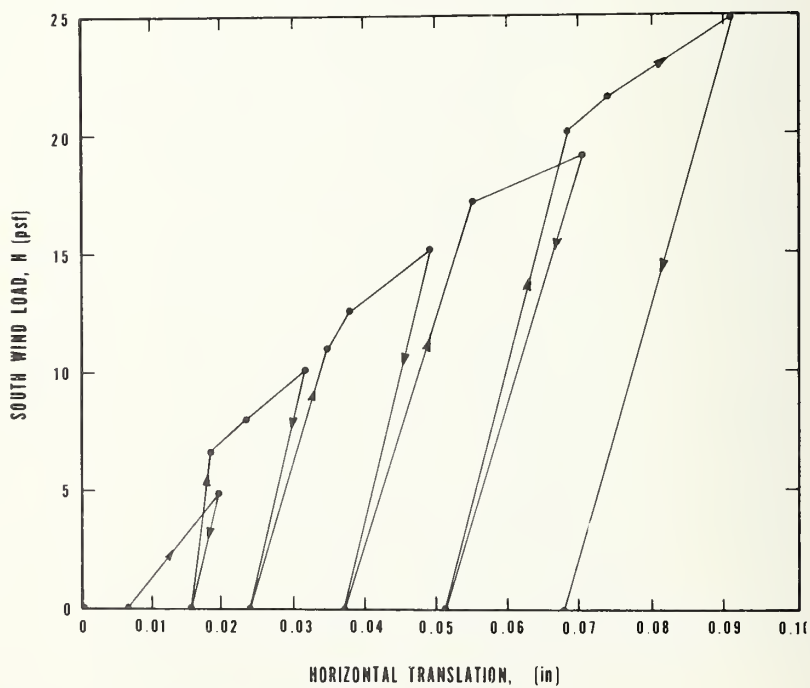FIGURE A. 1. *Test No. 1, column load versus beam deflection. (Output Channel 72)*

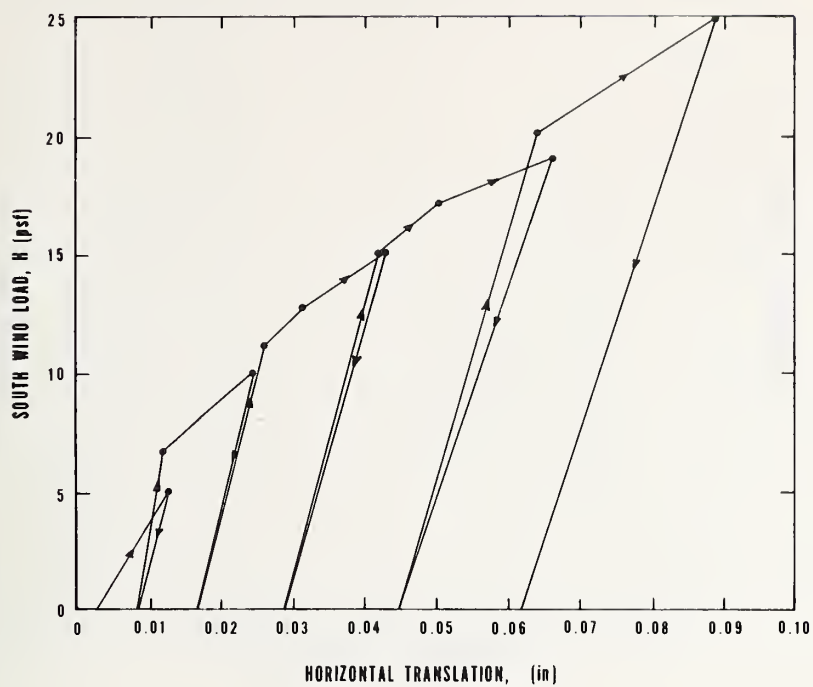FIGURE A. 2. *Test No. 2, south wind load versus translation. (Output Channel 43)*

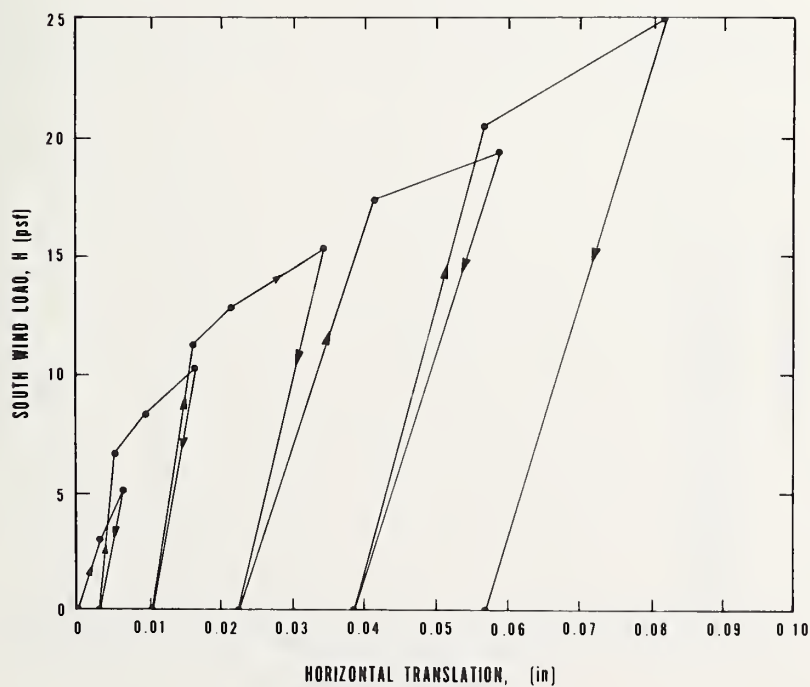FIGURE A. 3. *Test No. 2, south wind load versus translation. (Output Channel 44)*

FIGURE A. 4. *Test No. 2, south wind load versus translation. (Output Channel 45)*
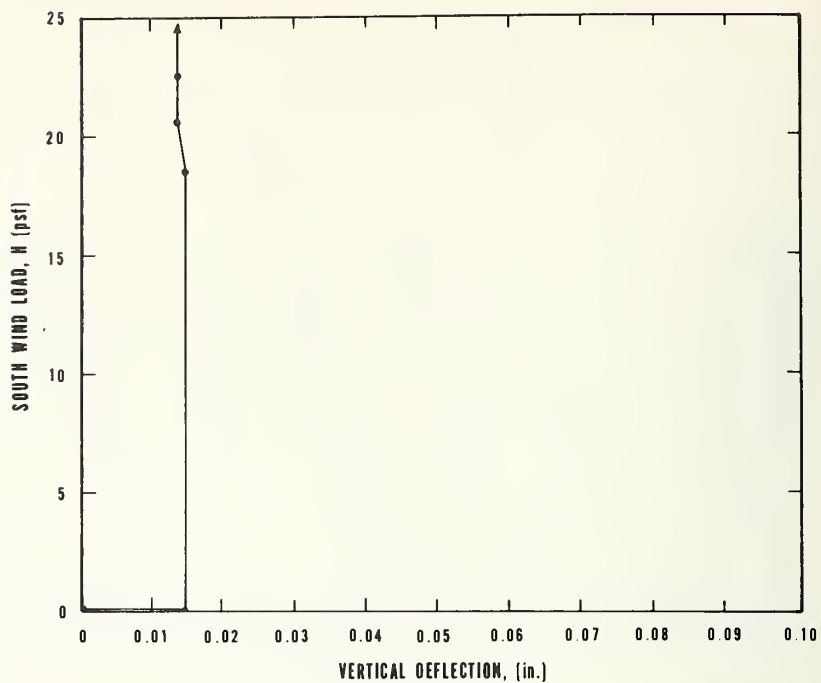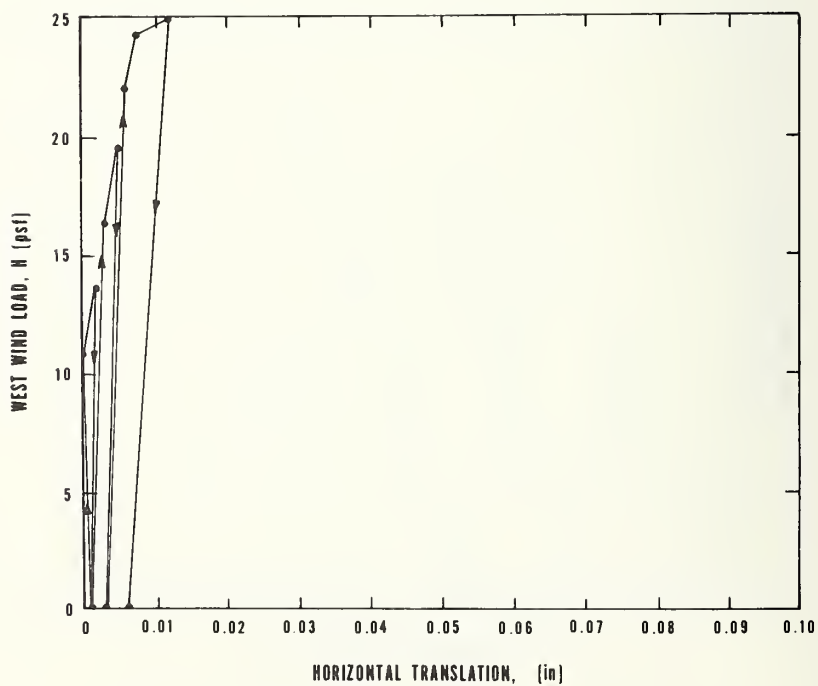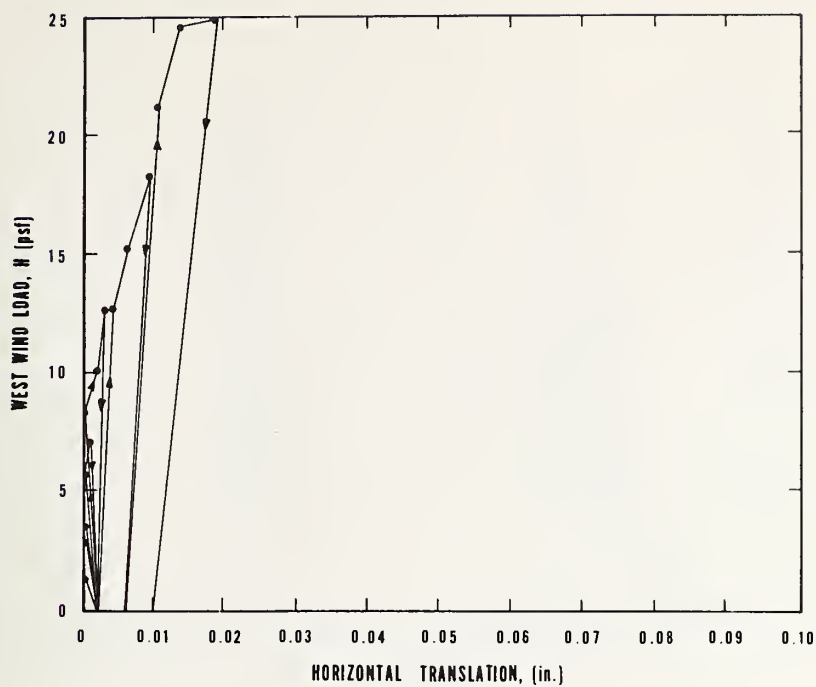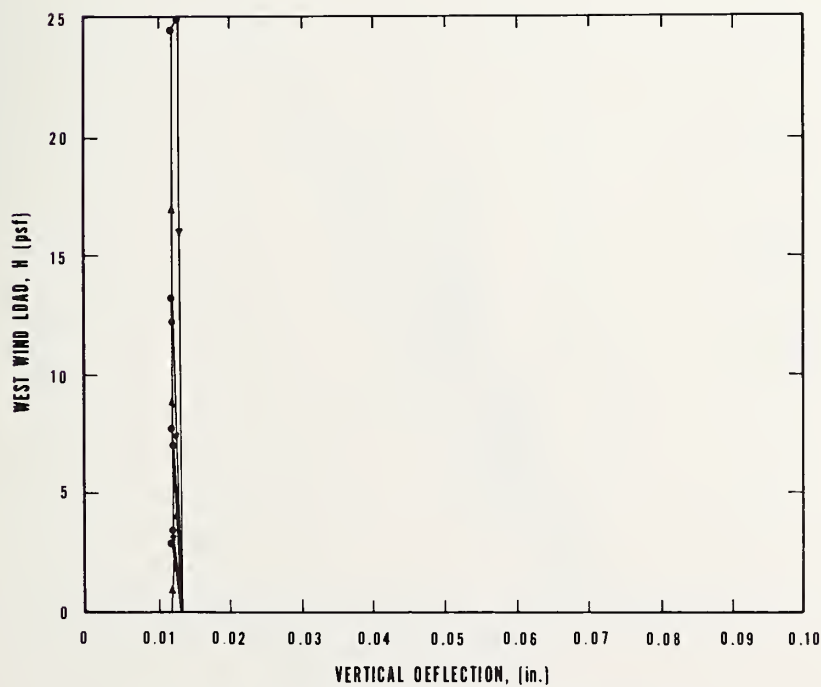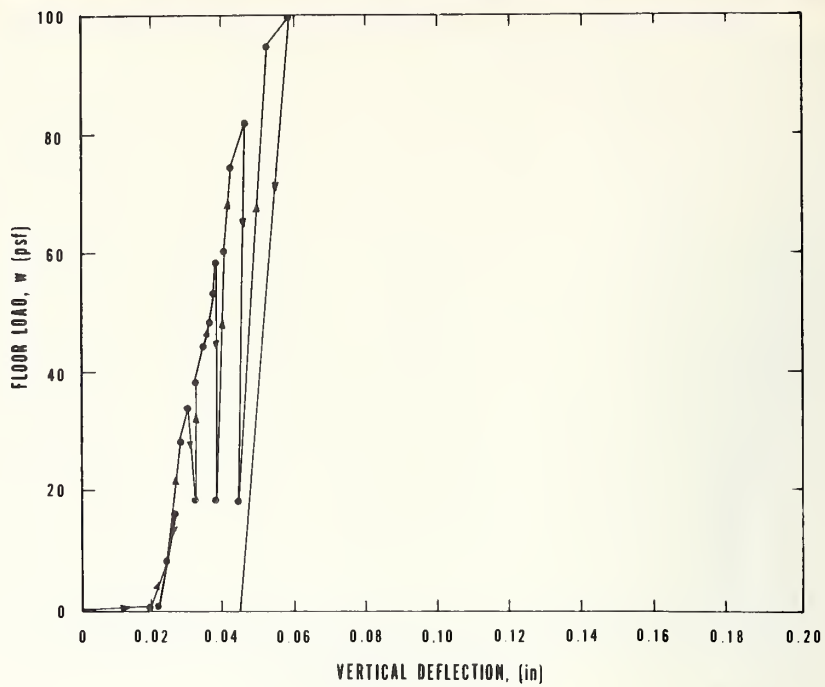
FIGURE A. 5. *Test No. 2, south wind load versus beam deflection. (Output Channel 72)*

FIGURE A. 6. *Test No. 3, west wind load versus translation. (Output Channel 46)*

FIGURE A. 7. *Test No. 3, west wind load versus translation. (Output Channel 47)*

FIGURE A. 8. *Test No. 3, west wind load versus beam deflection. (Output Channel 72)*

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FIGURE A. 9. *Test No. 4, floor load versus beam deflection. (Output Channel 40)*
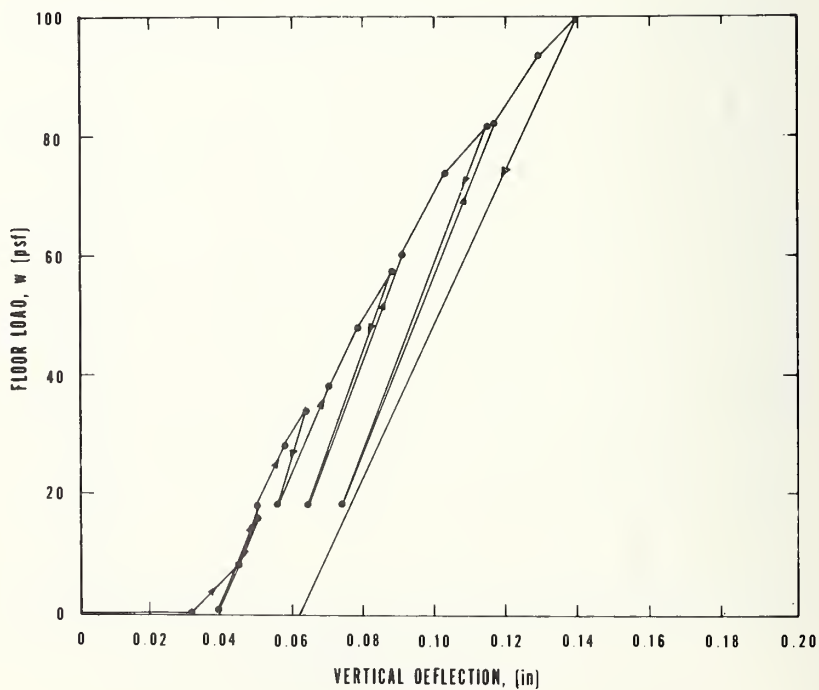
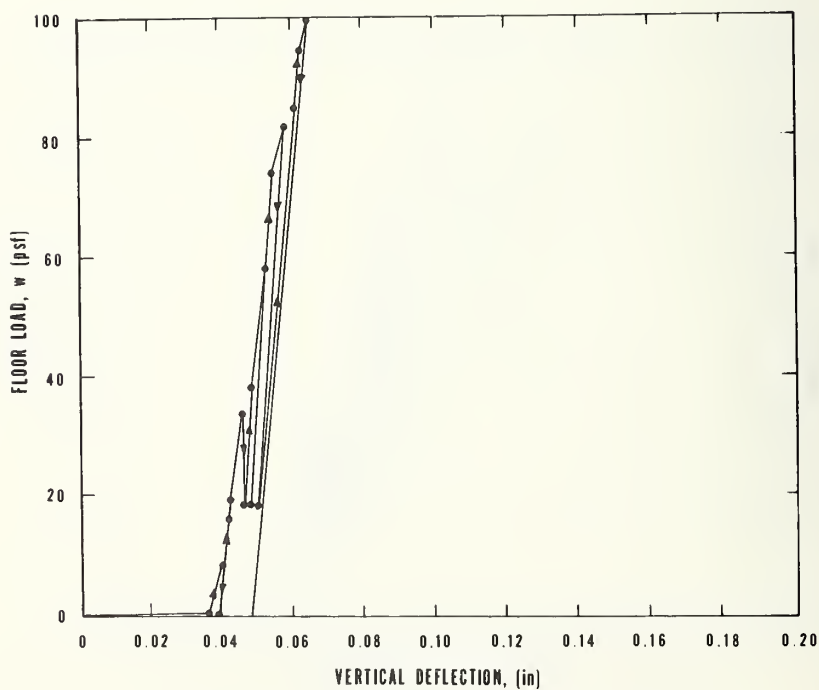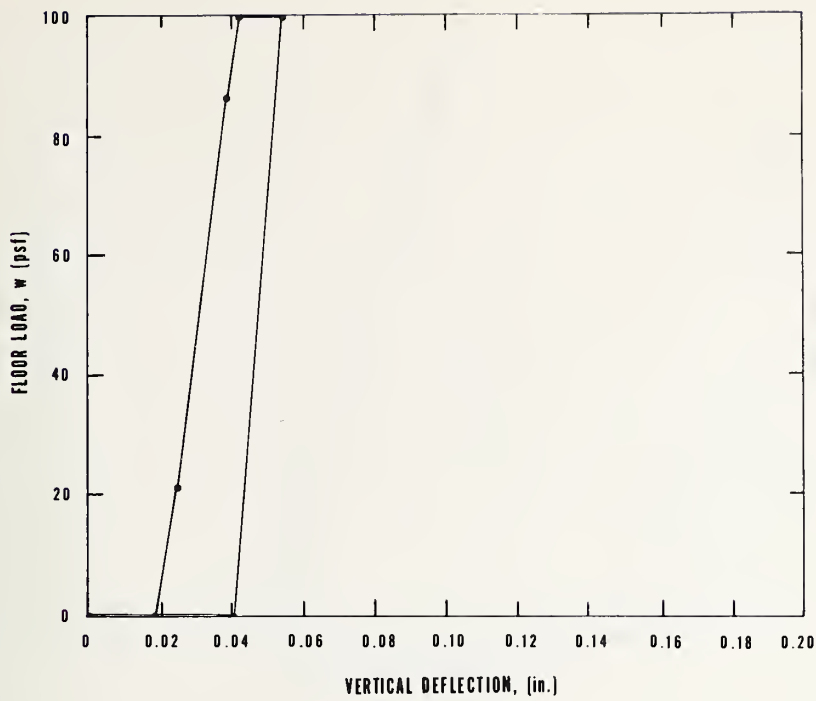FIGURE A. 10. *Test No. 4, floor load versus beam deflection. (Output Channel 72)*
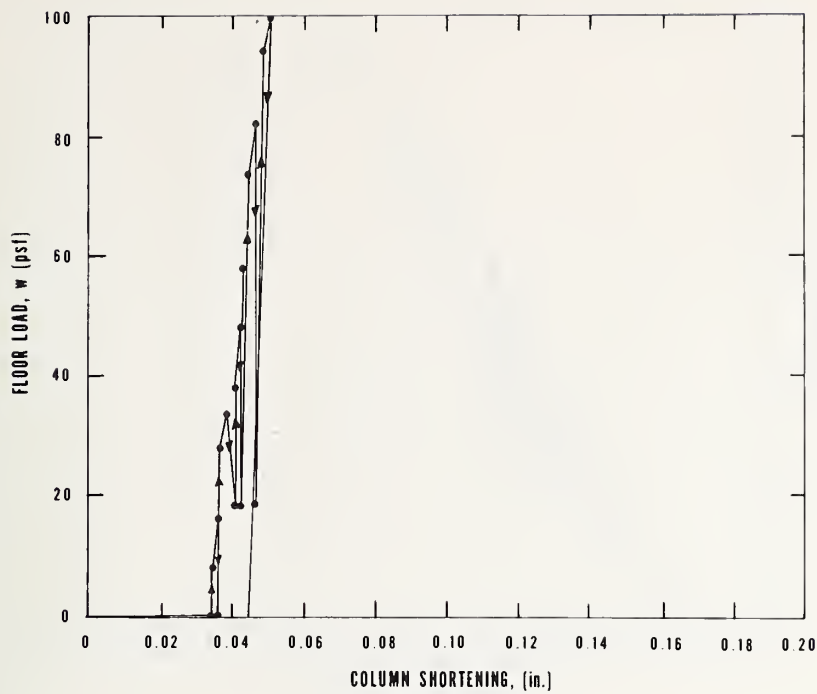
FIGURE A. 11. *Test No. 4, floor load versus column shortening. (Output Channel 73)*
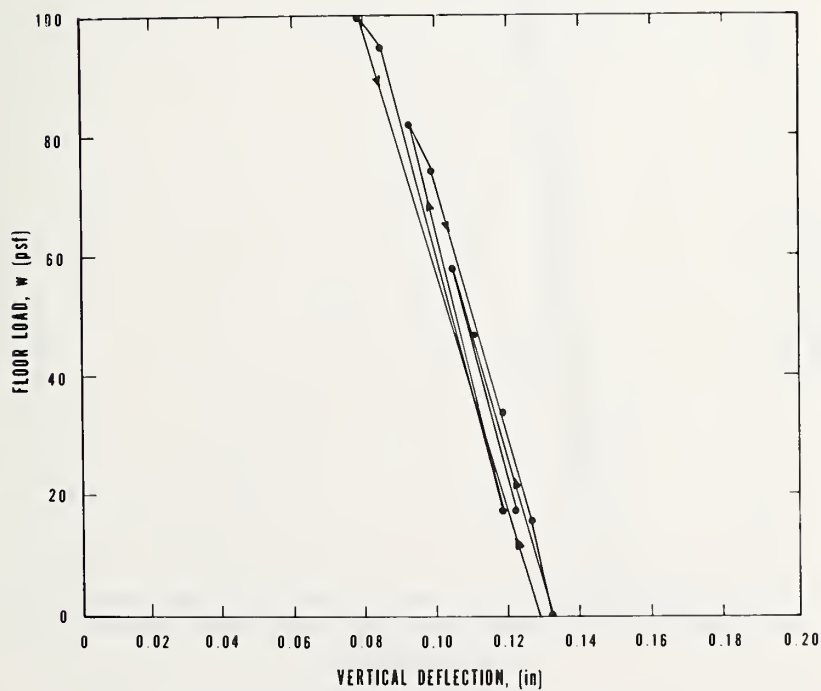
FIGURE A. 12. *Test No. 4, floor load versus beam deflection. (Output Channel 74)*

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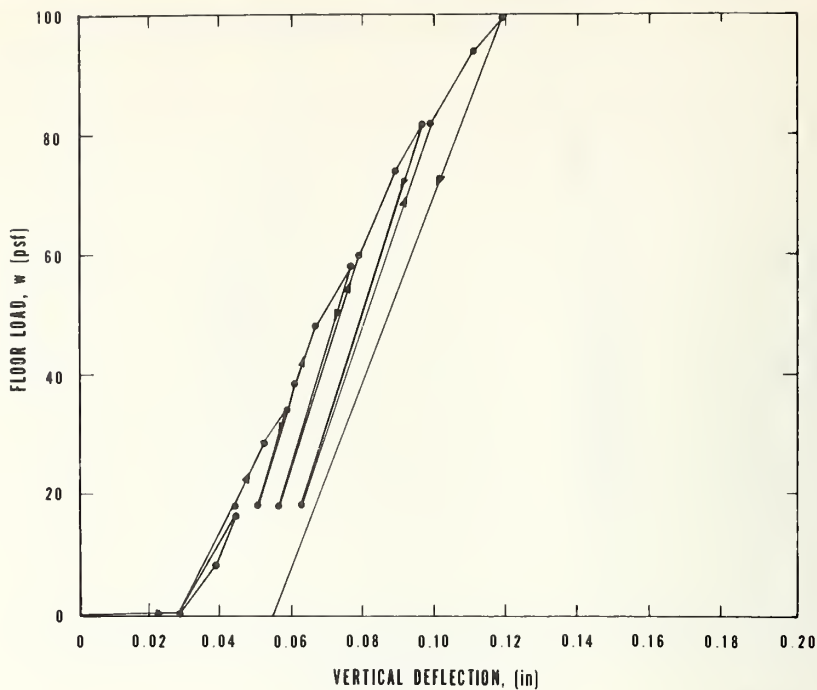
FIGURE A. 13. *Test No. 4, floor load versus slab deflection. (Output Channel 76)*

FIGURE A. 14. *Test No. 4, floor load versus beam deflection. (Output Channel 77)*

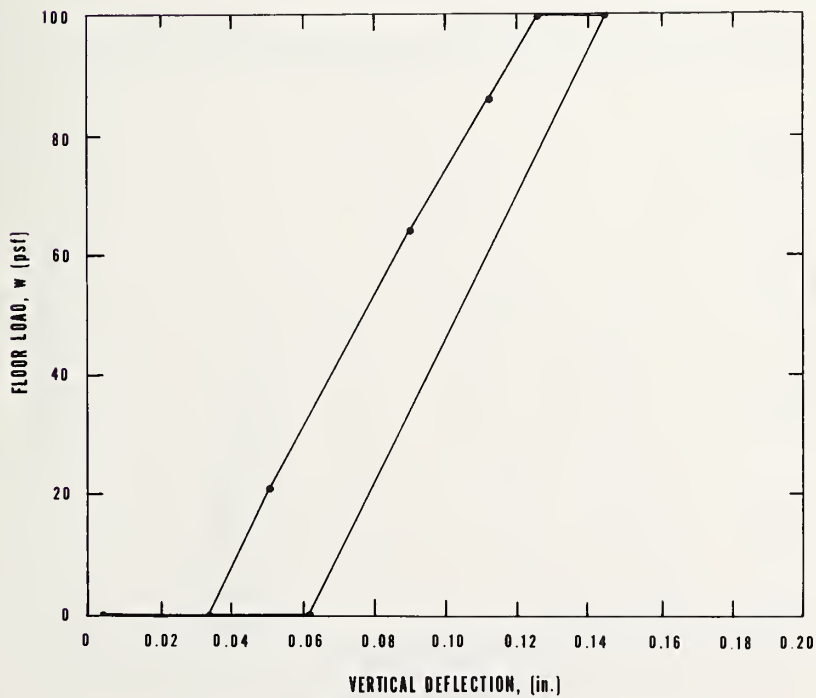FIGURE A. 15. *Test No. 5, sustained floor load versus beam deflection. (Output Channel 40)*

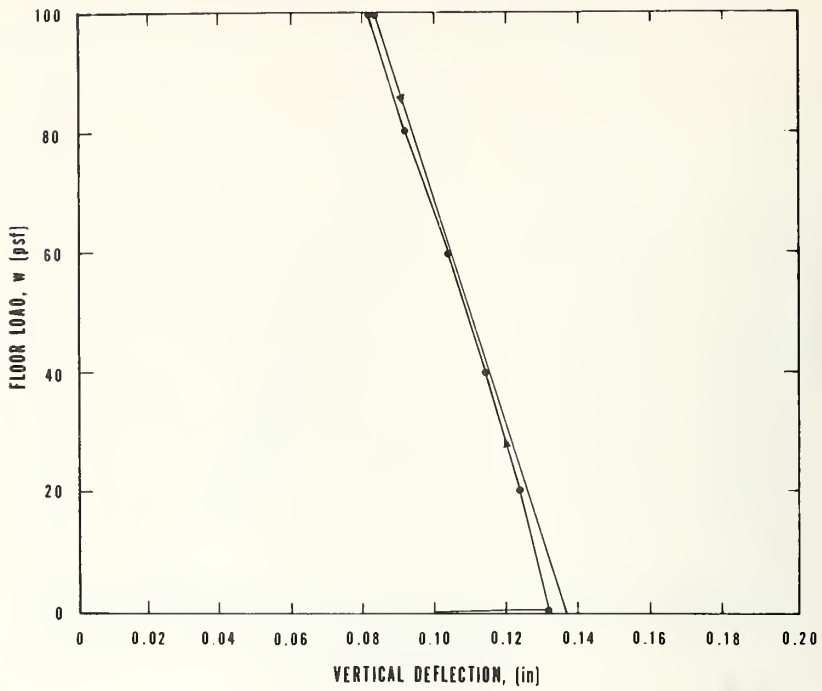FIGURE A. 16. *Test No. 5, sustained floor load versus deflection. (Output Channel 72)*

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FIGURE A. 17. *Test No. 5, sustained floor load versus beam deflection. (Output Channel 74)*
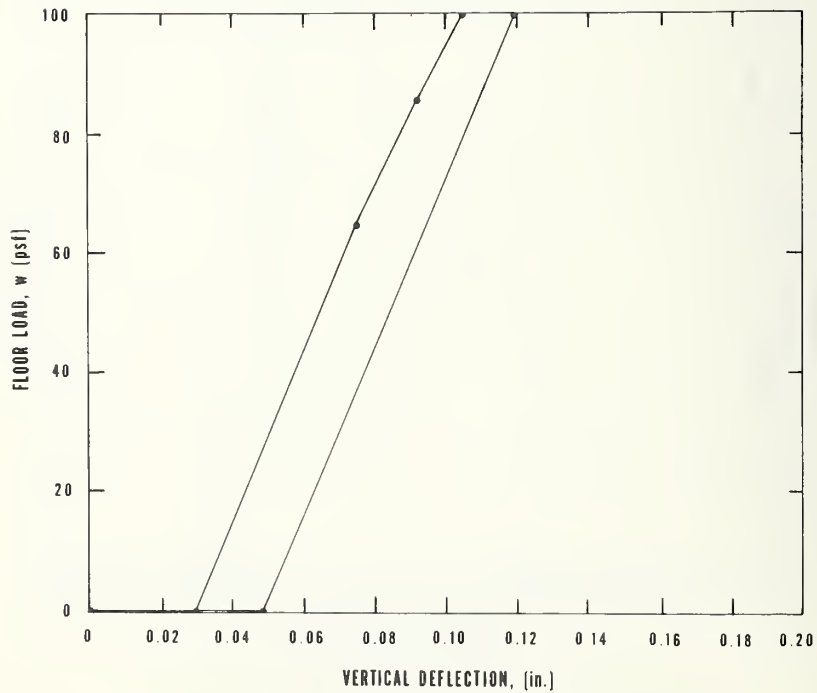

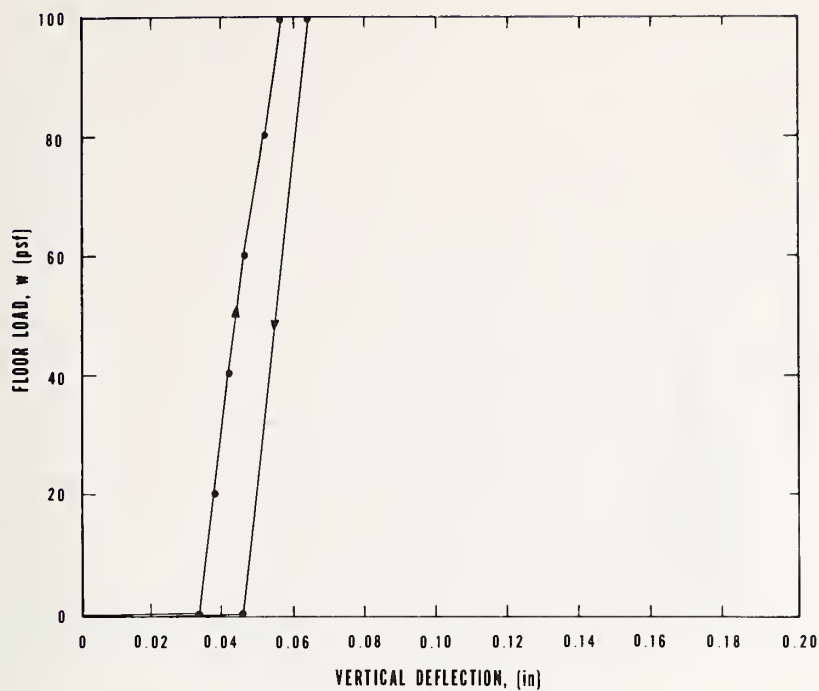FIGURE A. 18. *Test No. 5, sustained floor load versus slab deflection. (Output Channel 76)*

FIGURE A. 19. *Test No. 5, sustained floor load versus beam deflection. (Output Channel 77)*
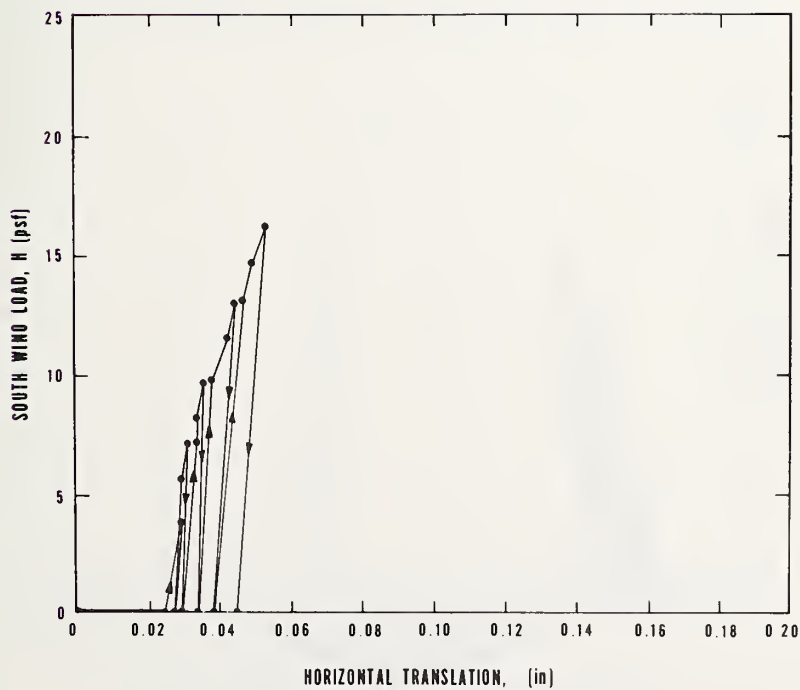

FIGURE A. 20. *Test No. 7, south wind load versus translation. (Output Channel 43)*
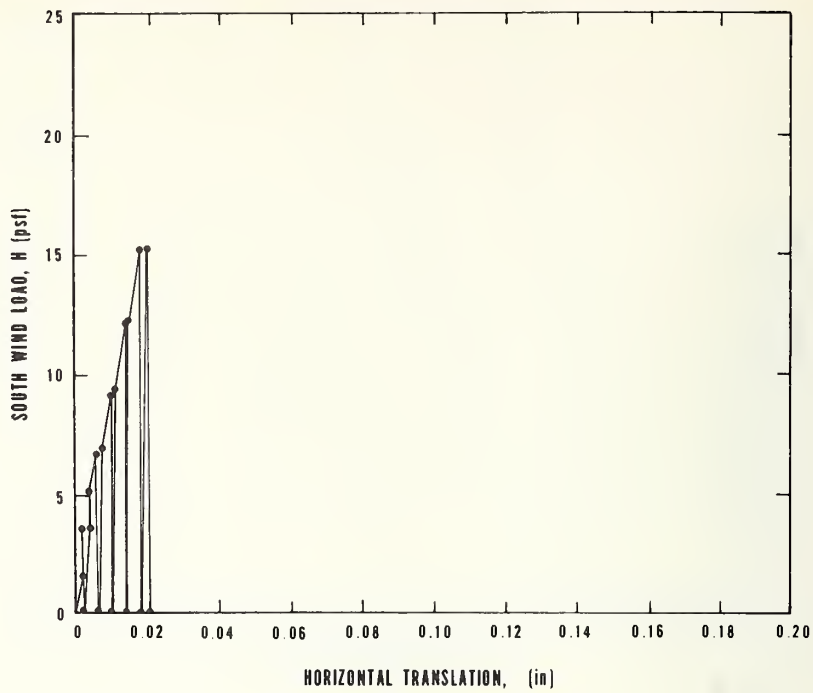
FIGURE A. 21. *Test No. 7, south wind load versus translation. (Output Channel 44)*
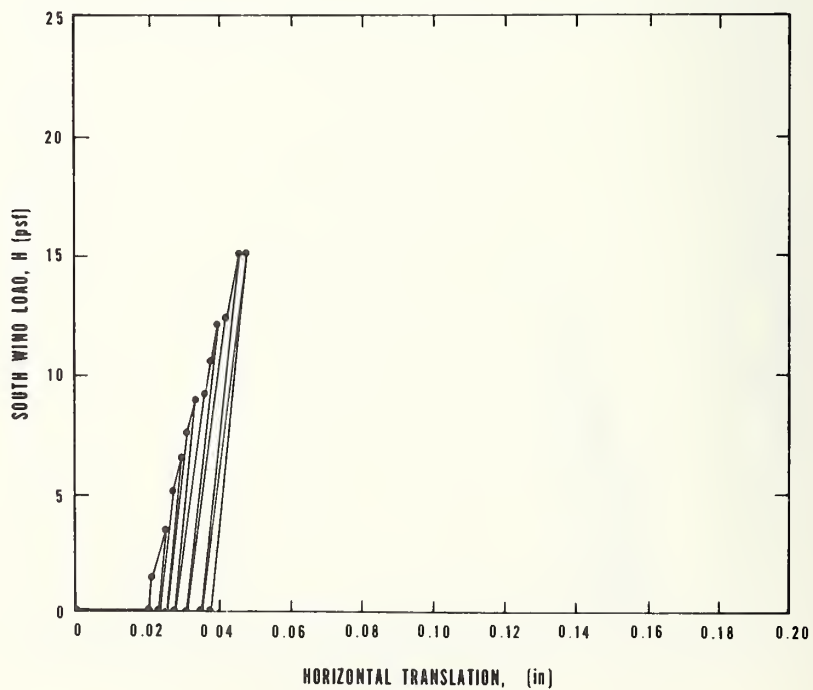

FIGURE A. 22. *Test No. 7, south wind load versus translation. (Output Channel 45)*
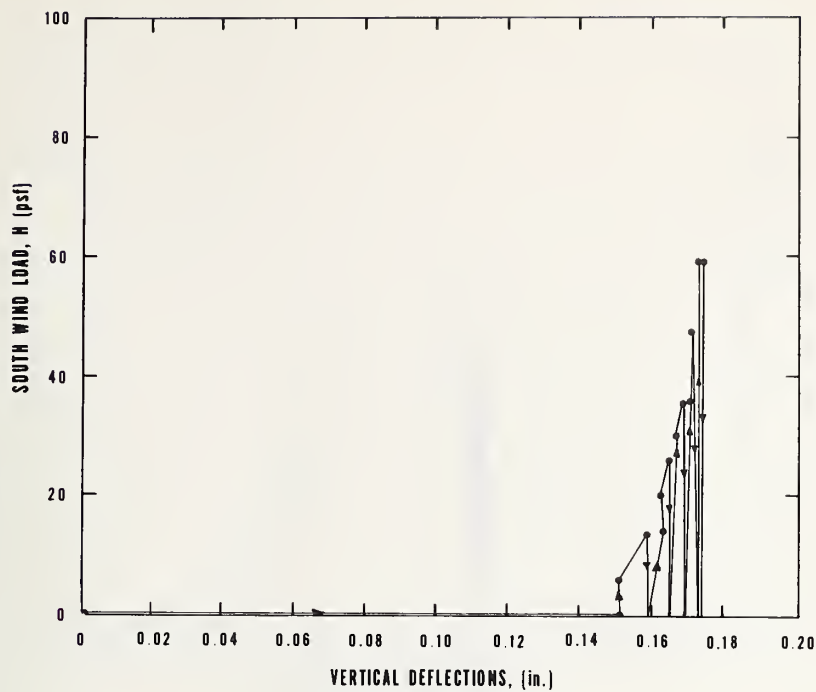
FIGURE A. 23. *Test No. 7, south wind load versus beam
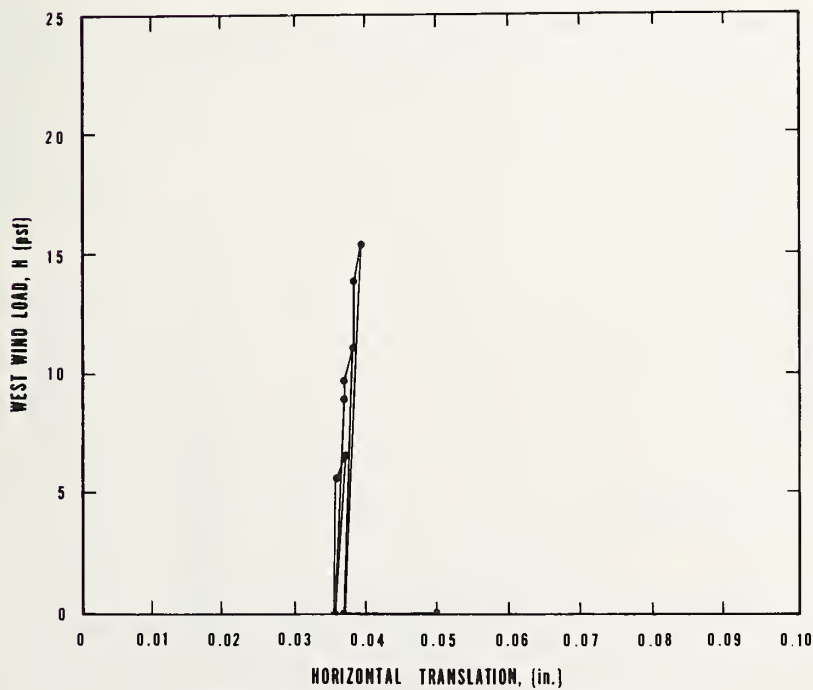deflection. (Output Channel 72)*

FIGURE A. 24. *Test No. 8, west wind load versus
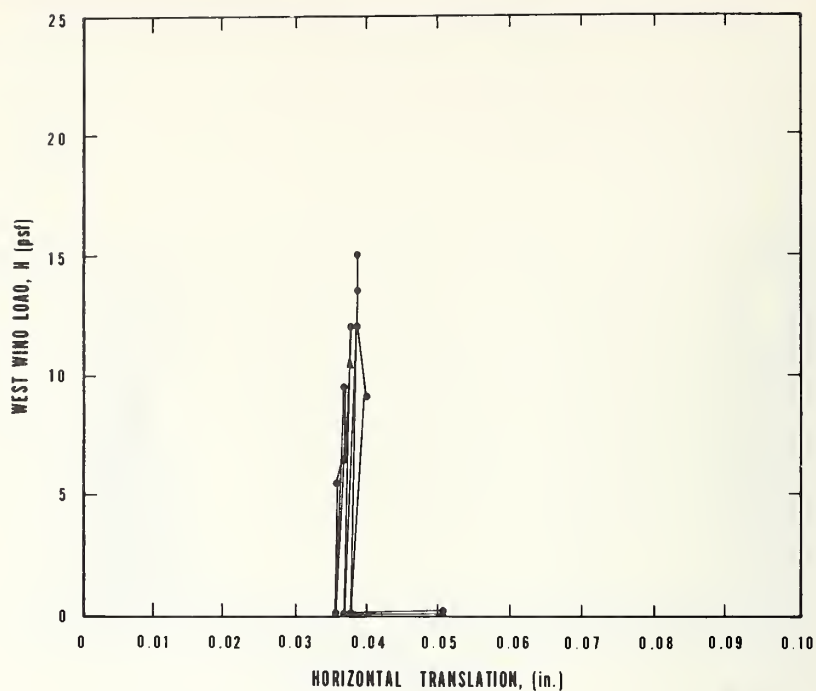translation. (Output Channel 46)*

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FIGURE A. 25. *Test No. 8, west wind load versus translation. (Output Channel 47)*

FIGURE A. 26. *Test No. 8, west wind load versus beam deflection. (Output Channel 72)*

FIGURE A. 27. *Test No. 9, major floor load versus beam deflection. (Output Channel 40)*

FIGURE A. 28. *Test No. 9, major floor load versus beam deflection. (Output Channel 72)*

FIGURE A. 29. *Test No. 9, major floor load versus columns shortening. (Output Channel 73)*
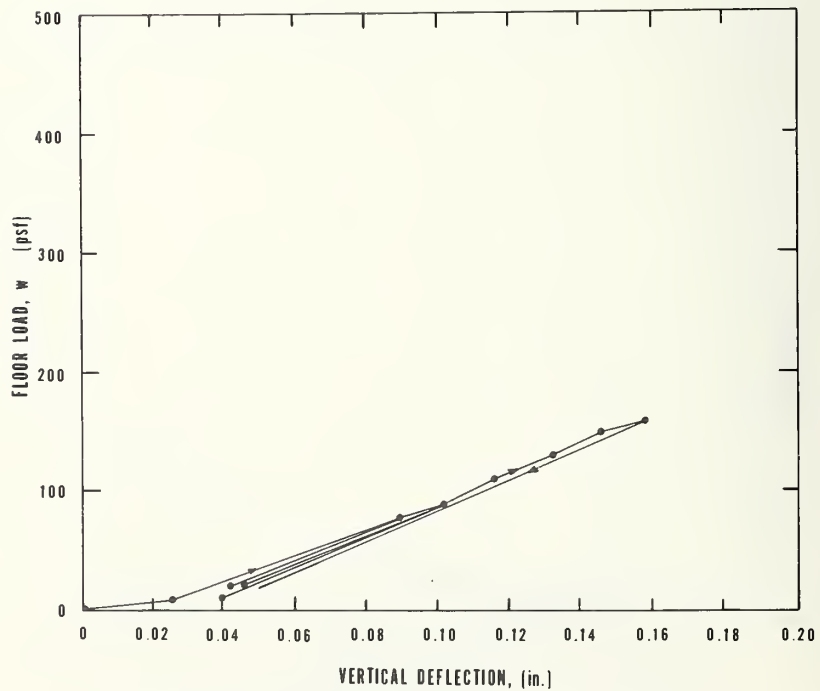
FIGURE A. 30. *Test No. 9, major floor load versus slab deflection. (Output Channel 76)*
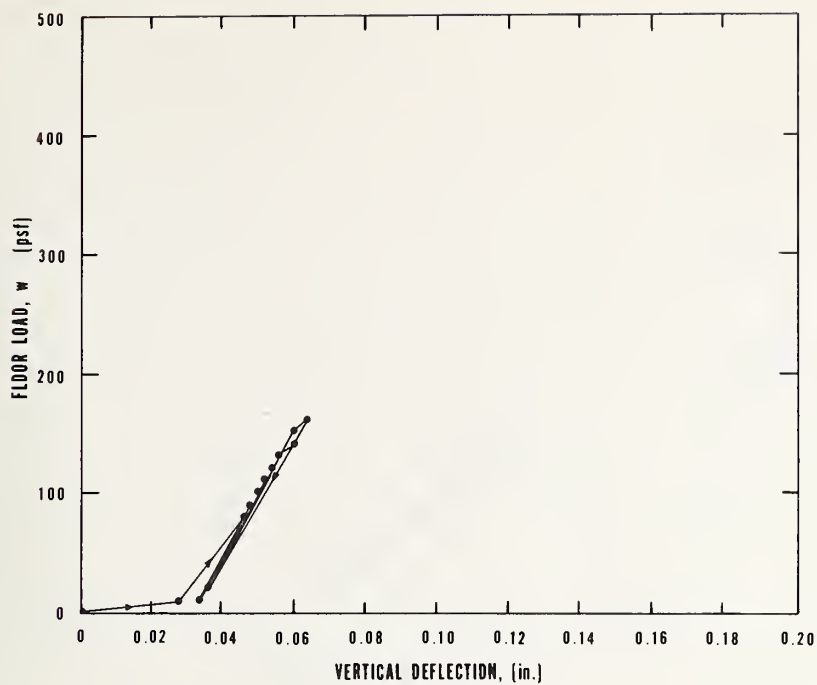
FIGURE A. 31. *Test No. 9, major floor load versus beam
deflection. (Output Channel 77)*
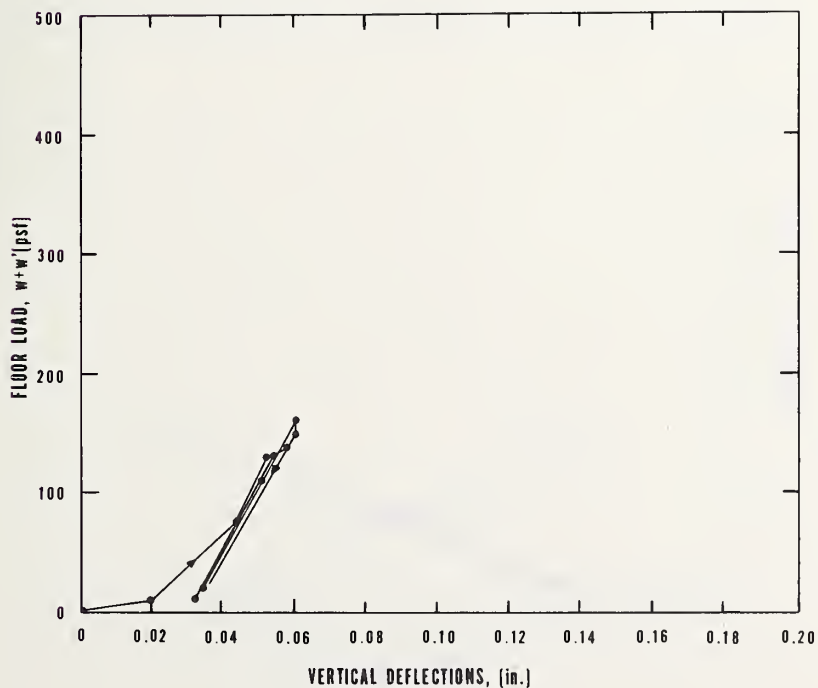
FIGURE A. 32. *Test No. 9A, floor loads versus beam
deflection. (Output Channel 40)*
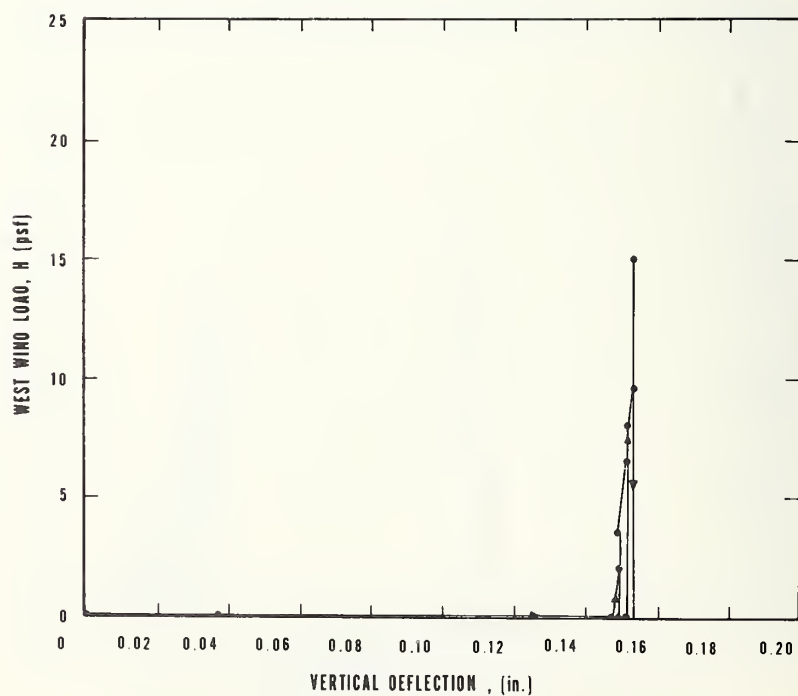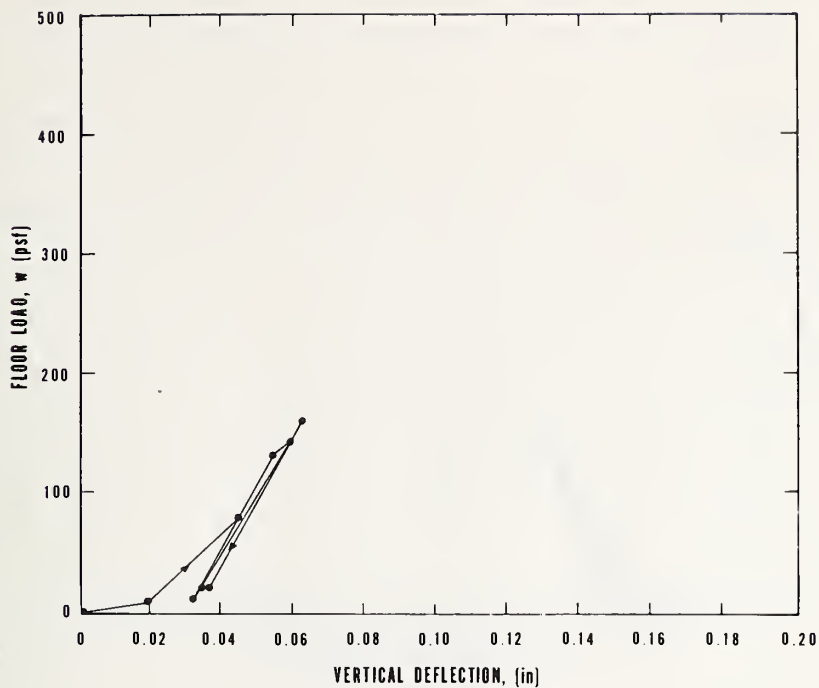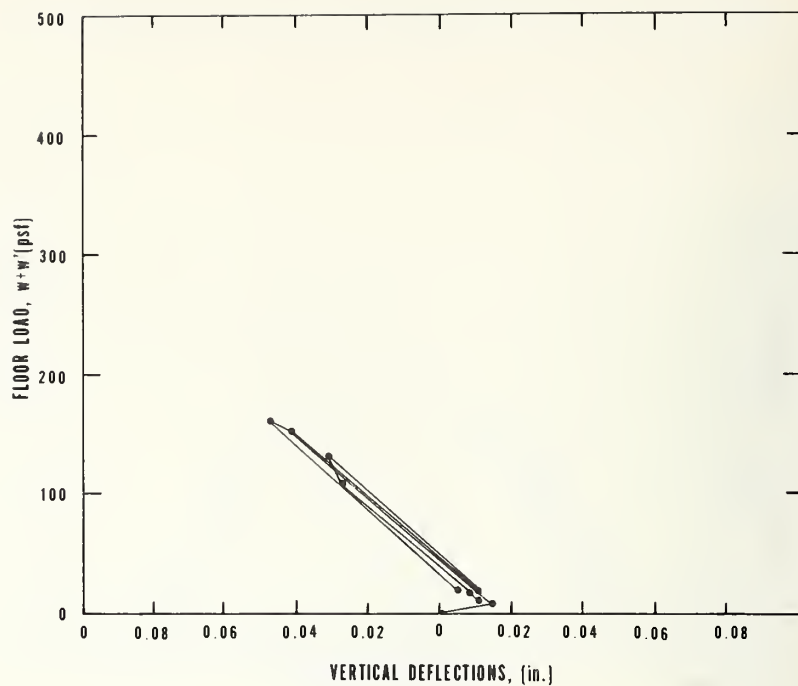
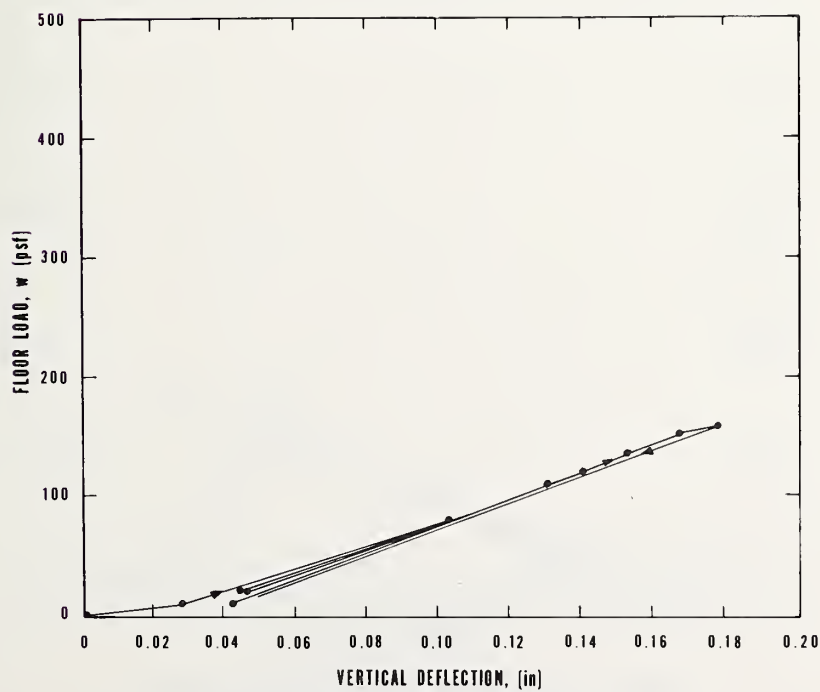FIGURE A. 33. *Test No. 9A, floor loads versus beam deflection. (Output Channel 70)*

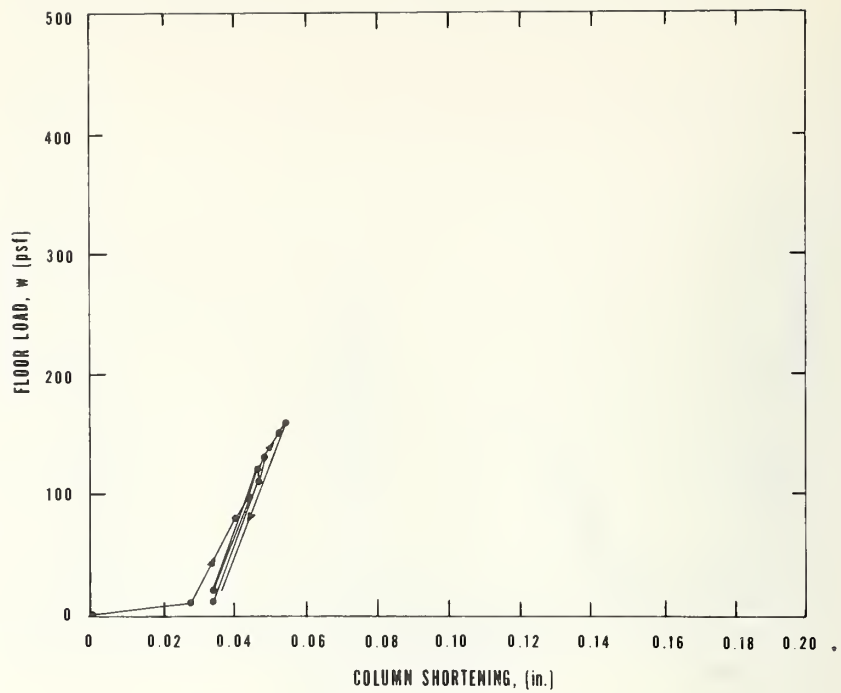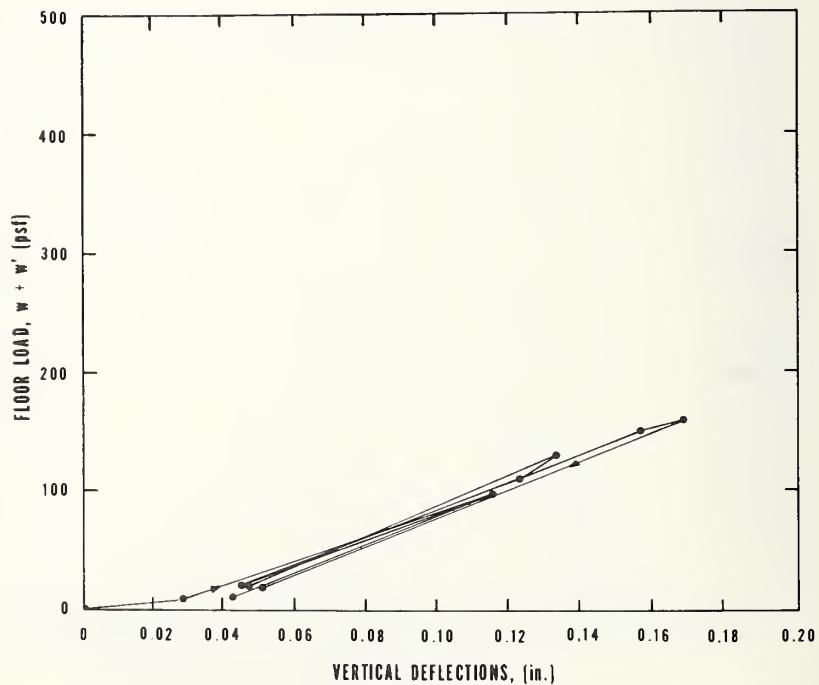FIGURE A. 34. *Test No. 9A, floor loads versus beam deflection. (Output Channel 72)*
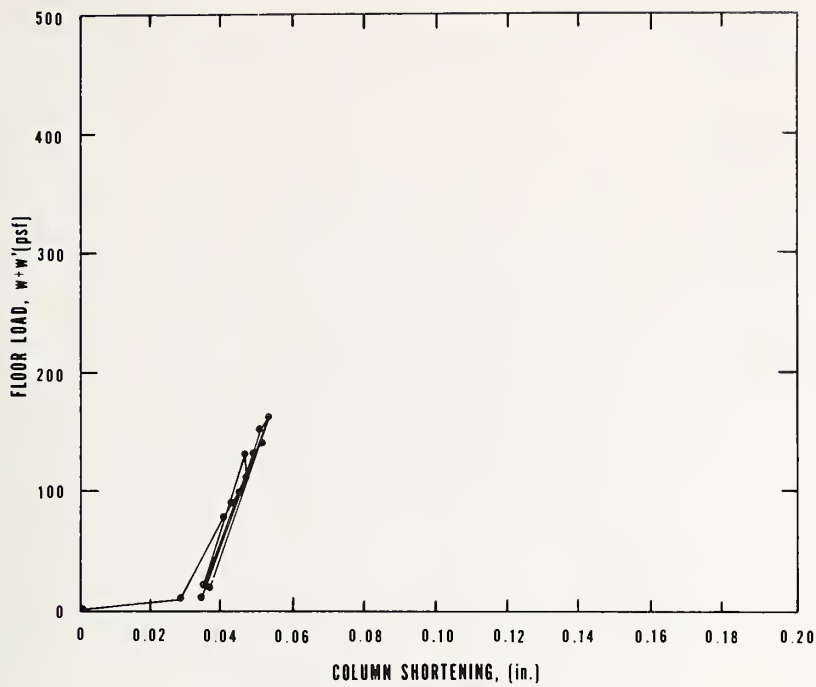
FIGURE A. 35. *Test No. 9A, floor loads versus column shortening. (Output Channel 73)*
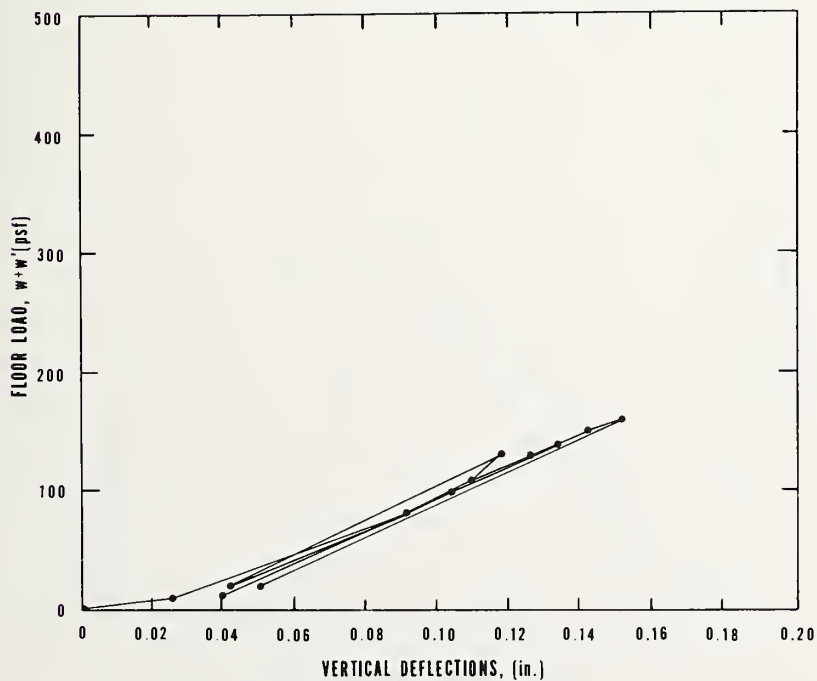
FIGURE A. 36. *Test No. 9A, floor load versus slab deflection. (Output Channel 76)*
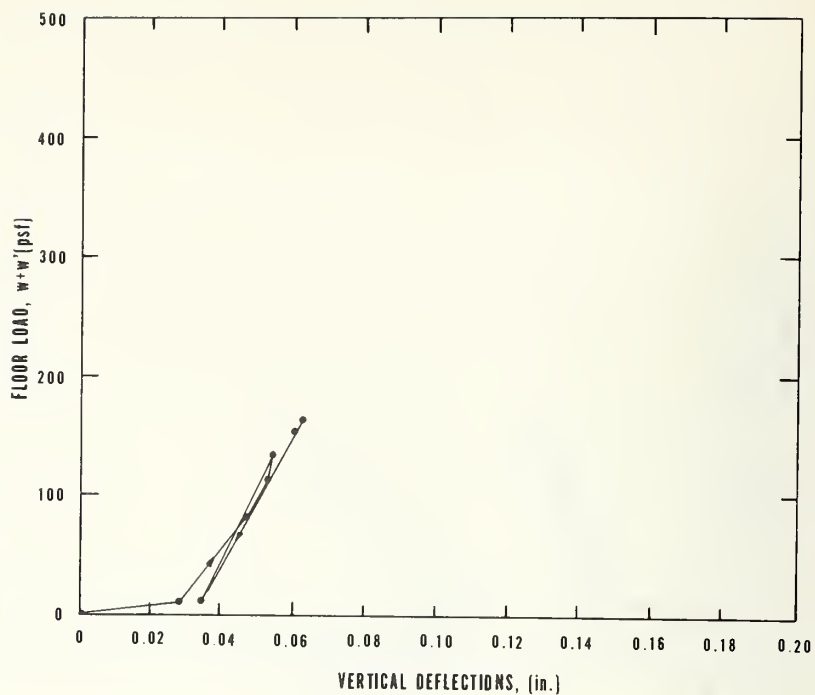
FIGURE A. 37. *Test No. 9A, floor load versus beam deflection. (Output Channel 77)*

FIGURE A. 38. *Test No. 10, south wind load versus translation. (Output Channel 43)*

FIGURE A. 39. *Test No. 10, south wind load versus translation. (Output Channel 44)*

FIGURE A. 40. *Test No. 10, south wind load versus wall compression. (Output Channel 56)*

FIGURE A. 41. *Test No. 10, south wind load versus wall compression. (Output Channel 57)*
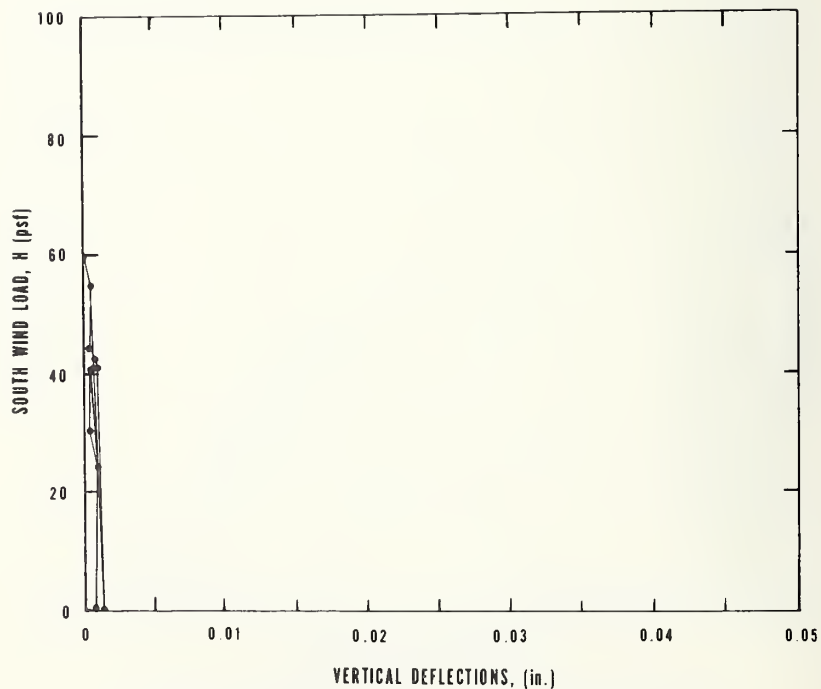
FIGURE A. 42. *Test No. 10, south wind load versus beam deflection. (Output Channel 72)*
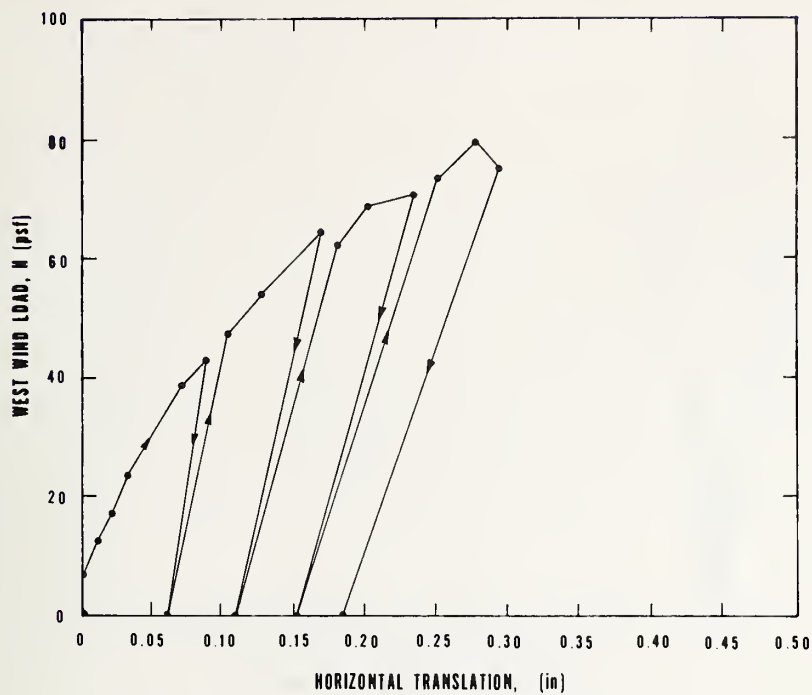
FIGURE A. 43. *Test No. 11, west wind load versus translation.* (*Output Channel 46*)

FIGURE A. 44. *Test No. 11, west wind load versus translation.* (*Output Channel 47*)
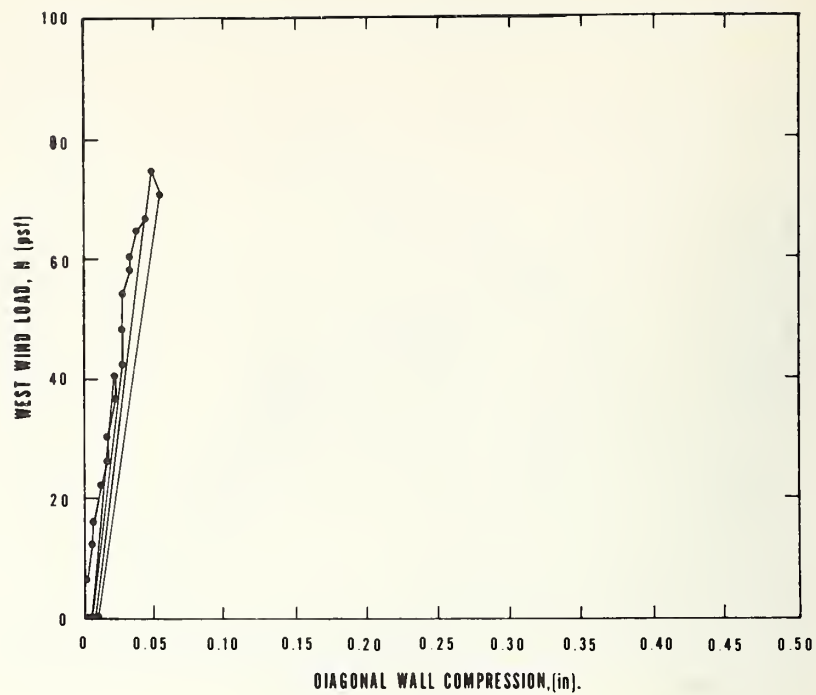
FIGURE A. 45. *Test No. 11, west wind load versus wall compression. (Output Channel 52)*
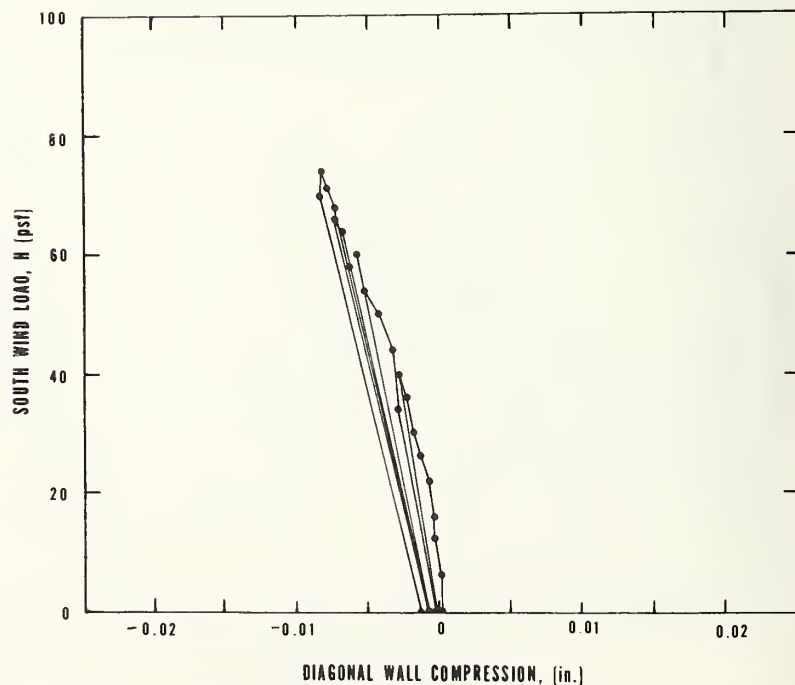
FIGURE A. 46. *Test No. 11, west wind load versus wall compression. (Output Channel 53)*
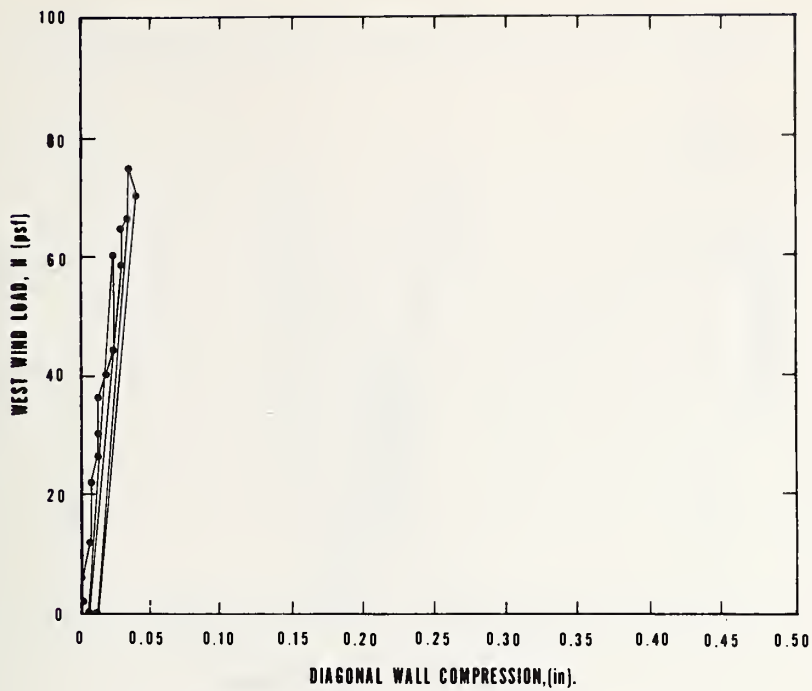
FIGURE A. 47. *Test No. 11, west wind load versus wall compression. (Output Channel 54)*
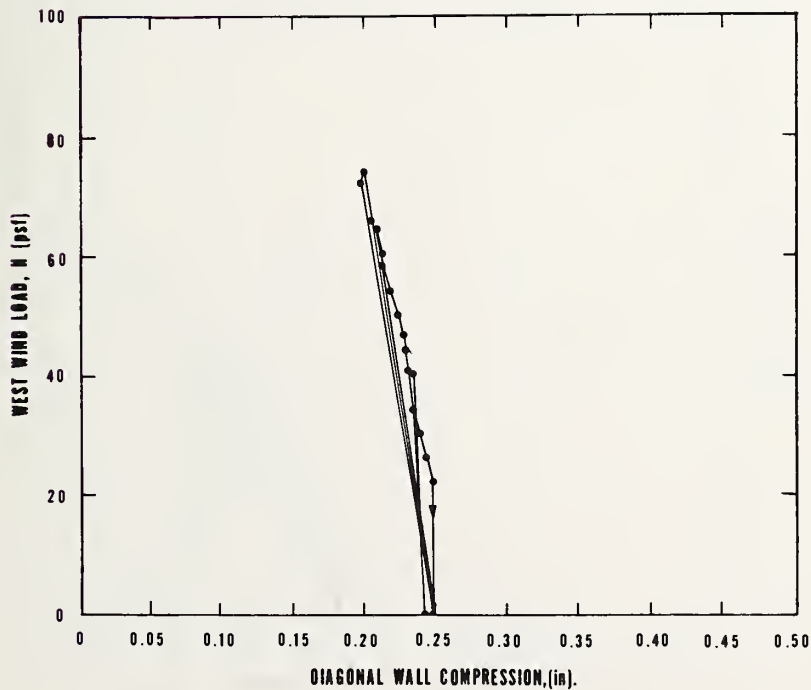
FIGURE A. 48. *Test No. 11, west wind load versus wall compression. (Output Channel 55)*
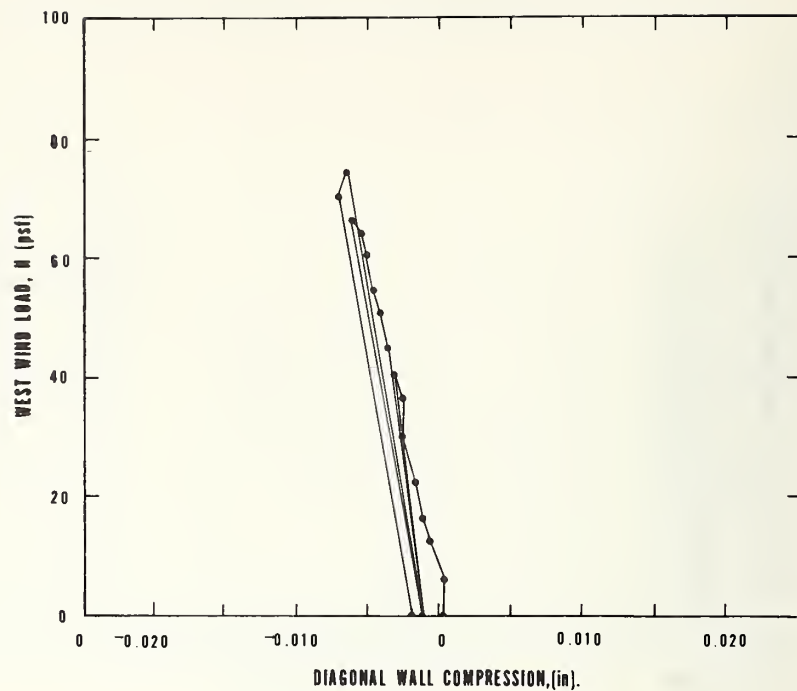
FIGURE A. 49. *Test No. 11, west wind load versus wall compression. (Output Channel 58)*
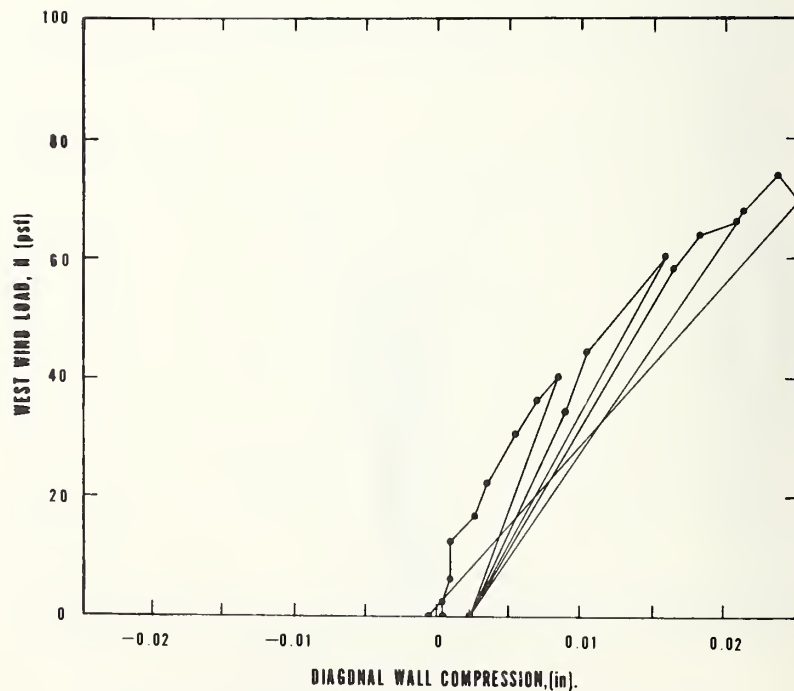
FIGURE A. 50. *Test No. 11, west wind load versus wall compression. (Output Channel 59)*
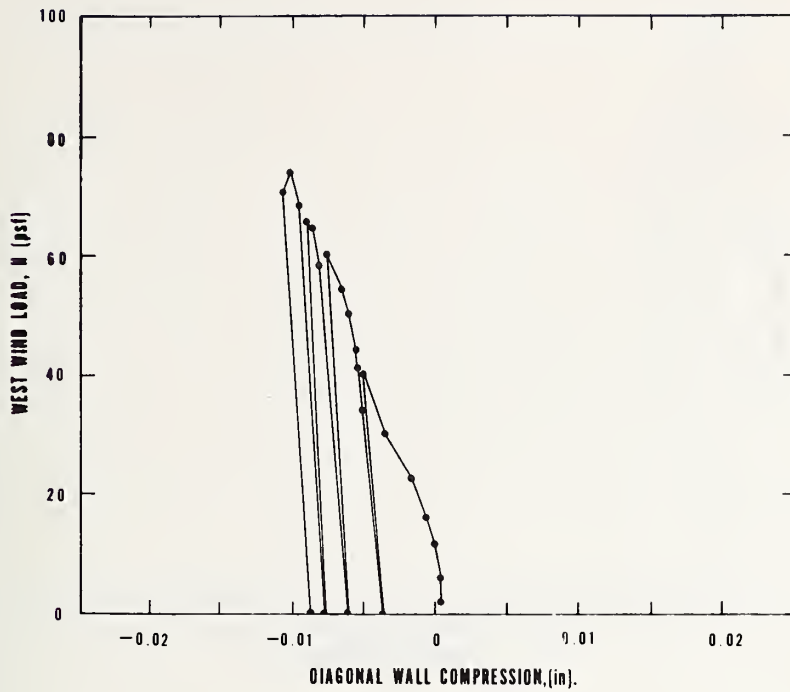
FIGURE A. 51. *Test No. 11, west wind load versus wall compression. (Output Channel 60)*
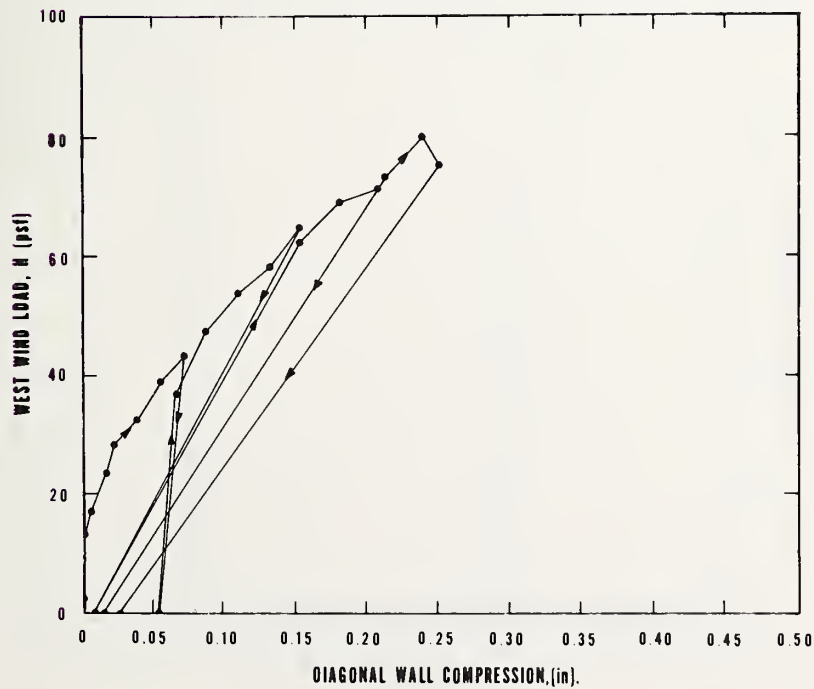
FIGURE A. 52. *Test No. 11, west wind load versus wall compression. (Output Channel 61)*

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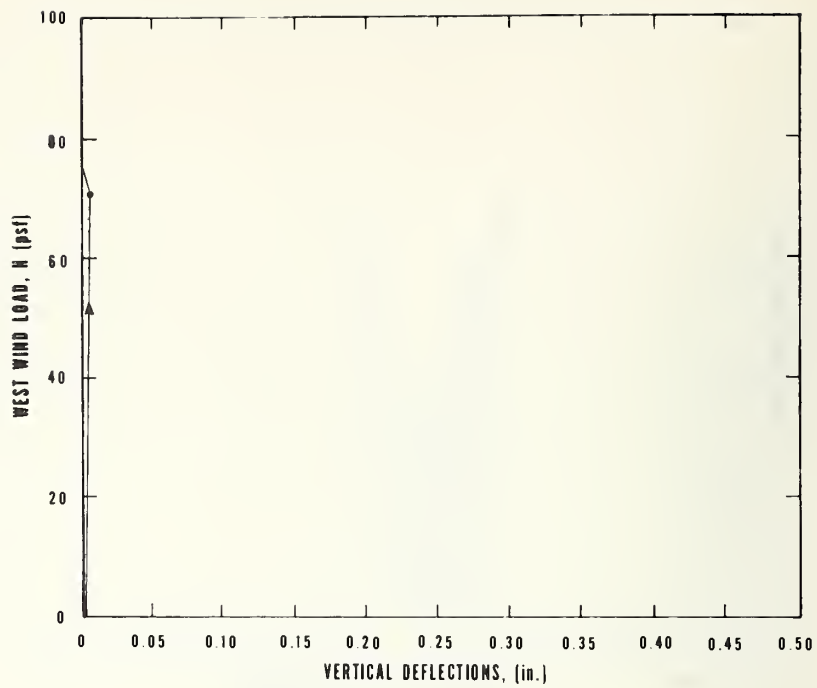
FIGURE A. 53. *Test No. 11, west wind load versus beam deflection. (Output Channel 72)*
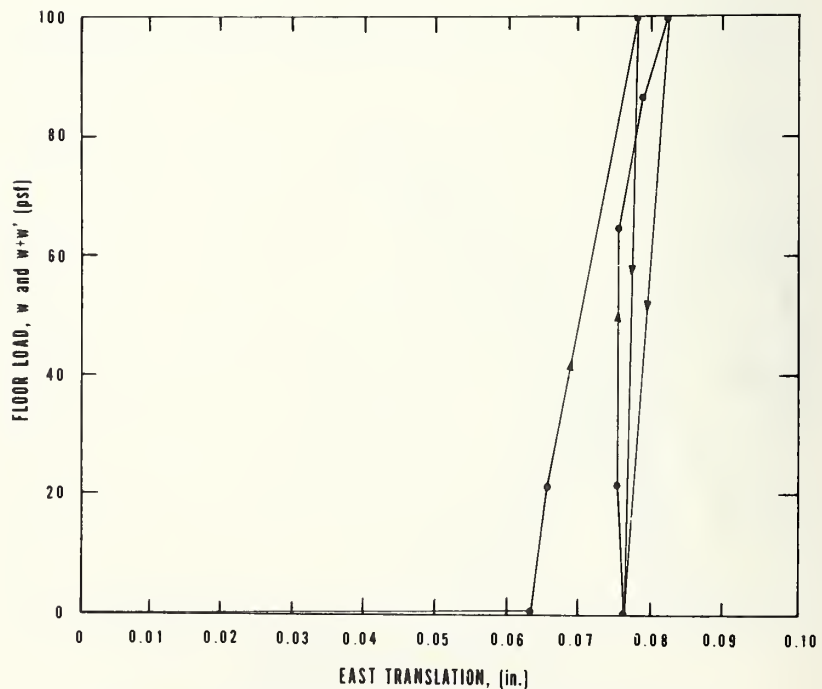
FIGURE A. 54. *Tests No. 12 and 13, vertical load versus east translation. (Output Channel 46)*
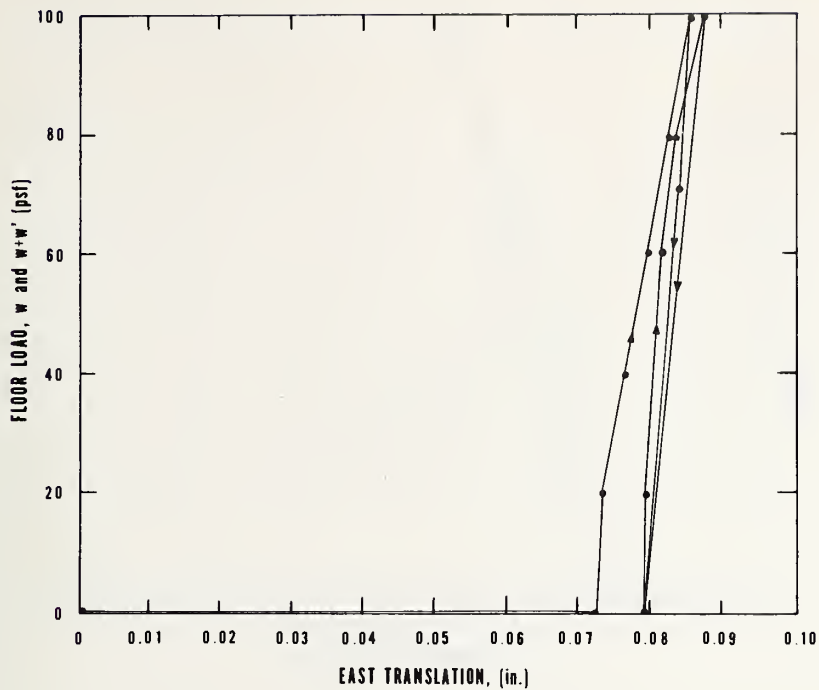
FIGURE A. 55. *Tests No. 12 and 13, vertical load versus east translation. (Output Channel 47)*
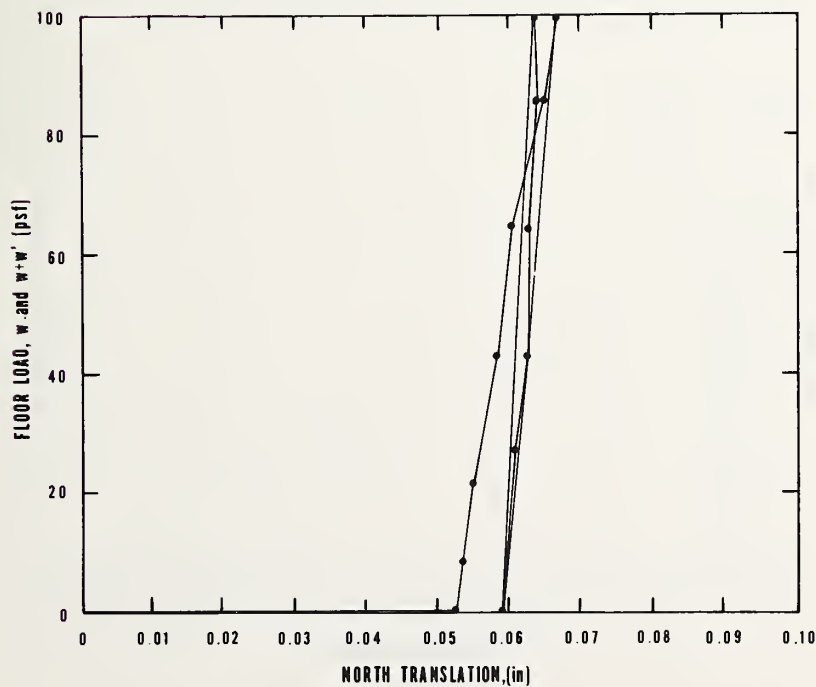
FIGURE A. 56. *Tests No. 12A and 13A, vertical load versus north translation. (Output Channel 43)*

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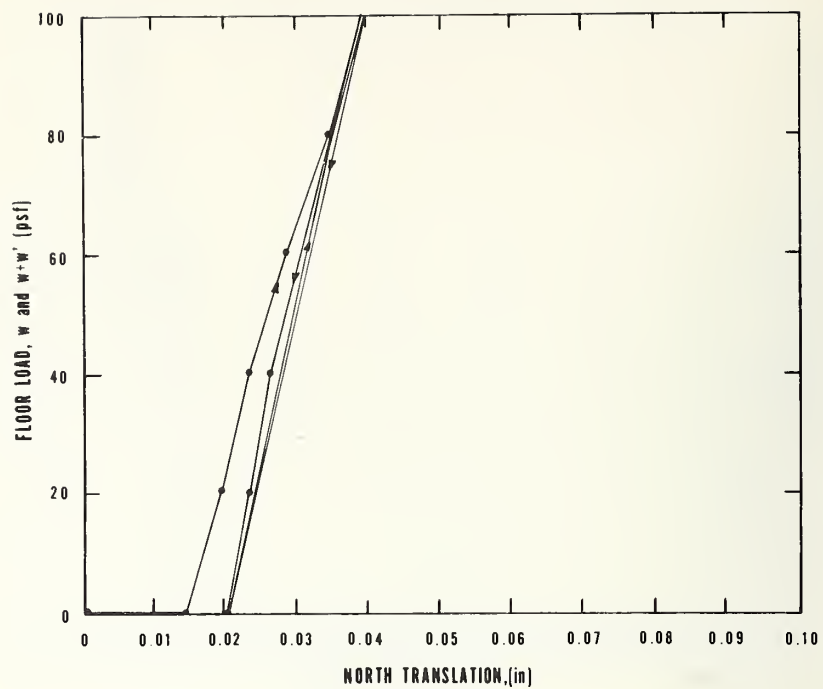
FIGURE A. 57. *Tests No. 12A and 13A, vertical load versus north translation. (Output Channel 44)*
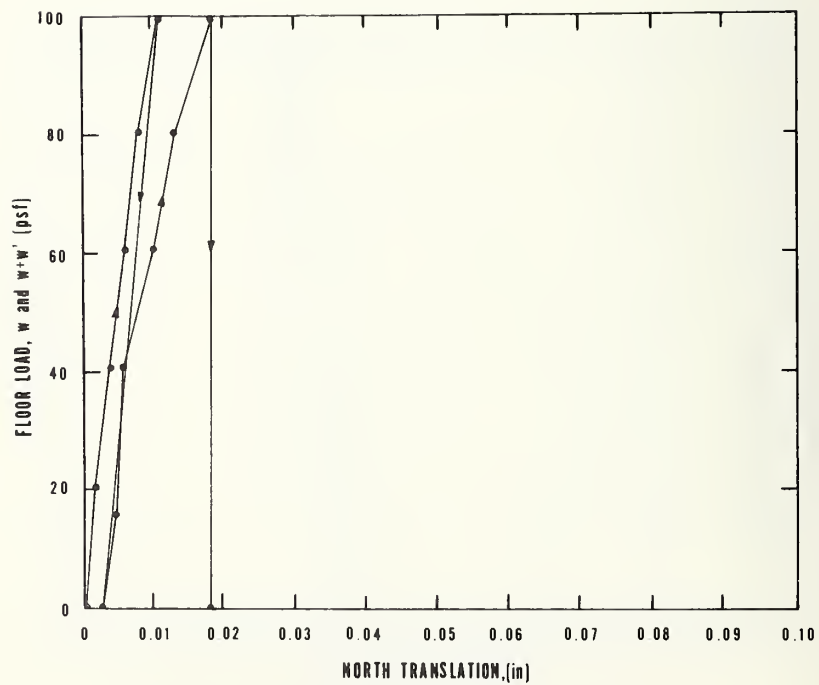
FIGURE A. 58. *Tests No. 12A and 13A, vertical load versus north translation. (Output Channel 45)*
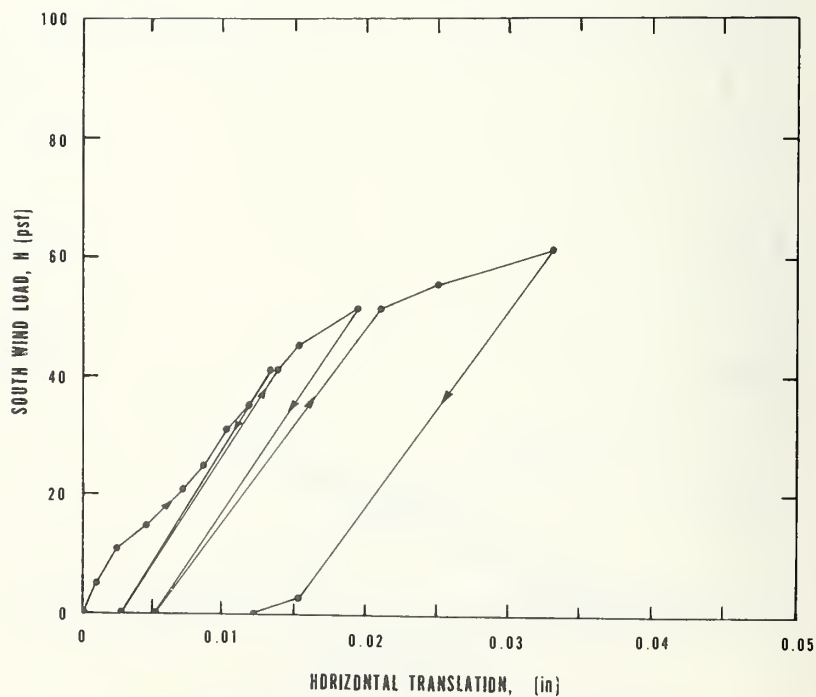
FIGURE A. 59. *Test No. 14, south wind load versus translation. (Output Channel 43)*
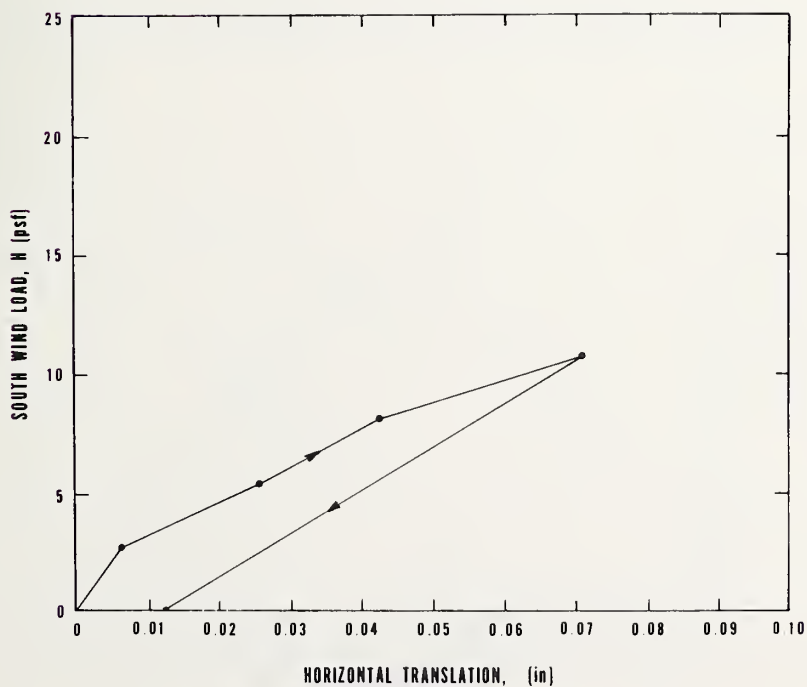
FIGURE A. 60. *Test No. 14, south wind load versus translation. (Output Channel 44)*
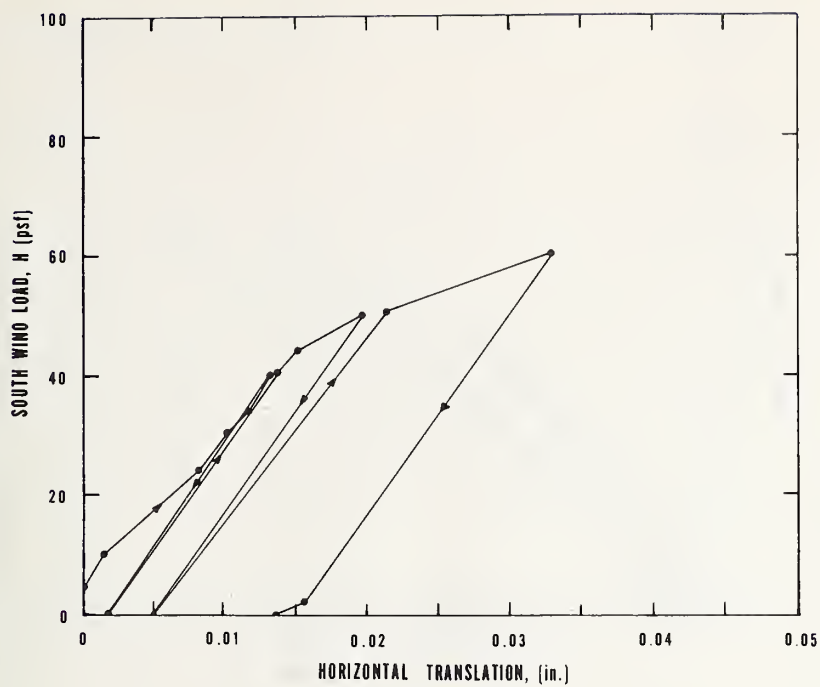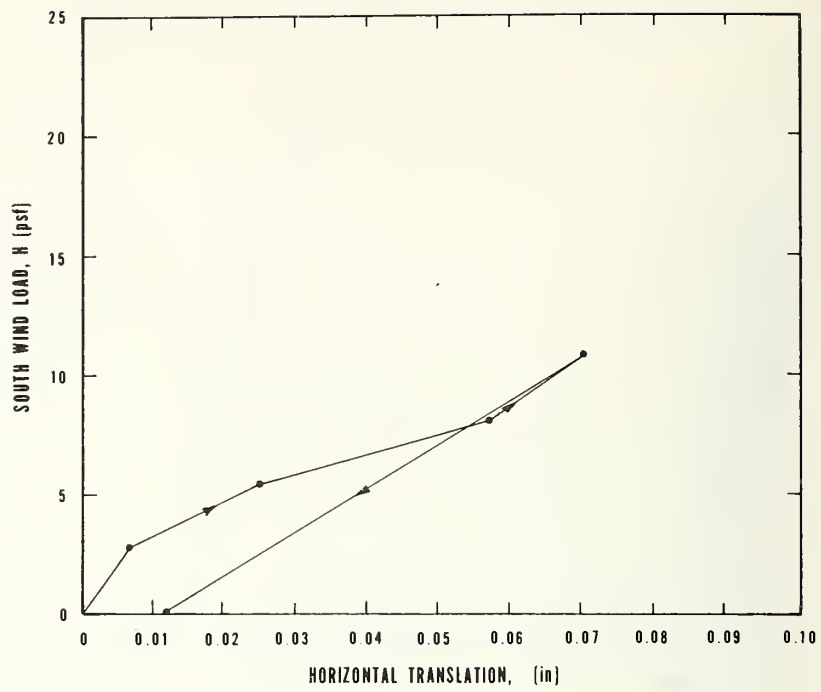
FIGURE A. 61. *Test No. 14, south wind load versus translation. (Output Channel 45)*
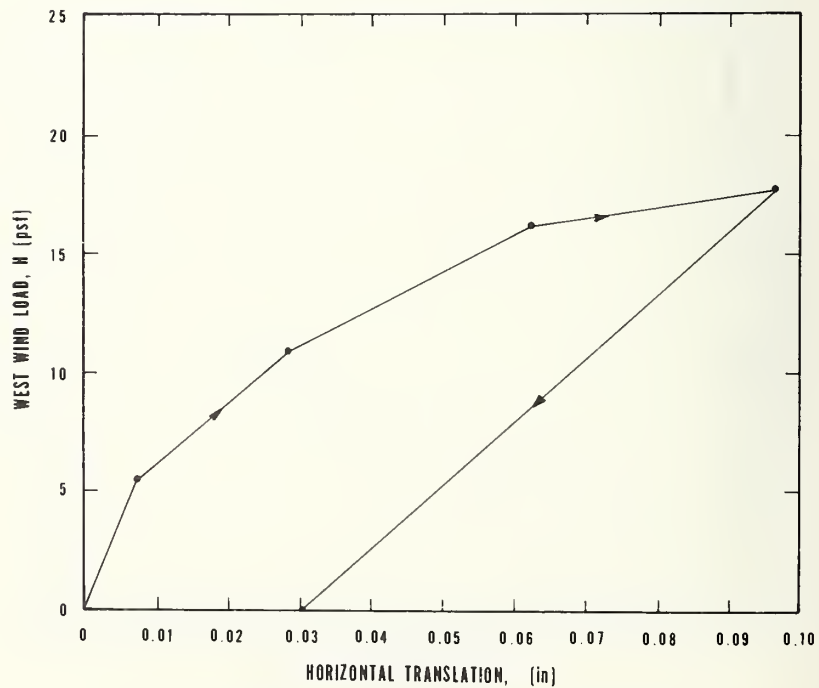
FIGURE A. 62. *Test No. 15, west wind load versus translation. (Output Channel 46)*
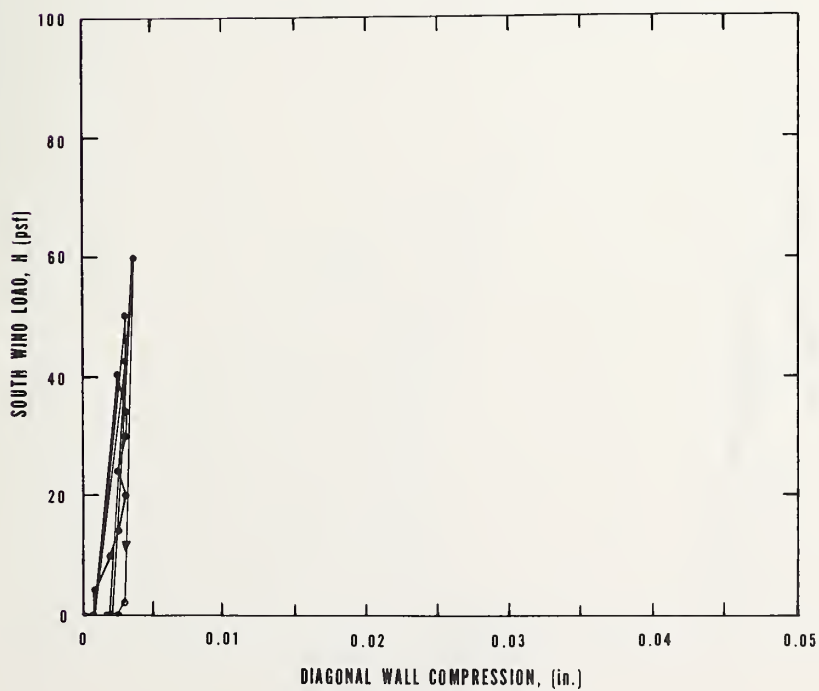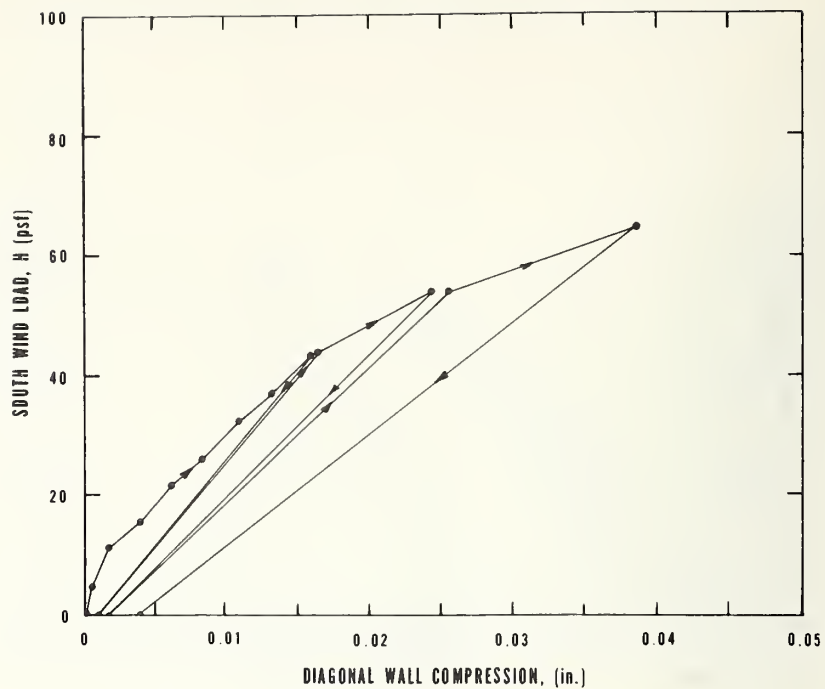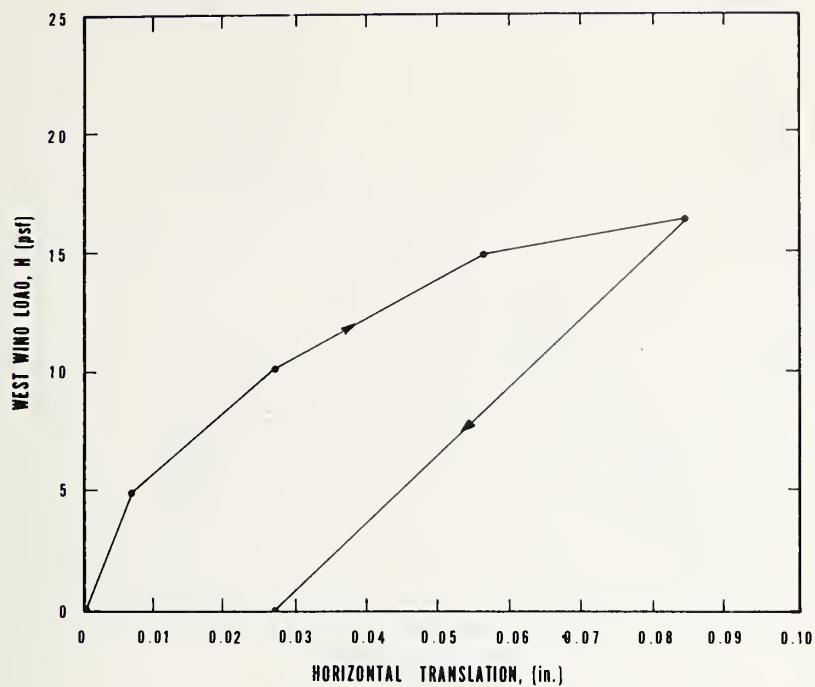
FIGURE A. 63. *Test No. 15, west wind load versus translation. (Output Channel 47)*
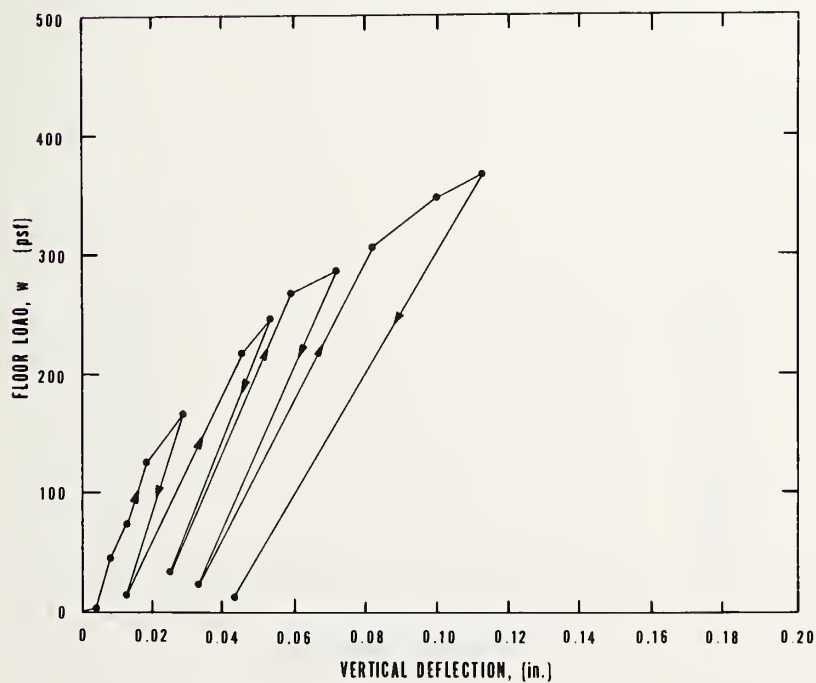
FIGURE A. 64. *Test No. 16, major floor load versus beam deflection. (Output Channel 40)*
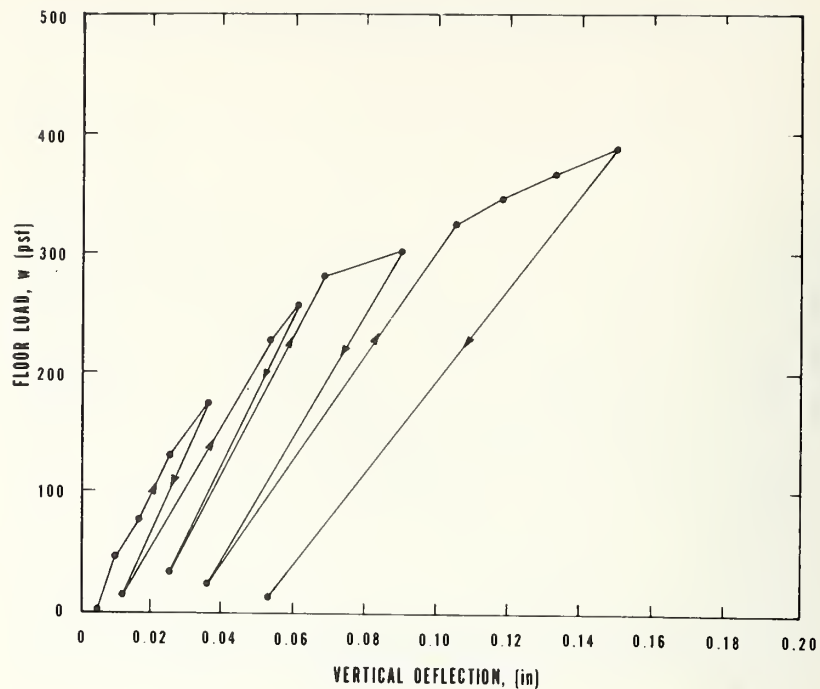
FIGURE A. 65. *Test No. 16, major floor load versus beam deflection. (Output Channel 72)*
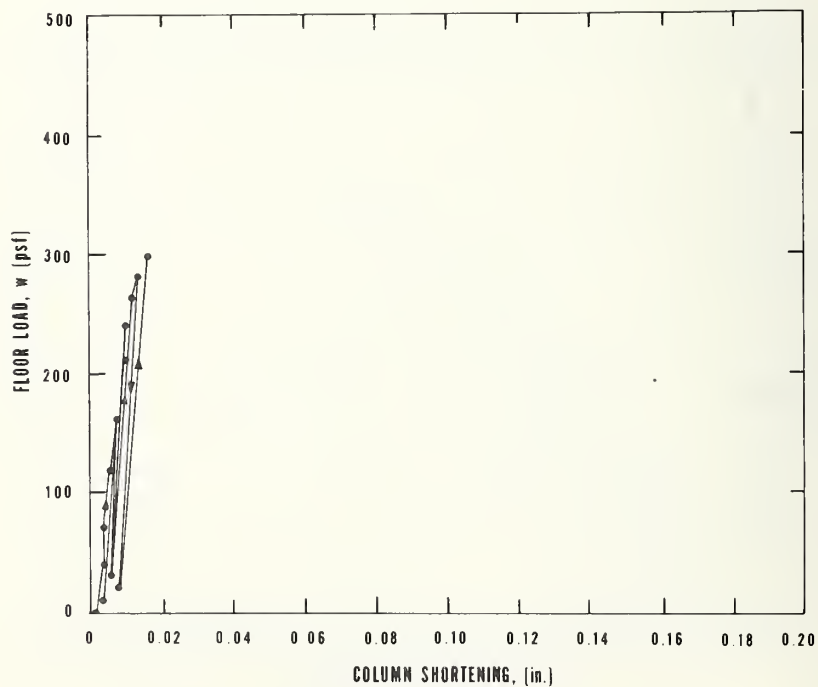
FIGURE A. 66. *Test No. 16, major floor load versus column shortening. (Output Channel 73)*
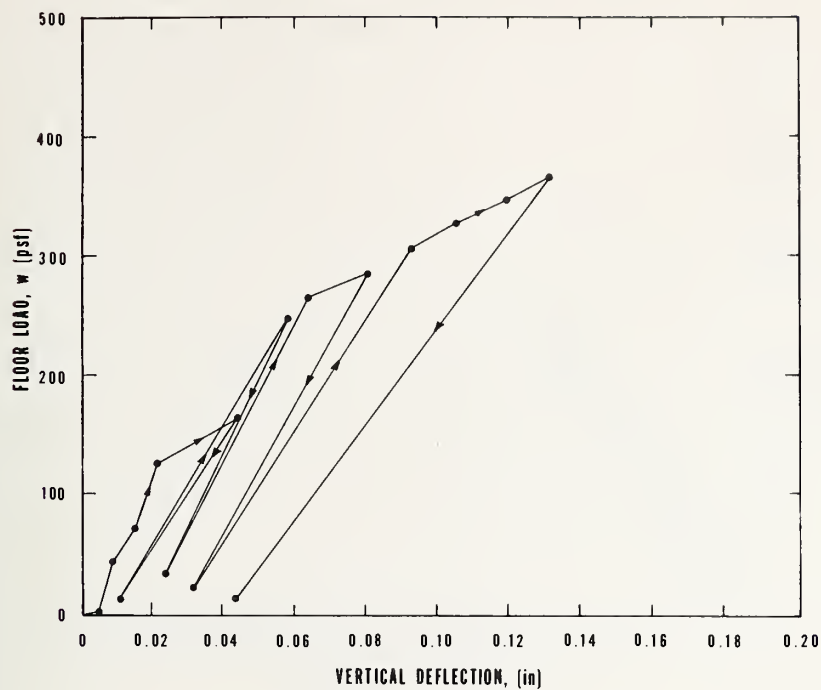
FIGURE A. 67. *Test No. 16, major flood load versus slab deflection. (Output Channel 76)*
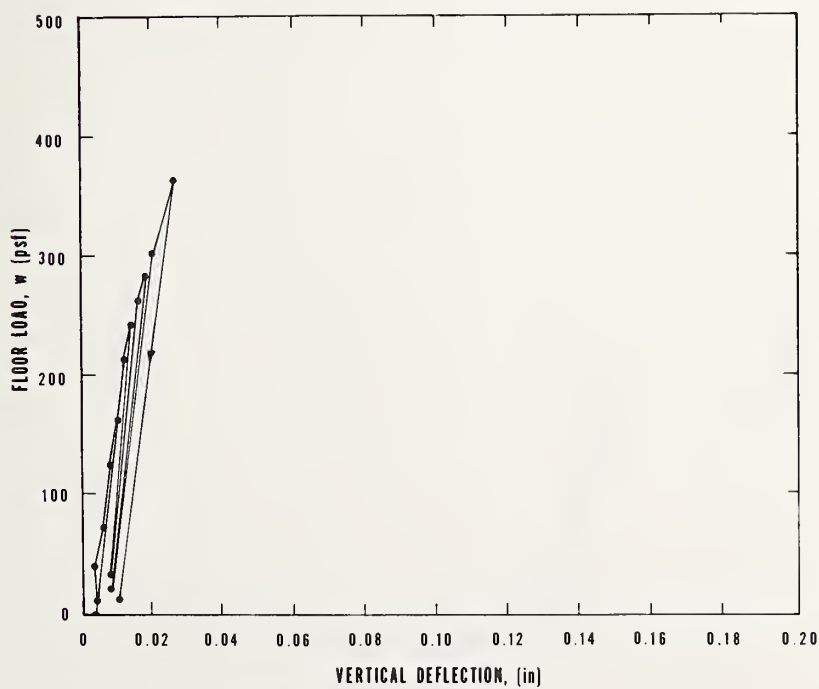
FIGURE A. 68. *Test No. 16, major floor load versus slab deflection. (Output Channel 77)*

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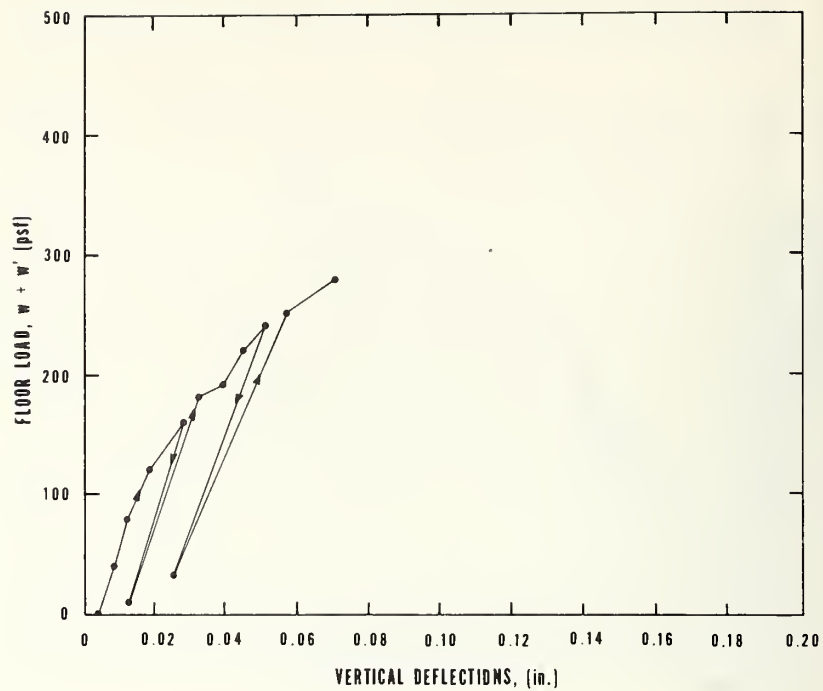
FIGURE A. 69. *Test No. 16A, floor loads versus beam deflection. (Output Channel 40)*
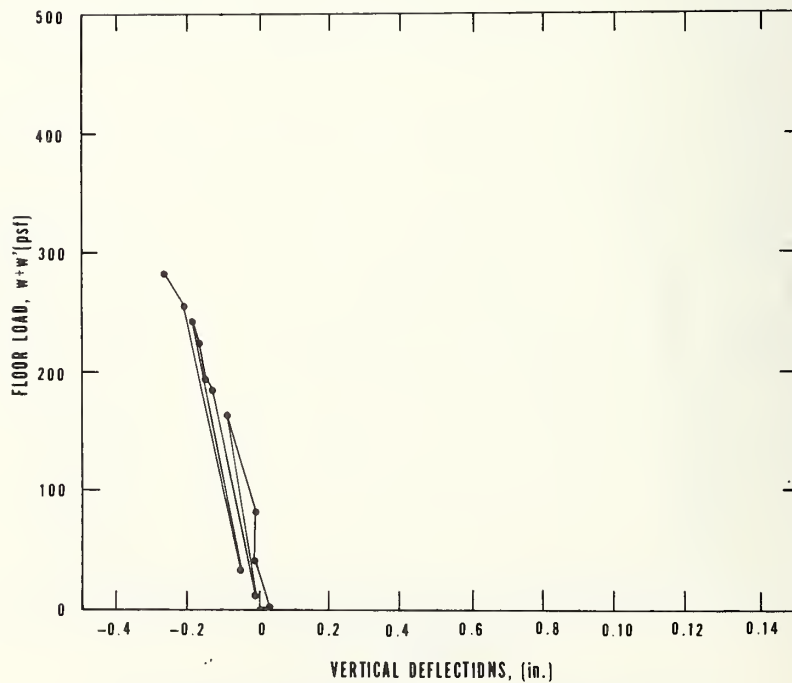
FIGURE A. 70. *Test No. 16A, floor loads versus beam deflection. (Output Channel 70)*
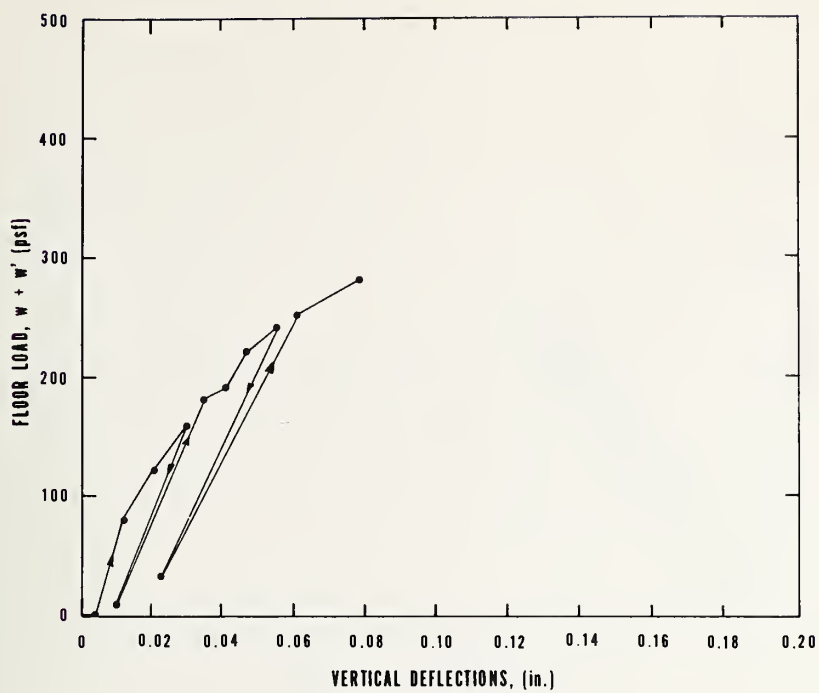
FIGURE A. 71. *Test No. 16A, floor loads versus beam deflection. (Output Channel 72)*
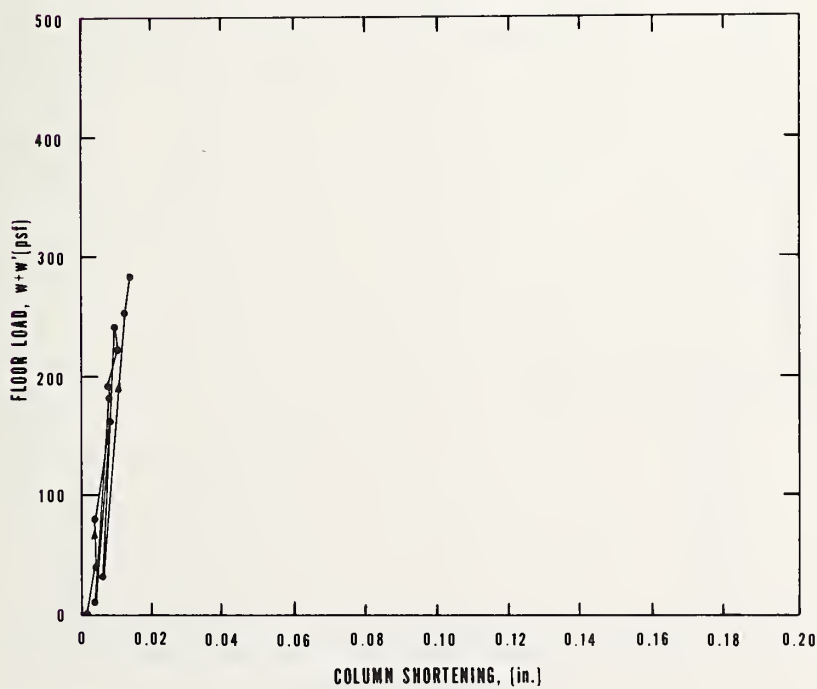
FIGURE A. 72. *Test No. 16A, floor loads versus column shortening. (Output Channel 73)*

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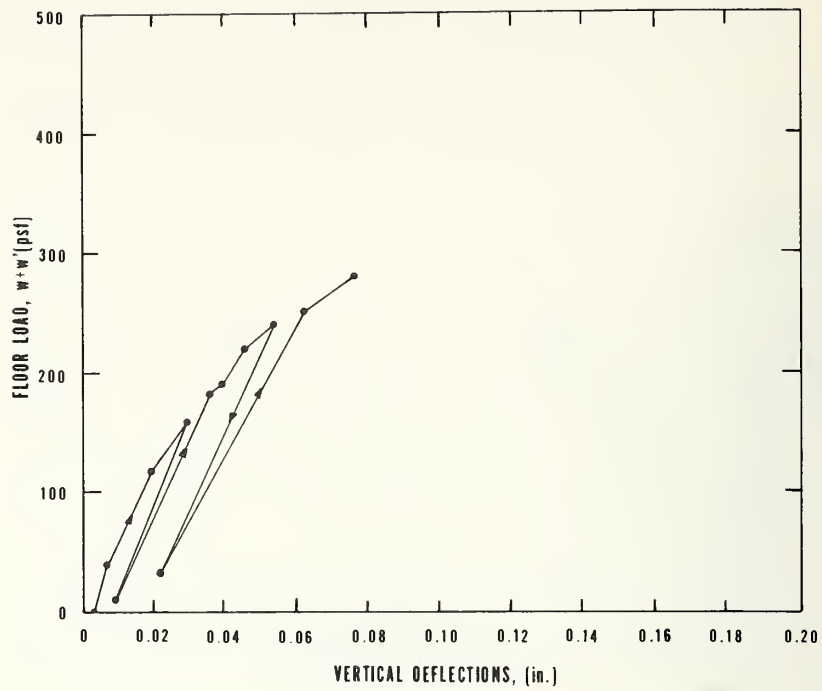
FIGURE A. 73. *Test No. 16A, floor loads versus slab deflection. (Output Channel 76)*
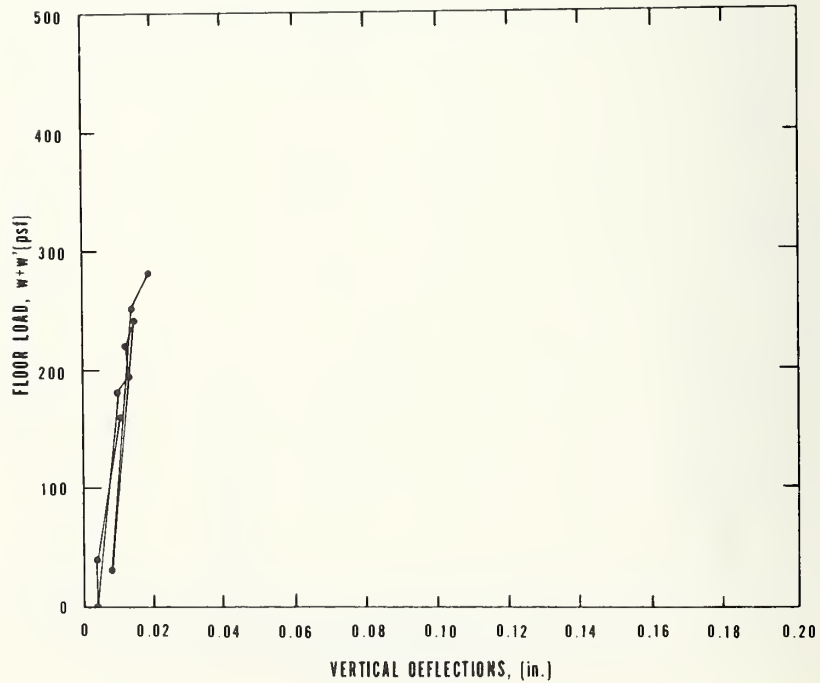
FIGURE A. 74. *Test No. 16A, floor loads versus beam deflection. (Output Channel 77)*
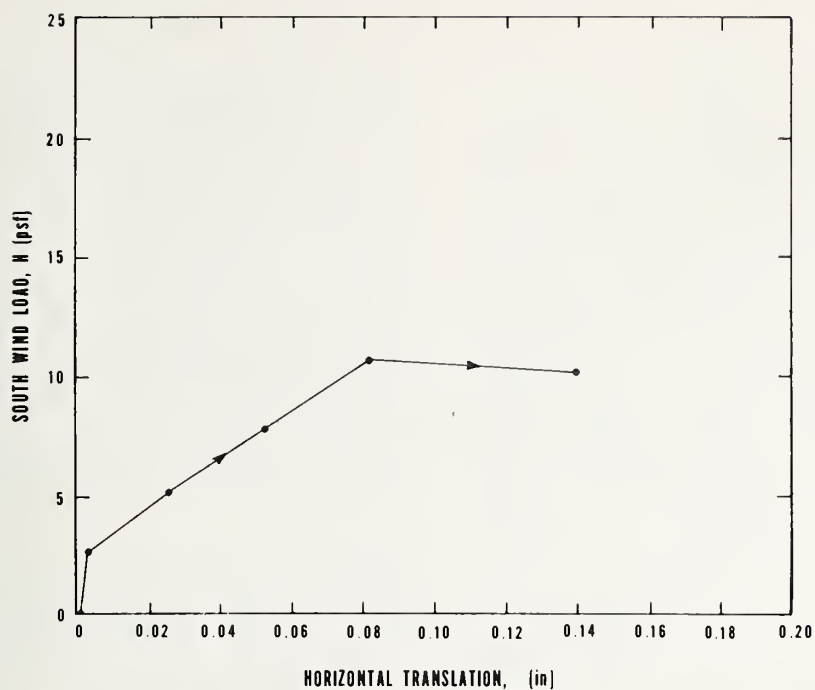
FIGURE A. 75. *Test No. 18, south wind load versus translation. (Output Channel 43)*
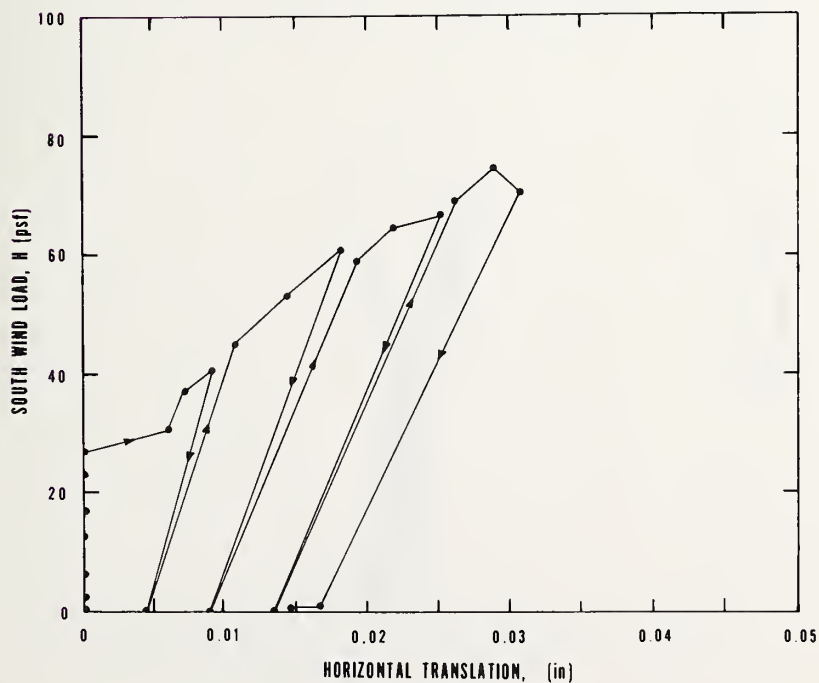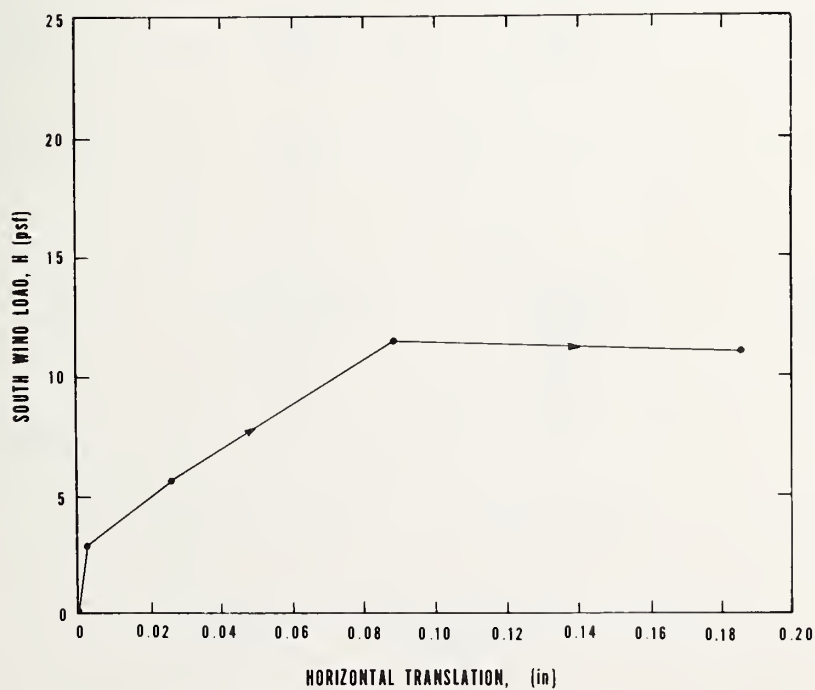

FIGURE A. 76. *Test No. 18, south wind load versus translation. (Output Channel 44)*

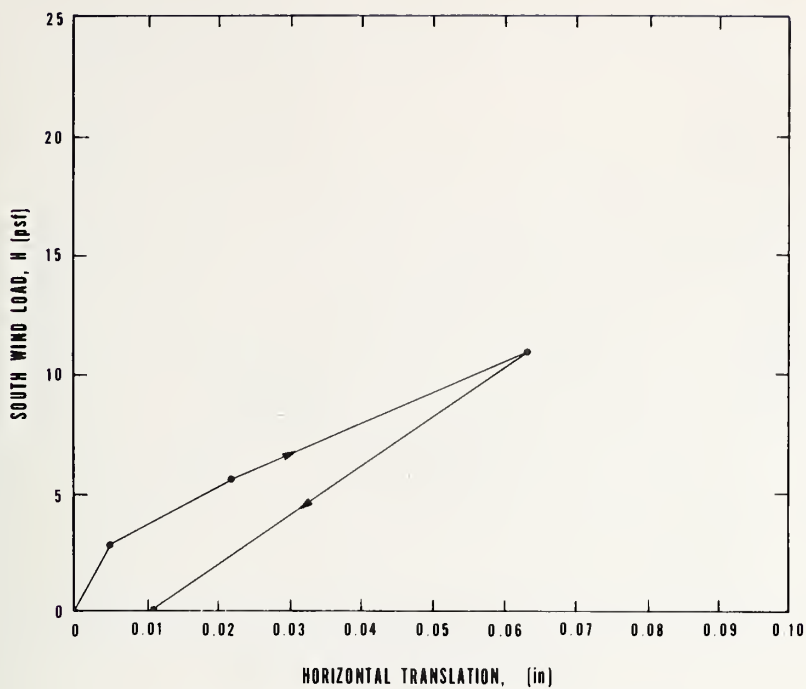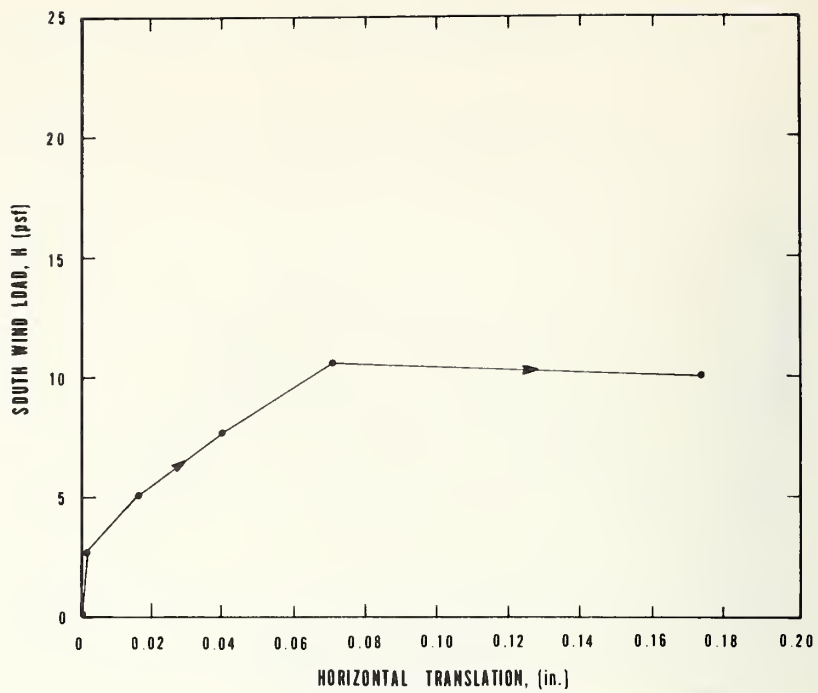FIGURE A. 77. *Test No. 18, south wind load versus translation. (Output Channel 45)*

Appendix B. Discussion of Column Load Simulation

Figure B.1 illustrates a comparison between column-beam connection moments due to simulated loads, and the moments anticipated in an actual structure. Figure B.1(a) shows the connection rotation direction due to actual loading of the middle span (12-ft span). Figure B.1(b), a detail of connection "A," shows the connection moments and axial forces caused by the actual loads. Figure B.1(c) illustrates the manner in which moments and forces actually applied by an upper story column may be exactly simulated, while figure B.1(d) shows the moments and forces applied in the test structure. The equations in figures B.1(c) and B.1(d) demonstrate qualitatively that the presence of moment Pa would:

(1) increase the negative moment in the main beam;

(2) decrease the lower story column moment.

This statement is generally valid for the case where column fixity in the test structure and in the real structure are equal. In this test it was decided to simulate foundation conditions by a "hinge" at the base of the column, since this was conservative in terms of column performance.

The introduction of this hinge decreases the column stiffness and therefore causes a decrease of the column connection moment (M_3' in fig. B.1).

To summarize, the upper story column load simulation, as applied, tends to increase the column moment, while the foundation simulation tends to decrease it. The net effect in the case of this test structure is illustrated in figure B.2. Figure B.2(a) shows the moment distribution in a three-story structure (comprising half of the real structure) caused by one (1) live load applied to the column strip. Figure B.2(b) shows the moment distribution due to the same loading in the test structure.

By comparing joint "A" in figure B.2(a) and figure B.2(b), it may be seen that at joint "A" in the test structure the column moment is *increased*. Also, the negative main beam moment is *decreased*, while the positive main beam moment at the center of the span is *increased*. Thus, the simulation of column loads in this test:

(1) Produced the most severe condition in the lower story column;

(2) Produced maximum dead and live load deflections and maximum positive beam moments, since midspan deflection and moment increases with decreasing negative moment at beam ends;

(3) Did *not* produce maximum negative moments at beam ends. However, negative moment capacities were adequately tested in Test 16 where the floor loads were increased to 370 psf.

(a)   (b)   (c)   (d)

M1 = M2 + M3

P x a = M2
M1 = M2 + M3

M1' = M3'
M1' < M1
M3' > M3
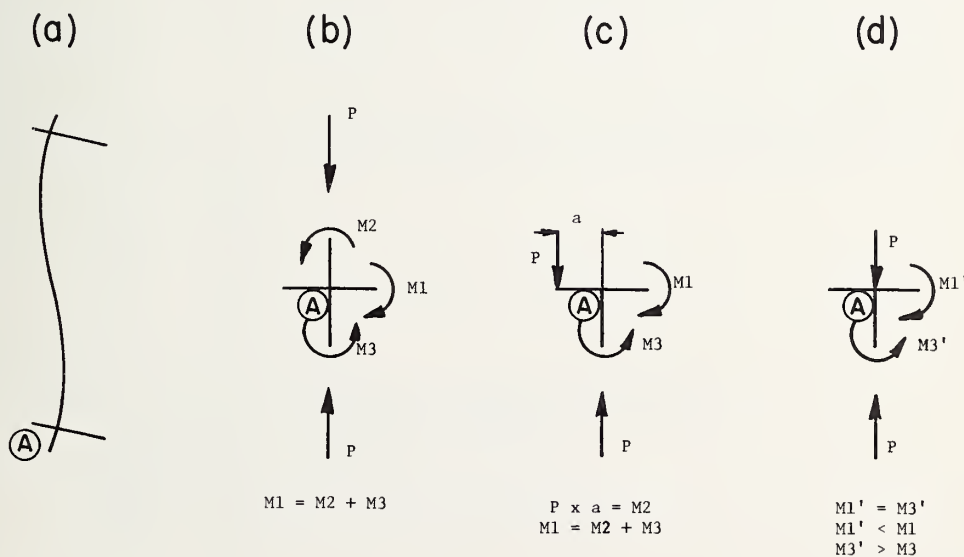
FIGURE B. 1. *Column simulation effect on connection moments.*

(a) Moment Distribution (kip-in.) due to one live load on three story structure.

(b) Moment Distribution (kip-in.) due to one live load on test structure.
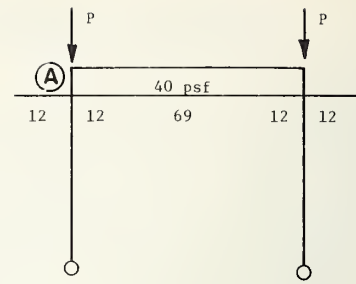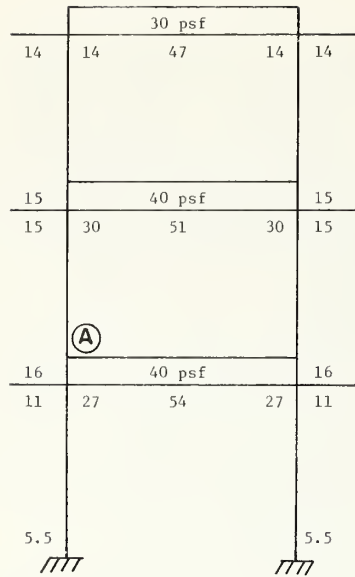
FIGURE B. 2. *Moment distribution.*

C. 1. Load Computations (refer to figure 6.1 and table 6.1)

I. Notations (see section 3)

D = Service dead load
L = service live load
H = service wind load
P = simulated 2d story column load
w, w' = simulated distributed dead & live loads exerted by air bags
Hs = simulated S-N wind load, at wall
Hs' = simulated S-N wind load, between walls
Hw = simulated W-E wind load

II. Summary

(a) *Simulated loads on test structures.*

	D	L	$.9D$	H	$.8H$	$1.1H$	$1.6H$	$D+L$	$1.3D+1.7L$	$1.25(1.5D+1.8L)$
P—kip	10	7	9					17	25	35
Hs—kip				4.05	3	4.5	5.7			
Hs'—kip				0.9	0.7	1	1.3			
Hw—kip				2.05	1.5	2.3	2.9			
w or w'—lb/ft^2	[a] 9.3	43						52	[b] 100	[b] 144

[a] Allowance for dead weight of partitions and fixtures.
[b] Includes dead weight of test structure (i.e., 43×1.07) multiplied by appropriate factor.

(b) *Simulated loads on lower column for component tests of columns.*

	$D+L$	$.9D+L$	$1.3D+1.7L$	$1.25(1.5D+1.8L)$	D	L
P—kip	25	24	37	51	14.4	11

(c) *Simulated load on main beam for fatigue test in kip per ram, (see fig. 10.23).*

$1D + 1L$ = 2.5
$1D + 1.5L$ = 3.125
$1D + 2L$ = 3.75
$1D + 2.5L$ = 4.375
$1D + 3L$ = 5.00

III. Vertical Load Computations

(1) *Conversion factor accounting for incomplete air bag coverage*

area of bags = 2 (9.915x15) = 297 ft^2
area of roof = 21.21 x 15.5 = 320 ft^2

$\dfrac{320}{297}$ = 1.07; convert all loads applied by bags.

(2) *Floor load elements (D) converted to "per square foot" on total floor area.*

(a) 4 in floor channels: 16.3 psf $\times \dfrac{18 \times 20}{19 \times 21.21}$ = 14.5 psf

(b) Topping slab: 2.25* in. $\times \dfrac{1 \text{ ft}}{12 \text{ in}} \times 110 \text{ lb/ft}^3$ = 21 psf

(c) Flooring and utilities (Mitchell)   = 1.75 psf
(d) Partitions (Mitchell)   = 7.00 psf
(e) Columns (prorated, see below)   = 1.55 psf
(f) Beams (prorated, see below)   = 5.78 psf
    3d floor unit load = a thru f 51.58 psf
    roof unit load = $a + b + f$ 41.28 psf
    added dead load on test structure.
      not transmitted by columns = $c + d$ 8.75 psf

(3) *Columns and beams prorated per unit area, say, 100 lb/ft³ concrete (includes reinforcing weight).*

T-beams: $\dfrac{43 \text{ in}^2}{144 \dfrac{\text{in}^2}{\text{ft}^2}} \times 19 \text{ ft} \times 100 \text{ lb/ft}^3 \times \dfrac{1}{200 \text{ ft}^2}$ = 2.86 psf ⎫

                                              5.78 psf

Ties: $\dfrac{42 \text{ in}^2}{144 \dfrac{\text{in}^2}{\text{ft}^2}} \times 20 \text{ ft} \times 100 \text{ lb/ft}^3 \times \dfrac{1}{200 \text{ ft}^2}$ = 2.92 psf ⎭

Columns: $\dfrac{(6 \times 4.75) \text{ in}^2}{144 \dfrac{\text{in}^2}{\text{ft}^2}} \times 2 \times 7.83 \text{ ft} \times 100 \text{ lb/ft}^3 \times \dfrac{1}{200 \text{ ft}^2}$ = 1.55 psf

(4) *Converted floor loads for air bags.*

(a) Live load = 40 psf $\times$ 1.07   = 43 psf
(b) Added dead load 1D = 8.75 $\times$ 1.07   = 9.35 psf
(c) Combinations: 1.3 D = 1.3 $\times$ 9.35 = 12.1
                 +0.3 $\times$ 1.07 $\times$ 42.8** = 13.7
                                     = 25.8
            1.7 L = 1.7 $\times$ 43   = 73
            1.3 D + 1.7 L = 98.8,   ≃ *100 psf*
            1.5 D = 1.5 $\times$ 9.35 = 14.1
                   +0.5 $\times$ 46*** = 23
                                       = 37.1
            1.8 L = 1.8 $\times$ 43   = 77.5
                                         114.6
           1.25 (1.5 D + 1.8 L) = 1.25 $\times$ 114.6   = 143.5,   ≃ *144 psf*

---

*2.25 in is used as average slab thickness, assuming that slab elevation over the top of the main beam is kept as 2.5 in.

**51.58 − 8.75 ∼ 42.8.

***42.8 × 1.07 = 46.

(5) *Simulated second-story column load.*

(a) *Dead Load:* Tributary floor area = $10.39 \times 9.5 = 99\ ft^2$

Column + partition: 99×8.55	= 850 lb
3d floor: 99×51.58	= 5,150 lb
roof: 99×41.28	= 4,050 lb
$1\ D$ =	10,050 lb, say *10 kip*

(b) *Live Load*: [40 psf (floor) + 30 psf (roof)] $\times$ 99   = 6,900 lb, say *7 kip*

(c) Combinations: $0.9D = 9$ kip; $1D = 10$ kip

$$1D + 1L = 17 \text{ kip}$$
$$1.3D + 1.7L = 25 \text{ kip}$$
$$1.25(1.5D + 1.8L) = 35 \text{ kip}$$

(6) Full lower story column load. (For component tests.)

(a) Dead Load:

2d story	5,150 lb (See 5a)
3d story	5,150 lb
roof	4,050 lb
$1D =$	14,350 lb, $\simeq$ 14 kip

(b) Live Load:

2d story = 99 sf $\times$ 40 lb/ft² =	3,960 lb
3d story = 99 sf $\times$ 40 lb/ft² =	3,960 lb
roof = 99 sf $\times$ 30 lb/ft² =	2,970 lb
$1L$ =	10,890 lb, $\simeq$ 11 kip

(c) Combinations:

$$1D + 1L = 25 \text{ kip}$$
$$1.3D + 1.7L = 37 \text{ kip}$$
$$1.25(1.5D + 1.8L) = 51 \text{ kip}$$

(7) Main beam loads for fatigue test.

¼ point loading: ½ the load resting on the beam, to be applied by each ram)

Area: = 65 ft²	
1 D: Slab: 6.25 ft $\times$ 7.5 ft (added slab width) $\times$ 21 psf =	945 lb (See (2))
Floor channel slab: 16.3 psf $\times$ 6.25 ft $\times$ 9 ft =	920 lb (See (2))
Walls, etc: 8.75 psf $\times$ 65 ft² =	570 lb (See (2))
1 dead load	2,435, lb $\simeq$ 2.5 kip

$1\ L = 40$ psf $\times$ 65 ft² = 2.6 kip, $\simeq$ 2.5 kip

¼ point loading—Kip:

$1D + 1L$	5.00
$1D + 1.5L$	6.25
$1D + 2L$	7.5
$1D + 2.5L$	8.75
$1D + 3L$	10.00

IV. Wind Loads

(1) South wall (Hs, Hs') (refer to fig. C.1 (1))

a = Story height $\times$ panel width = $8.62 \times 10.39 \times 90\ ft^2$

at point A-Hs. at point $B - Hs'$.

Hs is at firewall.

Hs' is midway between two firewalls.

Assume stiff floor distributes shear between two walls over width of structure, then test assembly carried ½ the wind load.

∴ at $H = 20$ psf: $Hs = \frac{1}{2} \times (4.5 \times 90)$ ft² $\times$ 20 lb/ft² = 4,050 lb = 4.05 kip

$Hs' = \frac{1}{2} \times 90$ ft² $\times$ 20 lb/ft² = 900 lb = 0.9 kip.

Therefore, 0.8 Hs $= 3.24$ kip*
1.25 Hs $= 5.1$ kip.

(2) West Wall (Hw) (refer to fig. C.1 (2))

$a' =$ story height $\times 9.5$ ft $= 8.62 \times 9.5 = 82$ ft^2

Assume $\frac{1}{2}$ the wind load carried by wall panel (conservative)

$\therefore$ at $H = 20$ psf: $Hw = \frac{1}{2} \times (2.5 \times 82)$ ft$^2 \times 20$ lb/ft$^2 = 2,050$ lb $= 2.05$ kip.

Therefore, 0.8 Hw $= 1.64$ kip**
1.25 Hw $= 2.56$ kip.

ASSUMPTIONS: (a) WIND FORCES RESISTED BY WALLS
(b) WIND FORCES BETWEEN WALLS
TRANSMITTED TO WALL BY STIFF FLOORS.

(a) SIDE VIEW (SOUTH WALL)
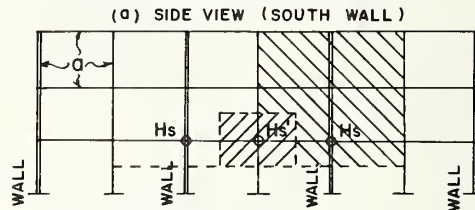
AREA TRIBUTARY TO Hs = 4.5a
AREA TRIBUTARY TO Hs'= a

(b) END VIEW (WEST WALL)
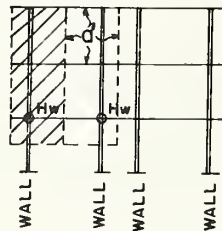
AREA TRIBUTARY TO Hw = 2.5a'

FIGURE C. 1. *Wind loads.*

C. 2. Load Schedules

For an explanation of symbols, refer to fig. 6.1, tables 6.1 and 6.2, appendix C(1)

I. Tests conducted on complete test structure with walls installed.

(a) Test 1: Column Loads to $0.9D$ ($0.9D$)

Start of test:	5/10/68 9:09 A.M.
Completion:	5/10/68 9:41 A.M.
Loading:	$P = 0.9D = 9$ kip
Increments:	9 in 1 kip increments

*3 kip was used in test, based on $H = 15$ psf rather than $0.8H$. This is not too low, considering the fact that a service wind load of 20 psf is extremely conservative for a built up area.

**1.6 kip was used in test, based on $H = 15$ psf.

(b) Test 2: Column loads of $0.9D$
South wind load to 25 psf
$(0.9D + .25\ H)$

Start of test:	5/10/68 11:50 A.M.
Completion:	5/10/68 1.02 P.M.
Loading:	$P = 0.9D - 9$ kip
	$Hs = 1.25H - 5.1$ kip
	$Hs' = 1.25H - 1.2$ kip
Increments:	Hs in 10 increments
	Hs' in 10 increments

NOTES: (1) Six cycles of loading and unloading were applied. Residual deflections were read 5 min after removal of all loads.
(2) P was applied initially and held constant.
(3) Hs and Hs' were applied simultaneously.

(c) Test 3: Column loads of $0.9D$
West wind load to 25 psf
$(0.9 + 1.25\ H)$

Start of test:	5/10/68 9:46 A.M.
Completion:	5/10/68 11:06 A.M.
Loading:	$P = 0.9D = 9$ kip
	$Hw = 1.25H = 2.6$ kip
Increments:	Hw in 10 increments

NOTES: (1) Six cycles of loading and unloading were applied at progressively larger loads. Residual deflections were read 5 min after removal of all loads.
(2) P was applied initially and held constant.

(d) Test 4: Column loads to $1.3D + 1.7L$
Major floor load to $1.3D + 1.7L$
$(1.3D + 1.7L)$

Start of test:	5/10/68 3:22 P.M.
Completion:	5/10/68 5:00 P.M.
Loading:	$P = 1.3D + 1.7L = 25$ kip
	$w = 1.3D + 1.7L = 100$ psf
Increments:	P: 1st increment 9 kip
	then 2 kip increments
	w in 10 psf increments

NOTES: (1) Unloaded at completion of test.
(2) w was applied in five cycles of loading and unloading at progressively larger loads. Residual deflections were read 5 min after removal of all loads.
(3) P was applied initially and held constant.

(e) Test 5: Column loads of $1.3D + 1.7L$
Major floor loads of $1.3D + 1.7L$
Loads sustained for 24 hr
$(1.3D + 1.7L)$

Start of test:	5/10/68 5:25 P.M.
Completion:	5/11/68 5:50 P.M.
Loading:	$P = 1.3D + 1.7L = 25$ kip
	sustained for 24 hrs

$$w = 1.3D + 1.7L = 100 \text{ psf}$$
$$\text{sustained for 24 hrs}$$

Increments: w in 20 psf increments

NOTES: (1) After unloading, an additional cycle of loading and unloading was applied. Additional reading of recovery was taken 24 hr after final unloading.
        (2) P was applied initially and held constant.

(f)   Test 6:   Column loads of $1.3D + 1.7L$
             Major floor load of $1.3D + 1.7L$
             $(1.3D + 1.7L)$

Start of test:	5/14/68 4:30 P.M.
Completion:	5/14/68 5:00 P.M.
Loading:	$P = 1.3D + 1.7L = 25$ kip
	$w = 1.3D + 1.7L - 100$ psf
Increments:	w in 5 increments

NOTE: P was applied initially and held constant.

(g)   Test 7:   Column loads of $1.3D + 1.7L$
             Major floor load of $1.3D + 1.7L$
             South wind load to 15 psf
             $(1.3D + 1.7L + 0.8H)$

Start of test:	5/14/68 5:02 P.M.
Completion:	5/14/68 6:13 P.M.
Loading:	$P + 1.3D + 1.7L = 25$ kip
	$w = 1.3D + 1.7L - 100$ psf
	$Hs = 0.8H = 3$ kip
	$Hs' = 0.8H = 0.7$ kip
Increments:	$Hs + Hs'$ in 10 increments

NOTES: (1) P and w were maintained from previous test and held constant throughout the test.
        (2) Six cycles of loading and unloading at progressively larger loads were applied for Hs and Hs'.

(h)   Test 8:   Column loads of $1.3D + 1.7L$
             Major floor loads of $1.3D + 1.7L$
             West wind load to 15 psf
             $(1.3D + 1.7L + 0.8L)$

Start of test:	5/14/68 3:10 P.M.
Completion:	5/14/68 4:15 P.M.
Loading:	$P = 1.3D + 1.7L = 25$ kip
	$w = 1.3D + 1.7L = 100$ psf
	$Hw = 0.8H = 1.5$ kip
Increments:	P and w applied in 1 increment
	Hw applied in 10 increments

NOTES: (1) P and w were applied initially and held constant.
        (2) Six cycles of loading and unloading were applied for Hw at progressively larger loads. Reading of residual deflection was taken 5 min after removal of all loads.

(i) Test 9: Column loads of $1D$
     Major floor load to 160 psf

Test 9-A: Column loads of $1D$
     Major floor load to 160 psf
     Minor floor load to 160 psf
     $(1D + 3.5L)$

Start of test:	5/15/68 12:10 P.M.
Completion:	5/15/68 2:00 P.M.
Loading:	$P = 1D + 1L = 17$ kip
	$w' = w = 1D + 3.5L = 160$ psf
Increments:	w and $w' =$ one increment of 80 psf, followed by 10 psf increments

NOTES: (1) Four cycles of loading and unloading were applied for w and w' at progressively higher loads.
(2) w and $w' + w$ were applied alternatively. For the purpose of data presentation then alternate load applications have been designated as Tests 9 and 9A. Test 9 is taken as though load w was applied alone, while Test 9-A is taken as though w and w' were applied simultaneously.
(3) P was applied initially and held constant throughout the test.

(j) Test 10: Column loads of $0.9D$
     South wind load to 60 psf
     $(0.9D + 3H)$

Start of test:	5/16/68 9:30 A.M.
Completion:	5/16/68 10:40 A.M.
Loading:	$P = 0.9D = 9$ kip
	$Hs = 12$ kip
Increments:	Hs in 1 kip increments

NOTES: (1) Three cycles of loading and unloading with progressively increased loads were applied. Loading of residual deflection was taken 5 min after removal of all loads.
(2) Walls were racked until 0.35 in drift was reached. Further racking was discontinued to preserve the integrity of the beam-column joints.
(3) P was applied initially and held constant.

(k) Test 11: Column loads of $0.9D$
     West wind load to 67 psf
     $(0.9D + 3.35H)$

Start of test:	5/16/68 11:15 A.M.
Completion:	5/16/68 12:00 P.M.
Loading:	$P = 0.9D = 9$ kip
	$Hw = 7$ kip
Increments:	Hw in 0.5 kip increments

NOTES: (1) Four cycles of loading and unloading were applied at progressively increased loads. Reading of residual deflection was taken 5 min after removal of all loads.
(2) Walls were racked until 0.3 in drift was reached. Further racking was discontinued to preserve the integrity of the column-beam joint.
(3) P was applied initially and held constant. After this test, all the walls were removed.

II. Tests conducted on the test structure after removal of the walls.

(a) Test 12: Column load of $1.3D + 1.7L$
     Major flood load to $1.3D + 1.7L$
     Rollers under column loads oriented to permit east-west sway
     $(1.3D + 1.7L)$

Start of test:	5/21/68 9:12 A.M.
Completion:	5/21/68 9:30 A.M.
Loading:	$P = 1.3D + 1.7L = 25$ kip
	$w = 1.3D + 1.7L = 100$ psf
Increments:	w in 20 psf increments

NOTES: (1) Rollers under P were oriented to permit E-W sway.
       (2) P was applied initially and held constant.
       (3) Reading of residual deflections was taken 5 minutes after removal of all loads.

(b) Test 12-A: Column loads of $1.3D + 1.7L$
     Major floor load to $1.3D + 1.7L$
     Rollers under columns oriented to permit north-south sway.
     $(1.3D + 1.7L)$

Start of test:	5/21/68 11:12 A.M.
Completion:	5/21/68 11:45 A.M.
Loading:	$P = 1.3D + 1.7L = 25$ kip
	$w = 1.3D + 1.7L = 100$ psf
Increments	w in 20 psf increments

NOTES: (1) Rollers under P oriented to permit N-S sway.
       P was applied initially and held constant.
       (3) Reading of residual deflections was taken 5 minutes after all loads were removed.

(c) Test 13: Column loads of $1.3D + 1.7L$
     Major floor load of $1.3D + 1.7L$
     Minor floor load of $1.3D + 1.7L$
     Rollers under column loads oriented to permit east-west sway.
     $(1.3D + 1.7L)$

Start of test:	5/21/68 9:32 A.M.
Completion:	5/21/68 11:09 A.M.
Loading:	$P = 1.3D + 1.7L = 25$ kip
	$w' = w\ 1.3D + 1.7L = 100$ psf
Increments:	$w' + w$ in 20 psf increments

NOTES: (1) P was maintained from preceding test and held constant.
       (2) ROLLERS oriented to permit E-W sway.
       (3) Reading of residual deflections was taken 5 min after all loads were removed.

(d) Test 13-A: Column loads of $1.3D + 1.7L$
     Major floor load of $1.3D + 1.7L$
     Minor floor load of $1.3D + 1.7L$
     Rollers under column loads oriented to permit north-south sway
     $(1.3D + 1.7L)$

Start of test:	5/21/68 11:00 A.M.
Completion:	5/21/68 12:28 P.M.
Loading:	$P = 1.3D + 1.7L = 25$ kip
	$w' = w = 1.3D + 1.7L = 100$ psf
Increments:	$w' + w$ in 20 psf increments

NOTES: (1) Rollers under P oriented to permit N-S sway.
        (2) P was maintained from preceding test and held constant.
        (3) Reading of residual deflections was taken 5 min after all loads were removed.

(e) Test 14: Column loads of $0.9D$
           South wind load of 10 psf
           $(0.9D + 0.5H)$

Start of test:	5/21/68 2:23 P.M.
Completion:	5/21/68 2:58 P.M.
Loading:	$P = 0.9D = 9$ kip
	$Hs = 2$ kip
Increments:	Hs in 0.5 kip increments

NOTES: (1) Racking load was carried to 2 kip and discontinued to prevent damage to beam column connections.
        (2) P was applied initially and held constant.

(f) Test 15: Column load of $0.9D$
           West wind load of 16.5 psf
           $(0.9D + 0.8H)$

Start of test:	5/21/68 3:05 P.M.
Completion:	5/21/68 3:55 P.M.
Loading:	$P = 0.9D = 9$ kip
	$Hw = 2.5$ kip
Increments:	Hw in 0.5 kip increments

NOTES: (1) Racking load was carried to 2.5 kips and discontinued to prevent damage to the column-beam connection.
        (2) P was applied initially and held constant.

(g) Test 16: Column loads of $1D$
           Major floor load to 370 psf
           $(1D + 8.4L)$

      Test 16-A: Column loads of $1D$
               Major floor load to 280 psf
               Minor floor load to 280 psf
               $(1D + 6.3L)$

Start of test:	5/21/68 4:05 P.M.
Completion:	5/21/68 7:15 P.M.
Loading:	$P = 1D + 1L = 17$ kip
	$w = 370$ psf
	$w' = 280$ psf

Increments: w and w': 40 psf increments to
                                                              160 psf
                                                     20 psf increments
                                                          thereafter

NOTES: (1) Loads w and $w + w'$ were alternately applied. w' was discontinued at 280 psf, recognizing that w alone was more critical. Loading was discontinued at $w = 370$ psf due to failure of the loading system.
  (2) In Test 16-A, three cycles of loading and unloading were applied at progressively larger loads; in Test 16, four cycles were applied.
  (3) Tests 16 and 16-A were performed simultaneously; w and $w' + w$ were applied alternately. For the purpose of data presentation, then alternate load applications have been designated as Tests 16 and 16-A. Test 16 is taken as though load w was applied alone, while Test 16-A is taken as though w and w' were applied simultaneously.
  (4) P was applied at the beginning and held constant throughout the test.

(h) Test 17: Column loads to 60 kip on four outer columns
              $(1D + 7L)$

Start of test:	5/22/68 10:30 A.M.
Completion:	5/22/68 10:42 A.M.
Loading:	$P = 60$ kip $1D + 7L$
Increment:	Continuous increase of load.

NOTES: (1) Only the four outside columns were loaded because of test frame capacity.
  (2) No deflection readings were taken in this test.

(i) Test 18: Column loads of $0.9D$
              South wind load to 10.5 psf
              $(0.9D + 0.5H)$

Start of test:	5/22/68 10:50 A.M.
Completion:	5/22/68 11:20 A.M.
Loading:	$P = 0.9D = 9$ kip
	$Hs = 2$ kip
Increments:	Hs in 0.5 kip increments

NOTES: (1) Racking load could not be further increased.
  (2) Loud crack was heard in S-E column at maximum deflection.

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[1] Headquarters and Laboratories at Gaithersburg, Maryland, unless otherwise noted; mailing address Washington, D.C. 20234.
[2] Located at Boulder, Colorado 80302.
[3] Located at 5285 Port Royal Road, Springfield, Virginia 22151.

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