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NATIONAL BUREAU OF STANDARDS REPORT

STRUCTURAL PERFORMANCE TEST:

neal mitchell housing system

Interim Report

prepared for

Department of Housing and Urban Development

~~Not for publication or for reference.~~

NBS

U.S. DEPARTMENT OF COMMERCE

NATIONAL BUREAU OF STANDARDS

REPORT NUMBER 9866

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NATIONAL BUREAU OF STANDARDS REPORT

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STRUCTURAL PERFORMANCE TEST

OF THE

NEIL MITCHELL HOUSING SYSTEM

INTERIM REPORT

Prepared For
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By
Building Research Division
The Institute For Applied Technology

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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. They have taken place in small carefully considered increments. Each incremental step has been based upon extensive laboratory and analytical investigation. Most of this progress has been based upon component testing and upon grossly simplified and conservative analyses which do not correctly 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 overlooked. As a result, in those few cases where tests on total building systems have been performed, results have been obtained which indicate strength and rigidity far in excess of that predicted by either component testing or by conventional simplified analysis. Strict reliance upon these conventional concepts inhibits innovative solutions to the building problem.

One solution to this dilemma would be full-scale system tests coupled with rigorous 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 rigorous 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 of large scale subsystem tests in the laboratory which simulate total system behaviour. 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

The Neal Mitchell Framing System is an innovative building system which is erected through the assemblage of precast concrete components. It uses five plant-produced components which can be easily erected with a minimum of equipment and onsite labor. The system is presently being proposed for construction as a part of the Phoenix Housing Project in Detroit, under a grant from the Department of Housing and Urban Development's Low-Income Housing Demonstration Program. The Mitchell System contains a number of deliberate innovative departures from presently accepted construction practice. In most cases, these departures are based upon rational computation; however, because of their innovative nature they are not supported by either extensive inservice experience or by intensive laboratory experimentation.

In order to ascertain the structural adequacy of this system, HUD has sought the advice of a special advisory panel consisting of Dr. Michael Soteriades of the National Academy of Science, Mr. William Heitmann of the U.S. Army Corp of Engineers, and Dr. Edward O. Pfrang of the National Bureau of Standards, and has contracted with the National Bureau of Standards for an intensive laboratory investigation of the systems structural response. Recognizing that the Mitchell System was designed as a system of integrated components with a high degree of interaction, it was deemed advisable to direct the main thrust of this experimental study at system response rather than at component response.

This report summarizes the results obtained from a comprehensive series of load tests on a full-scale portion of the Mitchell System. The test portion of the structure was constructed in the laboratory by Neal Mitchell Associates Inc. in accordance with plans and specification prepared by them for the Phoenix Housing Project in Detroit. The test structure was one story in height and was chosen and loaded in a manner that simulated the structural response of a three-story high 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, from short and long term column tests, and from repeated load tests on the main structural beams of the system.

Although the primary motivation for the investigation was to determine the structural adequacy of the Mitchell System and the applicability of performance testing to such an innovative system, it was recognized early in the development of this program that the testing of a complex structural subsystem offered a great deal of additional potential for providing longer-range information concerning structural interaction.

3. Notation

The following notation is adopted for use throughout this report:

3.1 Service Loads

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

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

P = 2nd story column loads
Hw = West wind load
Hs = South wind load at firewalls
H_s' = South wind load between firewalls
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 deflections

3.4 Lengths

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

4. Performance Criteria

4.1 Introduction

Some criteria for performance testing have been developed (ACI Committees 318-2 427, N. Y. State Building Code); however, these criteria are not sufficient for the evaluation of comprehensive building systems. The criteria developed for the purpose of this evaluation incorporates the available information supplemented by new criteria where necessary.

The performance criteria used in this report are presented and discussed in the following 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.

4.2 Definitions

4.2.1 Length of Members

The length of horizontal members is taken as the distance between the centerlines of their support 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

Deflections at a point on a structure are taken as the magnitude of the maximum displacement of the point as a result of a specific loading condition, measured in the direction indicated. Horizontal deflections were measured in a direction perpendicular to the direction of the wall containing the point of measurement. Gross deflection is taken as the total maximum displacement at a point.

Net deflection is that part of the displacement of a point which is attributable solely to the deformation of a member between its supports.

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 4.4.4, occurring within 10 minutes* 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 performance test the following Test Criteria must be met.

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.

*The time limit in 4.2.4 was introduced in order to distinguish between long term creep and a deformation occurring over a relatively short period of time.

Commentary on Criterion 4.3.1

A similar criterion is developed in ACI 318-63; however, the emphasis here 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 than the conditions in the full-scale structure which provide the least margin of safety. Foundation conditions will be simulated in a manner representing the most adverse conditions that may exist in a complete structure in the field.

Commentary on Criterion 4.3.2

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

4.4 Performance Criteria

4.4.1 Lateral Deflection under Dead and Wind Loads

At a load level of 0.9 dead + 1.25 wind (0.9 D + 1.25H) the following lateral deflection shall not be exceeded:

$$Dh \leq 0.002h$$

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.

There has been limited experience with high-rise apartment structures which indicates that when such a structure is designed under a condition which permits lateral deflection in excess of h/400 to h/500, discomfort is experienced by some of the occupants under severe wind conditions. Although it is probably extremely conservative for low-rise structures, this deflection limitation is adopted here for lack of a more realistic criterion.

4.4.2 Lateral Deflection under Dead, Live and Wind Load

At a load level of 1.3D + 1.7L + 0.8H the following deflection shall not be exceeded:

$$Dh \leq 0.002h$$

Commentary on Criterion 4.4.2

Although the most critical loading with respect to lateral deflection of a structure is generally under minimum vertical and maximum lateral load, it is also possible that maximum vertical combined with lateral load can be more critical. This criterion is a check against such a possibility.

It would be unrealistically conservative to consider maximum vertical load acting simultaneously with maximum lateral load; thus a lesser lateral load is adopted for this criterion.

4.4.3 Sustained Load Deflection

At a load of 1.3D + 1.7L, sustained for 24 hours, the following deflections shall not be exceeded:

(a) $dv \leq \frac{1}{480} \times \left(\frac{1.3D + 1.7L}{D + L} \right)$

(b) $Dh \leq 0.002h$

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

(c) If $dv > \frac{1^2}{20,000t}$ dvr (Residual vertical deflection) $0.25\ dv$

if $dv \leq \frac{1^2}{20,000t}$ $dvr \leq \frac{1^2}{80,000t}$

(d) if $dv > \frac{1^2}{20,000t}$ $Dvr \quad 0.25\ Dv$

if $dv \leq \frac{1^2}{20,000t}$ $Dvr \leq dvr + 0.25\ (Dv - dv)$

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

<u>Commentary on Criterion 4.4.3</u>

Criterion 4.4.3 (a) is based on the proposition that l/480 is a reasonable maximum allowable deflection under service loads. Prevailing codes usually set l/360 for a deflection limitation. However several studies have indicated that this deflection is excessive in terms of user comfort. The l/480 deflection limitation is proposed as a reasonable compromise based upon the present inadequate state of knowledge in this area.

Criterion 4.4.3 (b) is similar to Criteria 4.4.1 and 4.4.2 which have been discussed earlier.

Criterion 4.4.3 (c) requires a 75% recovery of vertical deflections. This guards against a structure which experiences significant irrecoverable inelastic deformation in each cycle of loading which might lead to progressive incremental collapse. The 75% recovery requirement is relaxed for very stiff structural systems, since there is invariably some small irrecoverable deformation in all structures.

Criterion 4.4.3 (d) requires 75% recovery of deflections in excess of "net" deflections. These deflections are primarily due to deformations of columns and/or walls and this criterion assures that they must remain essentially elastic.

4.4.4 Vertical deflection at service loads

At a load of 1D +1L the following deflection shall not be exceeded

$$dv \leq 1/480$$

Commentary on Criterion 4.4.4

See Commentary on Criterion 4.4.3.

4.4.5 Ultimate Strength

The Structure or any portion thereof shall not fail at a load smaller than 1.25 (1.5D + 1.8L)

Commentary on Criterion 4.3.7

This is a criterion for ultimate load which has reasonable universality.

In the case of concrete structures it is assumed that ϕ, the standard deviation from average ultimate strength, is about 20%. In absence of a statistical sample of any size, it is reasonably safe to assume that the laboratory sample has a strength of $1 + \sigma$. It is therefore realistic to require that the laboratory sample be capable of withstanding a load of $1/(1 - \sigma)$ of the design ultimate load.

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 Test Criteria

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.

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. Foundation conditions will be simulated in a manner representing the most adverse conditions that may exist in a complete structure in the field.

4.4 Performance Criteria

4.4.1 Lateral Deflection under Dead and Wind Load

At $0.9D + 1.25H$: $Dh \leq 0.002$

4.4.2 Lateral Deflection under Dead, Live and Wind Load

At $1.3D + 1.7L + 0.8H$: $Dh \leq 0.002$

4.4.3 Sustained Load Deflection

At $1.3D + 1.7L$ Sustained for 24 hours:

(a) $dv \leq \frac{1}{480} (\frac{1.3D + 1.7L}{D + L})$

(b) $Dh \leq 0.002h$

Residual deflections measured not later than 24 hours after removal of loads:

(c) if $dv > \frac{l^2}{20{,}000t}$ $dvr \leq 0.25dv$

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

(d) if $dv > \frac{l^2}{20{,}000t}$ $Dvr = 0.25Dv$

if $dv \leq \frac{l^2}{20{,}000t}$ $Dvr = dvr + 0.25\ (Dv - dv)$

4.4.4 Vertical Deflection at Service Loads

At D + L: $dv \leq 1/480 \times l$

4.4.5 Strength

The Structure or any portion thereof shall not fail at a load smaller than 1.25 (1.5D + 1.8L).

5. Test Structure

The structure as erected and tested in the laboratory was a full-scale section of a modular building system. It was designed and constructed by Neal Mitchell Associates Inc. in accordance with 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.

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 represented in this test.

5.1. The proposed Structure

A typical complete structure is illustrated in Fig. 5.1. The proposed structure consists of:

1. Pre-cast components
2. Cast-in-place topping slabs.
3. Gypsum walls, and
4. Foundations, grade-beams and slabs on grade.

5.1.1. <u>The pre-cast elements</u>

The pre-cast elements of the proposed structure are: (1) columns, (2) main beams, (3) tie beams, and (4) floor channels.

The assembled structural frame is illustrated in Figs. 5.2, and 5.3. Figures 5.4 through 5.11 show the design drawings for the pre-cast components.

Typical column details are illustrated by Fig. 5.4 and 5.5. Main beam sections and details are shown by Figs. 5.6, 5.7, 5.8 and 5.9. Tie beams are illustrated by Figs. 5.10 and 5.11. All components are pre-cast of cellular concrete with light-weight aggregate. The nominal wet density of the concrete is 95 pcf. Specified nominal 28-day strength is 3,500 psi. The wet density of this concrete is controlled by the addition of a preformed foam at the time of mixing. Reinforcing bars are ASTM-A61 steel (60 ksi)*.

The floor channel details are shown in Fig. 5.12. These elements are standard commercially available pre-cast concrete rooftile.

*Plans and specifications for the Neal Mitchell Housing System permit the option of using 50 ksi or 60ksi steel. The steel used in the test structure had a nominal yield of 60ksi. See Section II for the actual properties.

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

Reinforcement consists of a #5 deformed structural grade (ASTM-A15) steel bar in each leg of the channel and a 34-1412 wire mesh (ASTM-A185) in the back of the channel, with the 14 gage wire in the longitudinal direction.

The top of the channels 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 "nominal" thickness of two ins.

Concrete is made of 3/4" maximum size lightweight aggregate, with a weight of 110 pcf 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 2- # 4 bars as shown in Fig. 5.9. This reinforcement is ASTM-A61 steel.

5.1.3 The Gypsum Walls

There are three kinds of gypsum walls:

(1) Fire walls

(2) Exterior walls, and

(3) Interior walls.

5.1.3.(1) Fire walls

Figure 5-13 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 the 12' space between two columns and have no openings in this space. The full width of a building will therefore contain two such uninterrupted 12' walls in every second bay.

The fire walls are standard dry-wall construction. Channels are attached to the concrete members with power-actuated fasteners at a 6" to 8" spacing. Metal studs (2 1/2" x 25 ga.) are spaced 16" on center. Wallboards on either side of the metal studs consist of one 3/8" gypsum backing board (ASTM-C442) and one 1/2" gypsum wall board (ASTM-C36).

The wall boards are fastened to the studs by screws spaced 8" - 12" o.c. which is a closer spacing than that used in standard practice. The detail of the actual fire wall installation in the test structure is illustrated in figure 5.14.

5.1.3.(2) Exterior Walls

Fig. 5.15 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. Each building will thus have two exterior walls. Exterior walls are located in the outer rows of columns and fill the 10' space between columns. The walls are not continuous and each panel may contain a door or a window (aluminum framing).

Exterior walls are standard dry wall construction. Channels and studs are as in the fire walls. Facing consists of 1/2" gypsum wall boards (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.

5.1.3.(3) Interior Walls

"Interior walls" as defined here extend along the two interior rows of columns in the long direction of the building. Each building thus has two interior walls in the long directions. These walls fill the 10' panels between columns. A three ft. door (wooden framing) may be expected in every second panel.

Several types of interior walls are used in the Mitchell System; of these the standard "structicore" partition wall construction was deemed to have the least resistance to lateral load and was thus chosen for the laboratory structure. Figure 5.16 shows a typical interior "structicore" wall partially dismantled.

5.1.4. Foundations, Grade Beams and Slabs on Grade

Foundation plans are shown in Fig. 5.17.

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.

5.1.5. Connections

A design drawing of the connection detail is shown in Fig. 5.18. Isometric views are shown in Figs. 5.19 and 5.20.

After joints are connected as shown, they are grouted with a grout of the following mix:

1 part type 1 cement
2 parts masonry sand
3 ounces of latex admixture per lb of cement

5.2 The Test Structure

5.2.1. The Test structure before and after installation of the walls is illustrated in Figs. 5.21 and 5.22 respectively. It comprises a part of the total structure, made up of full-scale components and erected in the laboratory.

The performance of the total 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, and

(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 consisted of:

(1) Pre-cast components
(2) A cast-in-place topping slab
(3) Walls
(4) A cast-in-place floor slab.

5.2.2.(1) <u>Pre-cast components</u> consisted of:

(a) Six precast columns, which are identical to the lower story columns in the proposed structure, except that they were shortened to a length of 8' - 5" since no embedding in foundations was included (Figs. 5.4 and 5.5).

(b) Three main beams (See fig. 5.6 through 5.9).

(c) Four tie beams (See fig. 5.10 and 5.11).

(d) Eight 2' wide and two 1' wide floor Channels, which were placed at the north and south edge of the structure (See fig. 5.12).

5.2.2.(2) The cast-in-place topping slab (1st story) in the test structure had an average thickness of 2 1/2 inches $\pm$1/8". This slab was originally cast too thick and had to be reduced to its final thickness by terrazzo grinders.

5.2.2.(3) <u>Walls</u>

The test structure had the following walls:

(a) East and west walls were "fire walls" as described in Section 5.1.3.(1).

(b) The south wall was an exterior wall as described in Section 3.1.3(2) except that the exterior siding was omitted.

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

Insulation between walls and vapor seals 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 on the safe side.

Each panel in the south wall contained a 5' x 7' aluminum door frame 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' x 7' wooden door frame on its east Side. This simulates field conditions.

5.2.2.(4) The 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 was subsequently post-tensioned against the laboratory floor by four 1 1/2" diameter bolts in order to prevent sliding due to lateral test forces applied to the structure.

Concrete had a nominal 28-day strength of 3,000 psi.

Reinforcement in the center of the slab consisted of a 66-1010 mesh (ASTM-A185).

Column seats were lined by 1/2" asphalt-impregnated fiber board to permit column rotation at the base and a 1/8" thick neoprene sheet was inserted between the column base and the laboratory floor. Channels for the wall systems were attached by power-actuated fasteners to this slab and into the structural frame.

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, 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 representative portion of the structure and to test it in a manner that simulated the performance of the total structural system.

The load program to which the 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

The test structure with the testing equipment installed is shown in Figures 5.23 and 5.24. The test portion of the structure would in the real structure be connected to the remainder by:

(1) Columns
(2) Abutting tie beams and main beams
(3) A continuous topping slab; and
(4) 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 at the upper stories.

In reality the columns would 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.

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 total 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 jacks at the centerline of the lower story columns as illustrated in Fig. 5.25. Rollers were inserted perpendicular to the direction of racking, to prevent the formation of 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 jacks 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 Cantilever Beams

Cantilever 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. 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. Since maximum negative moments will occur on the middle support of a span that is continuous over three supports, 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 specimen, wind loads equal to 1/2 the wind loads generated by the entire contributary portion of the building are imparted at the top of each wall by a jack load, as illustrated by Fig. 5.26. In the case of the south-north direction, a wind load is also applied at the top of the column in between the two fire walls. Due to the stiffness of the floor system these wind loads will have a net effect equal to the effect that may be expected on a structure in the field. The reason for applying only 1/2 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 south-north direction and to more than two wall panels in the west-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 5 times after these walls were removed. As will be noted later, the loading applied in Test #9 would more than compensate 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' depth below grade for exterior column, and to 3' depth below the top of the floor slab for interior columns (See Fig. 5.17). Exterior column footings are also tied into the perimeter wall for added fixity. This configuration will provide 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" thick asphalt-impregnated fiber board expansion joint against which the floor slab was cast, thus providing a detail similar to that of the real structure. The base of the column was set on a

1/8" thick neoprene bearing pad which rested on the laboratory test floor. The resultant column connection was a conservative simulation of the real structure.

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. 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.

Horizontal live loads were applied by horizontal 10-ton jacks as illustrated in Fig. 5.27. The validity of wind load simulation was discussed in Section 5.3.1.(4).

Computations substantiating the magnitude of applied loads are presented in Section 6 and Appendix C of this report.

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, as applicable to the Detroit area. The following unit "service" loads were used:

Occupancy Loads (floor) - 40 psf

Snow Loads (roof) - 30 psf

Wind Loads (walls) - 20 psf

6.3 Load Schedule

Figure 6.1 shows schematically how the test loads were applied to the structure. Table 6.1 explains the symbols used for the computations relating to these magnitudes as shown in Appendix C. Table 6.2 summarizes the magnitude of test loads, used in the different tests.

Tests were conducted on the test structure with walls installed, and subsequently on the same structure after the walls were removed.

Load tests were conducted between May 10, 1968, and May 22, 1968, and are listed hereafter:

6.3.1 <u>Tests conducted on the structure with walls installed:</u>

Test #1: column loads to 0.9D
(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 + 1.25H)

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)

Test #9: column loads of 1D

major floor load to 160 psf

(1D + 3.7L)

Test #9-A: column loads of 1D

major floor load to 160 psf

minor floor load to 160 psf

(1D + 3.7L)

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 67 psf

(0.9D + 3.35H)

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 of 1.3D + 1.7L

rollers under column loads oriented to permit north - south sway (1.3D + 1.7L)

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)

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)

Test #14: column loads of 0.9D

south wind load of 10psf

(0.9D + 0.5H)

Test #15: column loads of 0.9D

west wind load of 16.5 psf

(0.pD + 0.8H)

Test #16: column loads of 1D

major floor load of 370 psf

(1D + 8.4L)

Test #16-A: column load of 1D

major floor load of 280 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 ninety-eight 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. Gage No. 52, 54, 55, 56, and 57 designate LVDT's having readout intervals of 0.0001 in., while 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 numbers 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 No.'s 43, 44, 45, 46, and 47 with reading accuracy of 0.0001 in.

Figure 7.3 is a plan view section showing vertical deflection transducers located under the 2nd floor of the test structure. In addition, two transducers (No.'s 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 calibration was also checked by a 0.0001 in. dial gage deflectometer read manually.

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

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

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 keypunched 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 eighteen load tests were carried out on the laboratory structure. Of these, seventeen 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 test model with drywall panels installed. Tests No. 12 through No. 18 were carried out on the model frame, with wall panels removed.

Instruments shown in Figures 7.1 through 7.4 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. A total of 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 1000 curves were plotted. In addition to the data acquired by the automatic digital system, a continuous plot of a 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 the accuracy 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 Figs. 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 appearing at the top of these figures corresponds to the instrument number shown on Figs. 7.1 through 7.4. Because most tests involved a cyclic loading procedure, the computer was programmed to plot the output of each cycle with a different symbol. The order of appearance of symbols is described in Figure A.1.

The abscissa of each curve measures the variable load. Load symbols are defined in Section 3. The ordinate measures deformation, where zero deformation is chosen prior to any load application. Thus in tests where an initial constant load is introduced, the ordinate measures 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 has been excluded due to the erratic behavior of these strain gages. Column gages were located six inches 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 structure with the performance criteria of

Section 5, as well as the manner in which the structure resists 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 effect structural behavior.

Data pertinent to the discussion in this Section are presented in Figs. 9.1 thru 9.23 and in Table 9.I.

9.2 Structural Resistance to 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 Table 6.1.

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 Structural Response

Figure 9.1* 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 hours and the subsequent recovery of deflections 24 hours 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 deflection, this curve illustrates the most critical point in the structure.

The following observations can be made concerning midspan deflection of the center main beam

*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 Figs. A1 through A.77 of Appendix A.

under the application of a load of 1.3D + 1.7L and its subsequent maintenance for 24 hours (Fig. 9.1):

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

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

(3) Observed recovery was 96% (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 (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 loading system (designed for 300 psf) failed before 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 with walls. In Test 9 the maximum applied floor load was 160 psf (1D + 3.5L). It is interesting to note the substantial increase in stiffness against gross vertical deflection resulting from the presence of the walls. 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. 9.23). Since there are stirrups in the beam (Fig. 5.6) 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 Fig. 9.1 and 9.2 illustrate the case of interior span loading (w) acting alone, since this appeared to be the more critical loading configuration. This is illustrated by Fig. 9.3, 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 Figs. 9.2 and 9.4. Figure 9.2 compares deflections at midspan of the center main beam in Test 9 with walls, and Test 16 without walls. These tests had identical loading, so 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. The graphs indicate that the structure without the walls had about twice the deflection of the complete structure.

Figure 9.4 compares deflections with and without walls at the position which is likely to be most sensitive to walls; namely, the center of the east main beam which rests on a firewall. 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) loaded to 1D + 8.4L.

9.2.1.4 Horizontal Deflections Due to Vertical Loads With 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 in the north-south direction and then in the east-west direction. The results of these tests are shown in Figs. 9.5 and 9.6. 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 inches and 0.08 inches and residual displacements were of the order of 0.01 inches.

9.2.2 Compliance with Performance Criteria - Vertical Loads

9.2.2.1 Performance Criterion 4.4.3 Sustained Load Deflections

At a load of 1.3D + 1.7L, sustained for 24 hours the following deflections shall not be exceeded:

(a) $dv \leq \frac{1}{480} \times \left(\frac{1.3D + 1.7L}{D + L}\right)$

(b) $Dh \leq 0.002h$

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

(c) If $dv > \frac{1^2}{20{,}000t}$ $\quad dvr \leq 0.25\ dv$

if $dv \leq \frac{1^2}{20{,}000t}$ $\quad dvr \leq \frac{1^2}{80{,}000t}$

(d) if $dv > \frac{1^2}{20{,}000t}$ $\quad Dvr \leq 0.25\ Dv$

if $dv \leq \frac{1^2}{20{,}000t}$ $\quad Dvr \leq dvr + 0.25\ (Dv - dv)$

where:

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

(a) Under the vertical loading of 1.3D + 1.7L sustained for 24 hours the midspan of the center main beam exhibited the largest net vertical deflection (dv). This deflection was less than 0.10" (see Fig. 9.1). The net deflection (dv) should be taken as the total deflection (Dv) less the support deflection. Unfortunately the instruments measuring the support deflection during the sustained load portion of this test 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 is considerably less than that allowed by Criterion 4.4.3(a).

(b) The lateral 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" under 1.3D + 1.7L on the floor and columns. This is considerably less than is permitted by Criterion 4.4.3.(b).

(c) Figure 9.1 shows the residual deflection to be considerably less than that permitted.

<u>Criterion 4.4.3 was therefore satisfied.</u>

9.2.2.2 <u>Performance Criterion 4.4.4 Vertical Deflection at Service Loads</u>

At a load of 1D + 1L the following deflection shall not be exceeded:

$dv \leq 1/480$

Figure 9.1 shows that the structure experienced far less deflection at 1D + 1L than permitted.

<u>Criterion 4.4.4 was therefore satisfied.</u>

9.2.2.3 <u>Performance Criterion 4.4.5 Ultimate Strength</u>

> The Structure or any member thereof shall not fail at a load smaller than 1.25 (1.5D + 1.8L).

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

<u>Criterion 4.4.5 was therefore satisfied.</u>

9.3 Structural Resistance to Horizontal Loads

Horizontal forces were applied to the structure in the form of the horizontal loads Hw, Hs, and Hs' (See Fig. 6.1 and Table 6.1). Racking tests were conducted in the S-N and the W-E direction with and without walls. The results of these tests are described and evaluated in the following sections.

9.3.1 South Direction

In this 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 S-N direction are illustrated in Fig. 9.7. This figure shows the development of lateral deflection measured at the level of the 2nd floor of the test structure during the application of loads simulating 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"), recovery was also small. Figure 9.8 shows the results of a later racking test which was carried to an equivalent of 60 psf windload. These two tests are simultaneously plotted in Fig. 9.9* and show good agreement.

Figure 9.10 is a graph of South Wind Load versus diagonal compressive deformation measured in one of the fire walls. The diagonal deformation shown in this figure was measured over a gage length of 147 inches. 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 are extremely small. It is interesting to note from this figure that the recovery of the walls was good for all levels of load.

*only a portion of the test to 60 psf is shown here since it exceeds the scale of this figure.

Referring to Fig. 9.9 no signs of distress were observed in the walls or other parts of the structure during Test 2. However at the upper limit of Test 10 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 also 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 then 1/8" 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 wind load from the south was applied to the structure. The results of this test are illustrated in Fig. 9.11. This test is also plotted in Fig. 9.12 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" versus 0.007"). 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").

9.3.1.3 Frame Action vs. Wall Action

The frame was racked after removal of the walls in Test #14 which is illustrated in Fig. 9.11. This test is also plotted along with the identical racking test performed before removal of the walls in Fig. 9.9. This plot clearly indicates that the major portion of the lateral stiffness is provided by the walls rather than by the frame.

Figure 9.13 shows the results obtained in a racking test (Test 18) on the structure with walls removed, carried to the point where the frame was no longer developing increased 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 react 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 9.13 indicates that the frame acting alone in the S-N direction cannot be expected to withstand a wind force in excess of 10 psf on the gross area of the structure. By the time that this test was performed the structure had been carried through a number of earlier tests which might have somewhat weakened the frame. However, it is not considered that this earlier testing had a significant influence on these results. It is, however, recognized that the lower column connection which was used in the test structure was extremely conservative compared to that used in the real structure, particularly for tests with out walls. Thus the results of this test possibly fall well below the results which would be obtained from a real structure.

9.3.2 <u>West Direction</u>

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 firewalls; however, their rigidity in the lateral load tests appeared to be comparable to that of the firewalls. 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 (Section 9.3.1.1). Figure 9.14 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"). Recovery characteristics were similar to those observed for the firewalls. In this test there appear to be two breaks in the load-deflection curve (Fig. 9.14), one at 10 psf and the other at 24 psf. Neither of these were accompanied by any visual signs of distress in either the concrete frame or in the gypsum walls. Figure 9.15 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 are shown in Fig. 9.16. 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 cannot be 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".

Figure 9.17* 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 walls 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 large shear cracks (Fig. 9.22). 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 these were not particularly pronounced and were similar to those discussed in Section 9.3.1.1.

9.3.2.2 <u>Racking Test at High Vertical Loads</u>

A racking test was performed in the west direction in Test 8 with 1.3D + 1.7L and 15 psf wind load. The results of this test are illustrated in Fig. 9.18. It should be noted that the application of the vertical loads

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

caused a 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 Fig. 9.16. 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 magnitude of vertical load on lateral stiffness.

9.3.2.3 Frame Action versus Wall Action

Figure 9.14 (Test 3) shows the response of the structure with walls while Fig. 9.19 (Test 15) shows it for the structure after removal of the walls. Both of these curves are repeated in Fig. 9.20.* As was the case in the S-N direction it is evident that in the W-E 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 S-N direction, which is the narrow direction for this system, was chosen for this

*Only a portion of Test 15 is shown on this figure because of the scale.

test and its results were reported earlier (Test 18) in Section 9.3.1.3. In the W-E direction the maximum load test to which the frame without walls was subjected was Test 15, which is shown in Fig. 9.19. 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. The conservative nature of the lower column connection which was used in the test structure as opposed to that used in the real structure 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 Lateral Deflection under Dead and Wind Loads

> At a load level of 0.9 dead + 1.25 wind (0.9D + 1.25H) the following lateral deflection shall not be exceeded:
>
> $Dh \leq 0.02h$
>
> where:
>
> Dh = horizontal gross deflection
> h = height above grade

In Test 2 (Fig. 9.7) under 0.9D and a wind load of 25 psf (1.25H) from the south the maximum lateral drift was 0.091" while in Test 3 (Fig. 9.14) with a west wind load the maximum lateral drift was 0.012" while the allowable drift under this criterion is 0.19".

Criterion 4.4.1 is therefore satisfied.

9.3.3.2 Performance Criterion 4.4.2 Lateral Deflection under Dead, Live and Wind Load

At a load level of 1.3D + 1.7L + 0.8H the following deflection shall not be exceeded:

$Dh \leq 0.002h$

In Test 7 (Fig. 9.21) under 1.3D + 1.7L and a wind load of 15 psf (0.8H) from the south the maximum lateral drift was 0.045" while in Test 8 (Fig. 9.18) with a west wind load the maximum lateral drift was 0.032"*, while the allowable drift under this criterion is 0.19".

Criterion 4.4.2 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 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 the laboratory and on the erection methods and materials used therein. Variations in materials and erection methods may influence performance.

(2) The structural system performed extremely well in terms of the performance criteria which were set for its evaluation. As a system, it exhibited strength and stiffness far in excess of service requirements.

(3) The walls of the system behaved as integral parts 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 structural frame without the wall has sufficient strength to carry the vertical loads; however without walls is not capable of carrying design wind loads.

10. Component Tests

10.1 Introduction

Load tests were conducted on the three principal load-bearing precast 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 upper story columns. The tests consisted of the following:

(1) Short-term application of load to failure. Loads were applied parallel to the column axis. Three columns were tested with load eccentric to the major axis and three with load eccentric to the minor axis.

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

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

10.2.2 Short-Term Tests

10.2.2.1 Specimens

Six columns were tested with three being tested with an eccentricity of 0.5 in. with respect to the minor axis (Columns 1, 5 and 8) and three with an eccentricity of 2.0 in. with respect to the major axis (Columns #2, 6 and 7).

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. These specimens were about 20 days old when tested. Columns 5, 6, 7 and 8 were cast from a similar concrete at a later date (May 15) and were about 30 days old when tested. This concrete had a compressive strength of 5400 psi at the time of test.

The reinforcing bars (No. 6 deformed bars) were approximately 3-in short of the full-length in Columns 1 and 2, but were only 1/4-in. short in Columns 5, 6, 7, and 8.

10.2.2.2 Loading

The loads were applied continuously until failure through a knife-edge loading plate (Figs. 10.1 and 10.2) by a 600 kip hydraulic testing machine at a rate of 8 kips per minute.

Deflections were measured with long-throw mechanical dial gages at mid-height.

10.2.2.3 Results

The test results are shown in Figs. 10.3 through 10.8 as load-deflection curves, and ultimate loads are tabulated in Table 10.1. The average maximum load for Columns 1, 5 and 8 (minor axis bending, e=0.5 in.) was 78.9 kips. 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% 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 kips. All three specimens failed in a similar manner with excessive bending of the channel-shaped, top-fixture and some spalling of the concrete near this fixture.

10.2.2.4 Interpretation of Results

The results indicate that with the small eccentricities (e=0.5 in.) with respect to the minor axis the columns are able to withstand short-term loads well in excess of the ultimate required by the performance criteria. A slight improvement (12%) was noted when the reinforcing bars were made longer.

The results for the columns with the larger eccentricity (e=2 in.) with respect to the major axis, indicate a weakness in the top connecting fixture. However, in the structure the bending of this fixture is restrained by the channel slabs and the topping concrete. Therefore as a part of a structural system, especially for the case of interior columns, these components probably could develop higher loads than indicated by these tests.

10.2.3 Sustained Loading (Creep) Test

10.2.3.1 Specimens

Two columns (Columns 3 and 4) were tested under sustained load, one column with an eccentricity of 0.5 in. with respect to the minor axis and one with an eccentricity of 2.0 in. with respect to the major axis. Both columns were cast with the test-structure components and were about 20 days old when placed under test.

10.2.3.2 Loading

The loading frames used in these tests are shown in Figs. 10.9 and 10.10. Column 3 was loaded with an eccentricity of 0.5 in. with respect to the minor axis while Column 4 had an eccentricity of 2.0 in. about the major axis. A detail of the bottom of the Column 4 loading frame is shown as Fig. 10.11. This figure also shows the heavy spring used to sustain the load on the specimen.

The 25 kip load (D+L) was applied by means of a 30-ton hydraulic ram and a load cell inserted between the top two plates of the test frame. When hydraulic pressure was applied to the ram 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 3/4" tie-bars were tightened against the top knife-edge plate. The deflections of the springs were about 1 1/2 in. at the 25 kips 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.

10.2.3.3 Results

Thirty-day results are presented in Figs. 10.12 and 10.13 as time-deflection curves. The initial deflections are included in the total deflection for information and comparison purposes.

10.2.3.4 Interpretation of Results

The results indicate that the increase in deflection with time (creep deflection) was not great. This can be attributed to the high percentage of steel in the column (p=6.2%). From the results obtained to date (June 18, 1968), it would not appear that creep buckling at service loads is a major concern in these columns. These tests are being continued and additional data will be reported in a supplementary report.

10.3 Channel Roof 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 the center. Deflection of the slab was measured at midspan with two 2 in. throw mechanical dial gages.

The test results are shown in Figure 10.13. The ultimate moment of the channel was in reasonable agreement with the predicted capacity on the basis of ultimate strength theory using the nominal specified reinforcement yield strength. The tests on the laboratory structure also indicated satisfactory performance of these components.

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 (Beams 5-11). Preliminary tests on three other slabs are not reported because the test conditions (quarter point loading) were found to be far too severe in relationship to service conditions.

Beams 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 they did not have tie beams framing in at these connections (a block of wood was used as a spacer to fill the void caused by omission of the tie beams); and the connections were not grouted. Beam 5 did not have the column stub or column fixtures.

In all of the beams tested the top surface of the precast components at the time the topping slabs were placed had not been roughened as required by the plans and specifications of the Neal Mitchell Housing System. All beams including the placing of the topping slab were prepared by Neal Mitchell associates and delivered to the laboratory for testing. These top surfaces had been cast against steel forms and were very smooth.

Three types of shear connectors were used in the seven beams. Beams 5, 7 and 9 used Star inserts spaced 19 in. on centers (which is similar to that used in the test structure). Beams 8 and 10 used Richmond (Kohler) inserts spaced 19 in. on centers. Beams 6 and 11 used the Richmond inserts spaced 9 1/2 in. on centers.

The beams and their topping slabs were cast at various times from several concretes. The age and strength data for these concretes are presented in Table 10.2.

10.4.2 Beam Loading

All beams were tested by applying the loads as shown in Fig. 10.15, while simply supported by rollers on a span of 12.5 ft. Figure 10.14 is a general view of the test set-up. 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 load to the eighth points of the test beam.

The beams were subjected to 1000 cycles of stress alternating between intensities corresponding to 1D and 1D + 1L, (for each ram 1D=2.5kips, and 1L=2.5kips; 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.

The rate of cyclic loading was 1 cycle per second except for a few cycles at the beginning and end of each increment. During this initial period when the rate was 0.01 cycle per second, mechanical dial gage readings were made. 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 voltage differential transducer (LVDT). Both measuring methods can be seen in Fig. 10.14.

In an effort to check 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 Fig. 10.14.

10.4.3 Test Results

Graphs drawn from strip-chart recordings of the midspan deflections are presented as Figs. 10.15 through 10.18. Table 10.3 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 dial slip gages installed at the ends of the beam.

10.4.4 Interpretation of Results

Beam 5 which had none of the beam column connection fixtures and had Star inserts at 10 inches on center performed rather poorly in terms of slip between the precast component and the topping slab. This beam experienced slip during the first cycle of application of the 1D + 1L load (Fig. 10.15). In terms of ultimate strength it performed considerably better, resisting 1000 cycles of 1D + 2L without failure.

The results of the test on Beam 7 are shown in Fig. 10.16. This beam, which was similar to Beam 5 except that it did have a partial beam-column connection, performed considerably better in terms of slip. Beam 7 was able to sustain 1000 cycles of 1D to 1D + 1L without exhibiting any signs of slip. First slip for this beam was observed after the first few cycles of the 1D + 1.5L loading. Its ultimate failure took place at approximately the same point in the loading sequence as that of Beam 5 (third cycle of D + 3L). The companion to Beam 7; Beam 9 exhibited signs of first slip during the first cycle of 1D to 1D + 1L and failed at the 820th cycle of 1D to 1D + 2.5L.

The results of the test on Beam 8 are shown in Fig. 10.17. Note that this beam, which had partial beam-column 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. The companion to Beam 8; Beam 10 slipped at about the same point in the loading sequence as did Beam 8, however its ultimate failure took place during the first cycle of 1D + 3L.

Beam 6 was similar to Beam 8 except that it had inserts spaced at 9 1/2 inches on center while Beam 8 had them at

19 inches. 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 + 2L and ultimately failed at 1230 cycles of 1D + 3L.

In the design of these beam repeated load tests, it was felt that from the standpoint of slip behavior, the beam should be capable of sustaining 1000 cycles of loading from 1D to 1D + 1L, and that it should be capable of sustaining a loading of at least 1D to 1D + 2L before ultimate failure. All of the beams tested which had partial connections, except for Beam 9, satisfied this criterion.

The beams tested as isolated components experienced considerably larger deflections at 1D + 1L than did the center main beam of the test structure (Fig. 9.2), indicating that the component test was conservative in comparison with the system test. Also the beam test specimens were not provided with complete beam-column connections; thus the results obtained in these beam tests are probably conservative relative to the behavior of a real structure.

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 results from the tests on the structure and on the 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. Specimens of these concretes 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 lighweight aggregate concrete.

This concrete was made from a 3/8 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 cu yd.), 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 pcf at the mixer. During the first day of casting (April 16) the unit weight was controlled very accurately at 96 1/2 pcf; however, on the second day (April 17) apparently some difficulty in control was encountered and a unit weight of 100 pcf resulted. The slump was judged to be about 2 in. although it was not measured.

Test specimens (6 x 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 components and were removed from the molds when about 8 days old. They were then stored in the laboratory air until tested.

Because the components were cast under commercial conditions, no attempt was made to maintain records 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 x 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, semi-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 course aggregate was a 3/4 in. maximum size expanded shale and the fine aggregate was a natural sand. The compressive test specimens were molded in 6 x 12 in. cast iron molds which were removed when 2 days old. 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 determinations are presented in Table 11.2. The values of the modulus of elasticity are shown in Table 11.3. The report on the air-content determination on batch 4/16-B is reproduced as Figure 11.1.

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

These indications point up three conclusions:

(1) The unit weight of the concrete at the mixer is not necessarily equal to the unit weight of the concrete in the form.

(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.

(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 strength. The results are presented in Table 11.4.

12. Summary and Conclusions

12.1 Summary

A full-scale one-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 actual service and ultimate 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 building systems were developed.

12.2 Conclusions

The major conclusions which can be drawn from this investigation are as follows:

(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) Within the framework of the present state of the art, performance criteria can be developed for the confident evaluation of the safety and structural adequacy of existing and innovative building systems.

(3) The structural system performed well in terms of the performance criteria which were set for its evaluation. As a system, it exhibited strength and stiffness far in excess of service requirements.

(4) The walls of the system behaved as integral parts 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.

(5) The structural frame without the wall had sufficient strength to carry the design vertical loads; however, without walls is not capable of carrying design wind loads.

(6) Based upon the minor amount of creep deflection which occurred during the 30 days under sustained load in the long term column tests, it does not appear that creep buckling of the column will be a critical factor at service loads.

(7) The precast main beams tested with a portion of the topping slab were not provided with a complete beam-column connection. This probably had an adverse effect on their behavior. However, all the beams (except one) with partial beam-column connections were able to sustain 1000 cycles of loading from dead to dead plus live load without evidence of slip, and all carried considerably greater cyclic loads before ultimate failure.

SYNOPSIS

A full-scale, one-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 actual service and potential ultimate 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 of one such system, the Neal Mitchell Housing System, and discusses their significance. The report also presents data concerning the complex interaction between components which takes place in these building systems.

The primary conclusions reached were:

(1) Within the framework of the present state of the art, performance criteria can be developed for the confident evaluation of the safety and structural adequacy of existing and innovative building systems.

(2) The Neal Mitchell Building System, which was erected in the laboratory, performed well in terms of the performance criteria which were set for its evaluation. As a system, it exhibited strength and stiffness far in excess of service requirements.

(3) The walls of the system behaved as integral parts 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.

TABLES

TABLE 6.1

SIMULATED LOADS (Symbols and Magnitude)
(For Computations, Refer to Appendix B)

Symbols for Loading:

D = Service Dead Load

L = Service Live Load

H = Service Wind Load

SIMULATED LOAD	SYMBOL	MAGNITUDE		
		1XD	1XL	1XH
2nd Story Column	P (kips)	10	7	----
W E Wind	Hw (kips)	--	--	2.05
S N Wind (point opposite firewall)	Hs (kips)	--	--	4.05
S N Wind (point between firewalls)	Hs'(kips)	--	--	0.9
Major Floor Load* (Equivalent distributed load between columns (center strip))	w (psf)	46** 9.3***	43	----
Minor Floor Load* (Equivalent distributed load between north end of structure and northern row of columns (cantilever strip))	w' (psf)	46** 9.3***	43	----

* w and w' were increased to allow for incomplete area coverage by the air bags (See Fig. 6.1).

** 46 psf is the dead load weight of the floor of the test structure.

*** 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 B)

LOADING	P(kips)	Hw(kips)	Hs(kips)	Hs'(kips)	H(psf)	w(psf)	w'(psf)
1D+1L	10					52	52
0.9D+1.25H	9	2.6	5.1	1.2	25		
1.3D+1.7L	25					100	100
1.3D+1.7L+0.8H	25	1.5	3	0.7	15	100	100
1.25(1.5D+1.8L)	35					144	144

TABLE 10.1

COLUMN ULTIMATE LOADS

Column #	Major Axis Eccentricity In.	Minor Axis Eccentricity In.	Ultimate Loac Kips
1	0.0	0.5	73.0
5	0.0	0.5	82.0
8	0.0	0.5	81.8
			Avg.= 78.9
2	2.0	0.0	56.6
6	2.0	0.0	51.0
7	2.0	0.0	45.9
			Avg.= 51.2

Table 10.2 Strength and Age Data - Beam Tests

Beam No	Inserts	Date Cast		Date Beam Tested	Concrete Strength, psi*	
		Beam	Topping		Beam	Topping
5	Star at 19"	4/17	5/10	5/20	7000	3400
7	Star at 19"	5/15	6/7**	6/13	6740	4100
9	Star at 19"	5/15	6/7**	6/15	6740	4100
8	Richmond at 19"	6/5	6/7**	6/14	6400	4100
10	Richmond at 19"	6/5	6/7**	6/17	6400	4100
6	Richmond at 9-1/2"	6/5	6/7**	6/12	6400	4100
11	Richmond at 9-1/2"	6/5	6/7**	6/18	6400	4100

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

** Column stubs cast 6/5 from concrete used in beams cast on that date.

Table 10.3 Repeated Load Test Results

Beam No	Inserts Used	Beam Mid-Span Deflections, in.[1/]										Results	
		Increment 1		Increment 2		Increment 3		Increment 4		Increment 5		Slippage Observed	Collapse Occurred
		D	D&L	D	D&1.5L	D	D&2.L	D	D&2.5L	D	D&3.L		
5	Star at 19"	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 at 19"	0.20	0.68	0.56	0.97	0.78	1.48	0.99	1.89	--	--	Few cycles of 1.5L	3 cycles of 3.L
9	Star at 19"	0.17	0.76	0.61	1.07	0.81	1.43	0.96	2.06	--	--	1 cycle of 1.L	820 cycles of 2.5L
8	Richmond at 19"	0.18	0.48	0.30	0.65	0.80	1.36	1.00	2.23	--	--	Few cycles of 1.5L	4 cycles of 2.5L
10	Richmond at 19"	0.15	0.50	0.38	0.74	0.46	1.07	0.72	1.45	0.86	--	100 cycles of 1.5L	1 cycle of 3.L
6	Richmond at 9-1/2"	0.17	0.54	0.42	0.82	0.70	1.28	0.82	1.63	1.28	3.05	500 cycles of 1.5L	17 cycles of 3.L
111	Richmond at 9-1/2"	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

1/ Deflections reported were measured at beginning of each increment and are based on initial zero-load condition.

TABLE 11.1 CONCRETE COMPRESSIVE AND SPLITTING STRENGTH*

Concrete	Batch	Date Tested	Age at Test	Compressive Strength	Splitting Strength
			days	psi	psi
Precast Component	4/16 - B	May 10	24	5100	346**
Precast Component	4/16 - C	May 10	24	4930	287**
Precast Component	4/17 - A	May 10	23	7160	369**
Precast Component	4/17 - B	May 10	23	7090	
Floor Slab		May 10	17	5600	
Topping Slab	A	May 21	26	3840	
Topping Slab	B	May 21	26	2560	

* 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.

** Tested May 22 at an age of 36 days after 25 days air drying.

TABLE 11.2 UNIT WEIGHTS OF CONCRETE

Concrete	Batch	Date Tested	Age	Air Drying Period	Unit Weight
			days	days	pcf
Precast Component	4/16 - B	May 16	30	21	98.5
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
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

TABLE 11.3 MODULUS OF ELASTICITY OF PRECAST COMPONENT CONCRETE

Batch	Age at Test	Date Tested	Compressive Strength (f'c)	Secant Modulus[1]
	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

[1] Secant Modulus at 0.4 f'c after prior loading to 0.5 f'c several times.

TABLE 11.4 REINFORCING STEEL TEST RESULTS

Specimen	Where Used	Yield Strength psi	Ultimate Strength psi
No. 5 Deformed Bar	Beams	65,000	96,000
No. 4 Deformed Bar	Cantilever Beams	73,000	117,000
No. 2 Plain Bar	Tie Bars in Columns	47,000	77,000

FIGURES

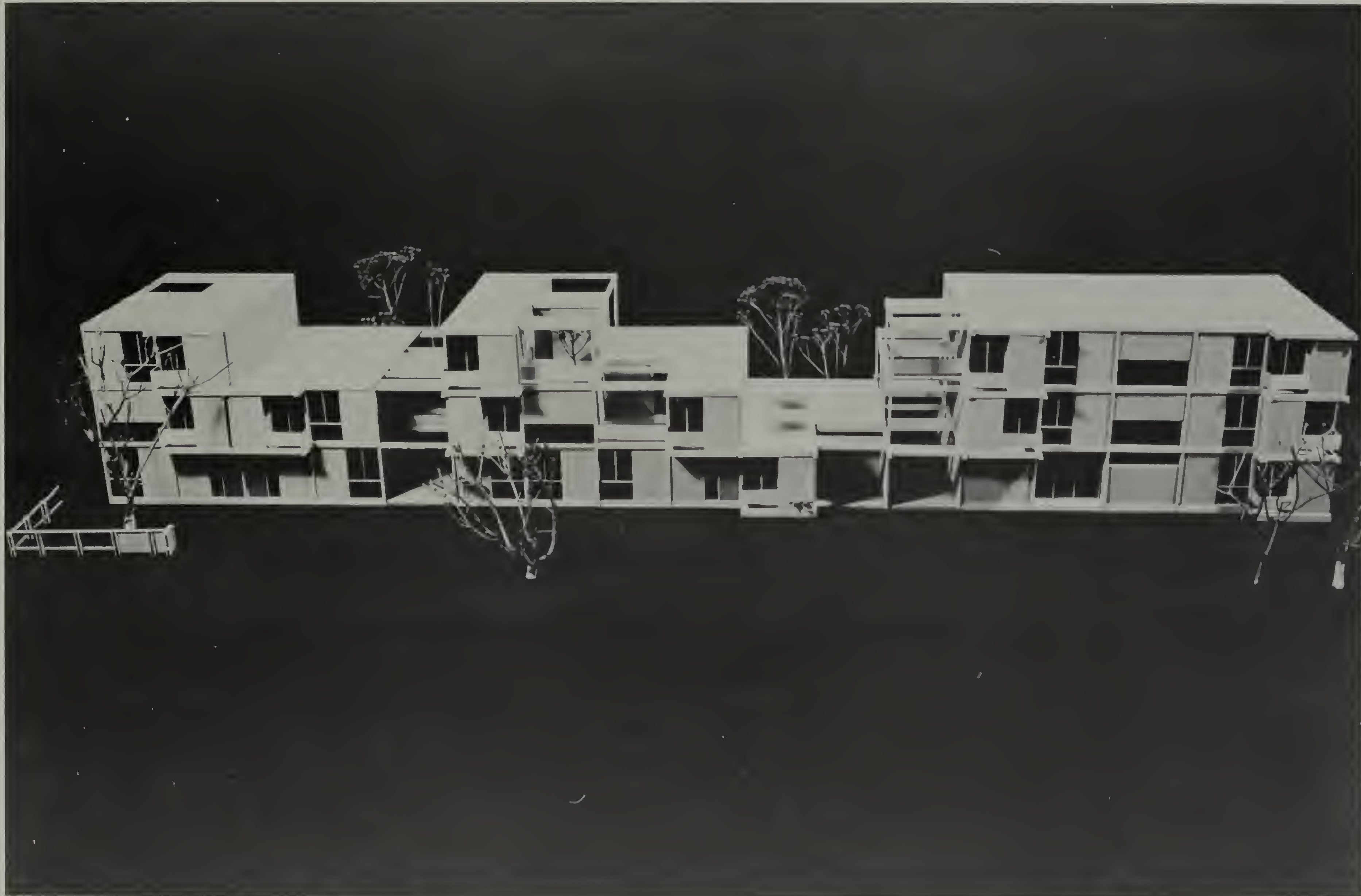


Fig. 5.1 Typical complete structure

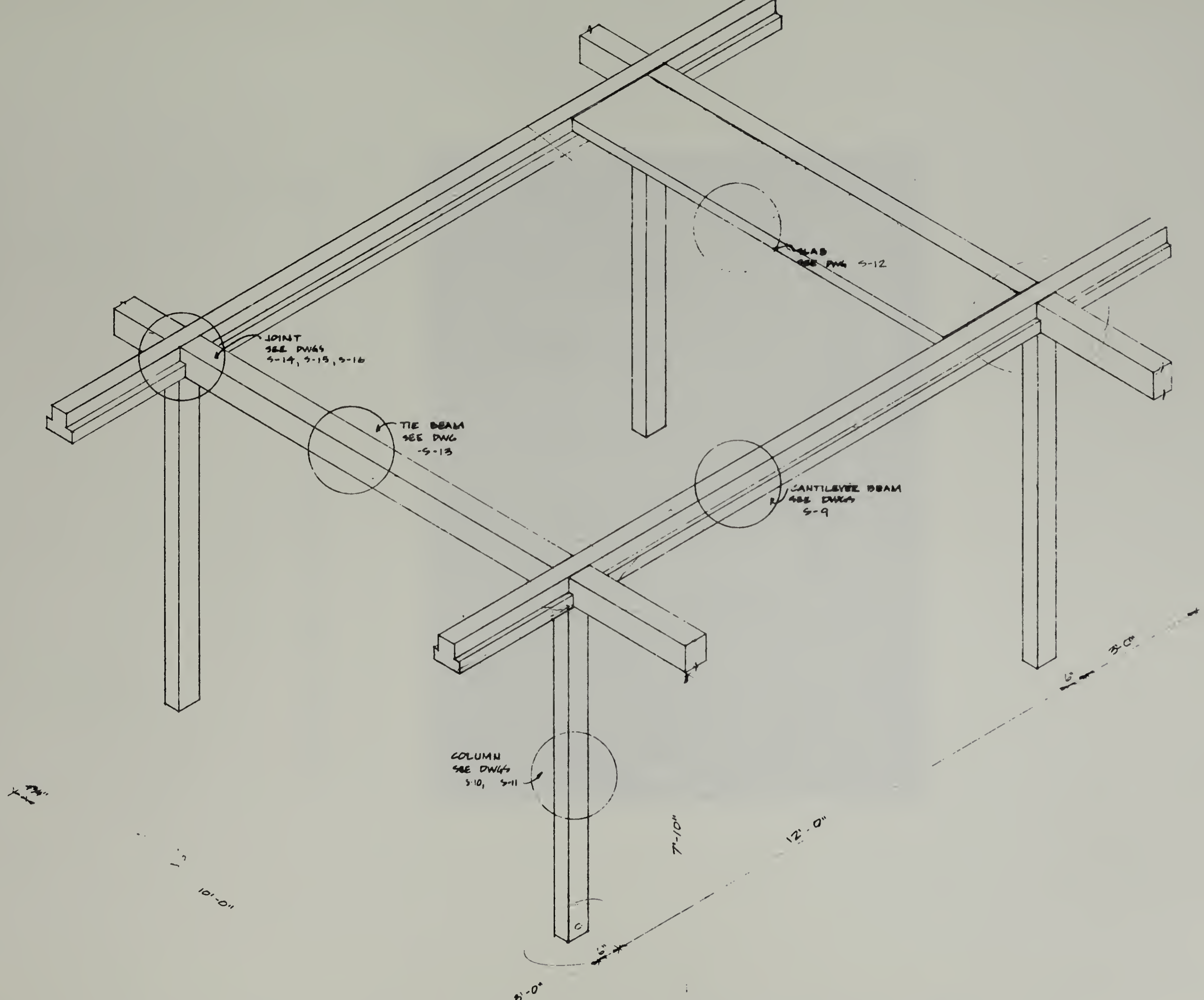


Fig. 5.2 Assembled structural frame

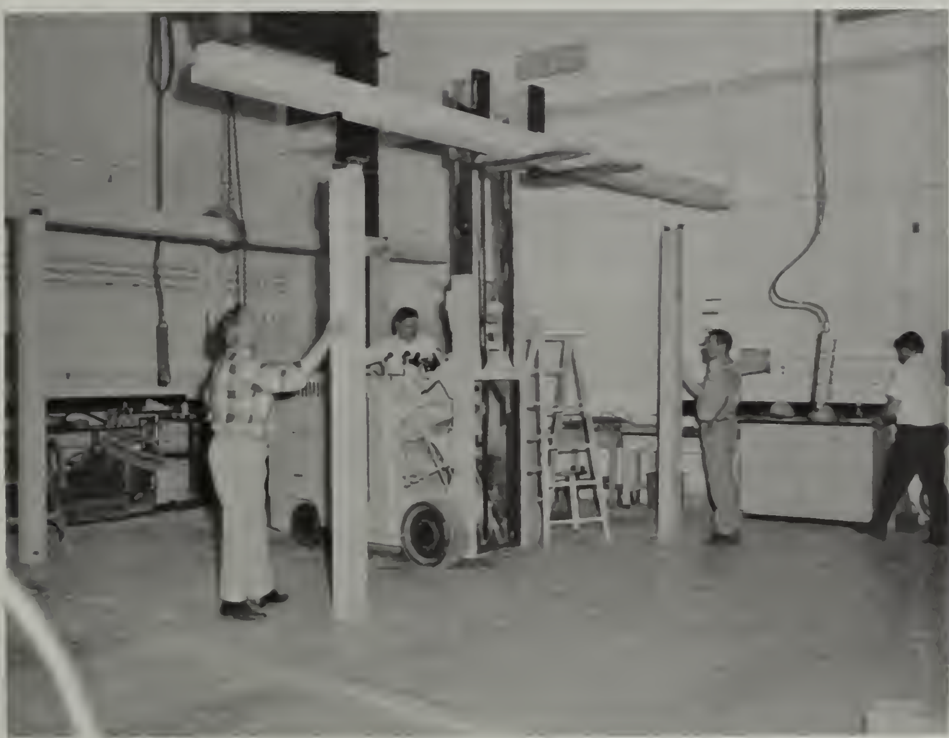


Fig. 5.3 Assembling of structural frame components

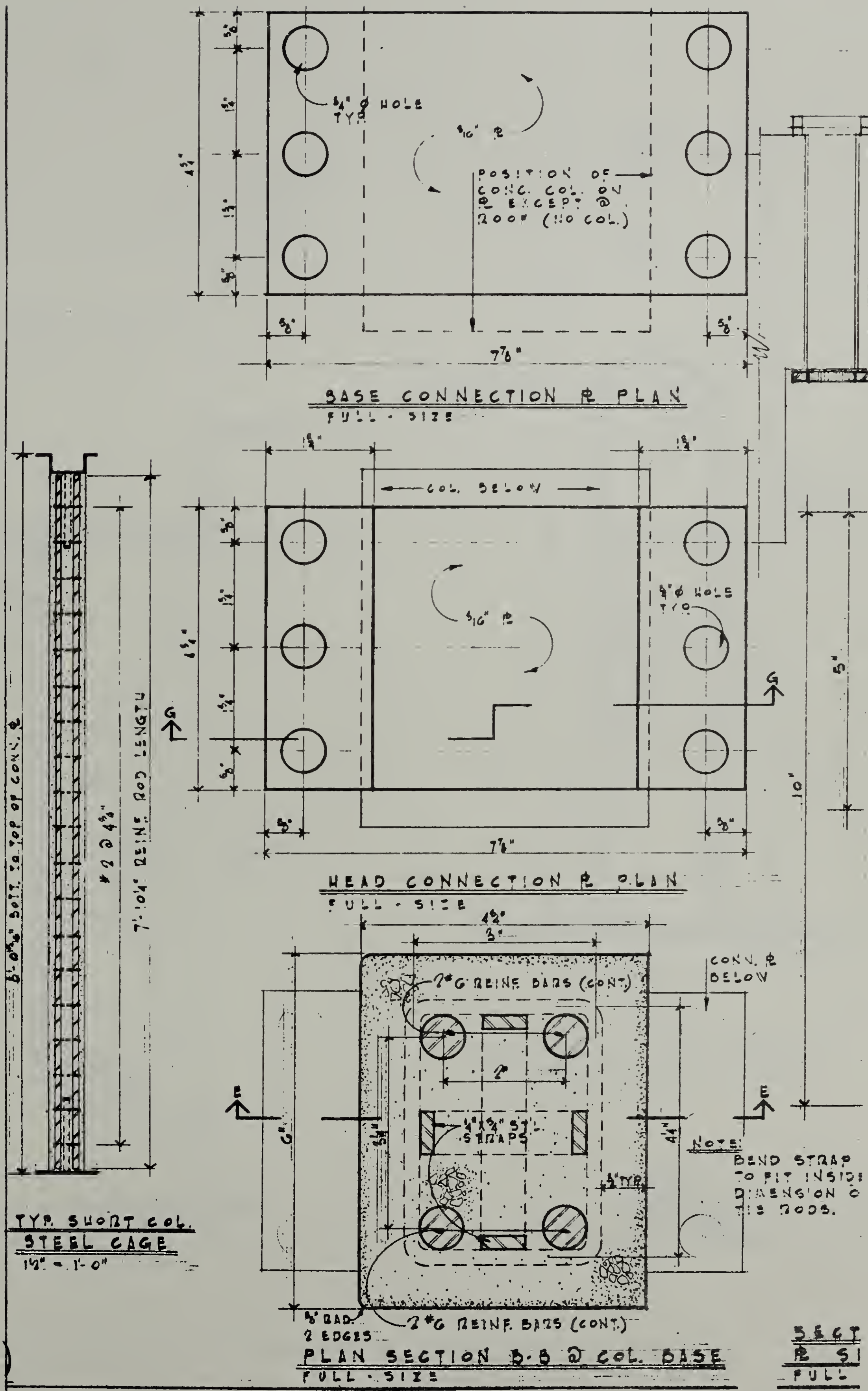


Fig. 5.4 Precast columns

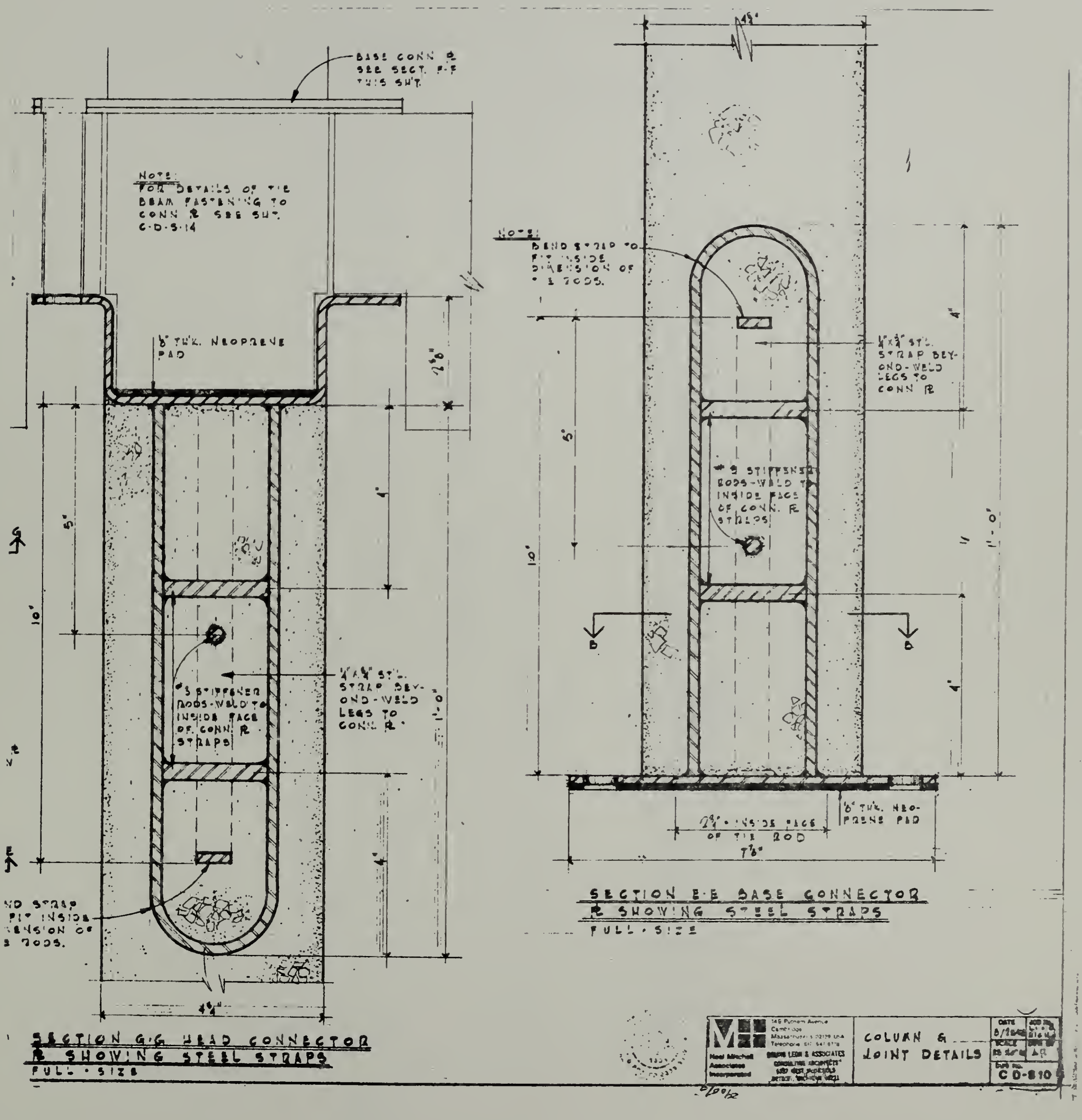


Fig. 5.5 Column head and base plate connectors

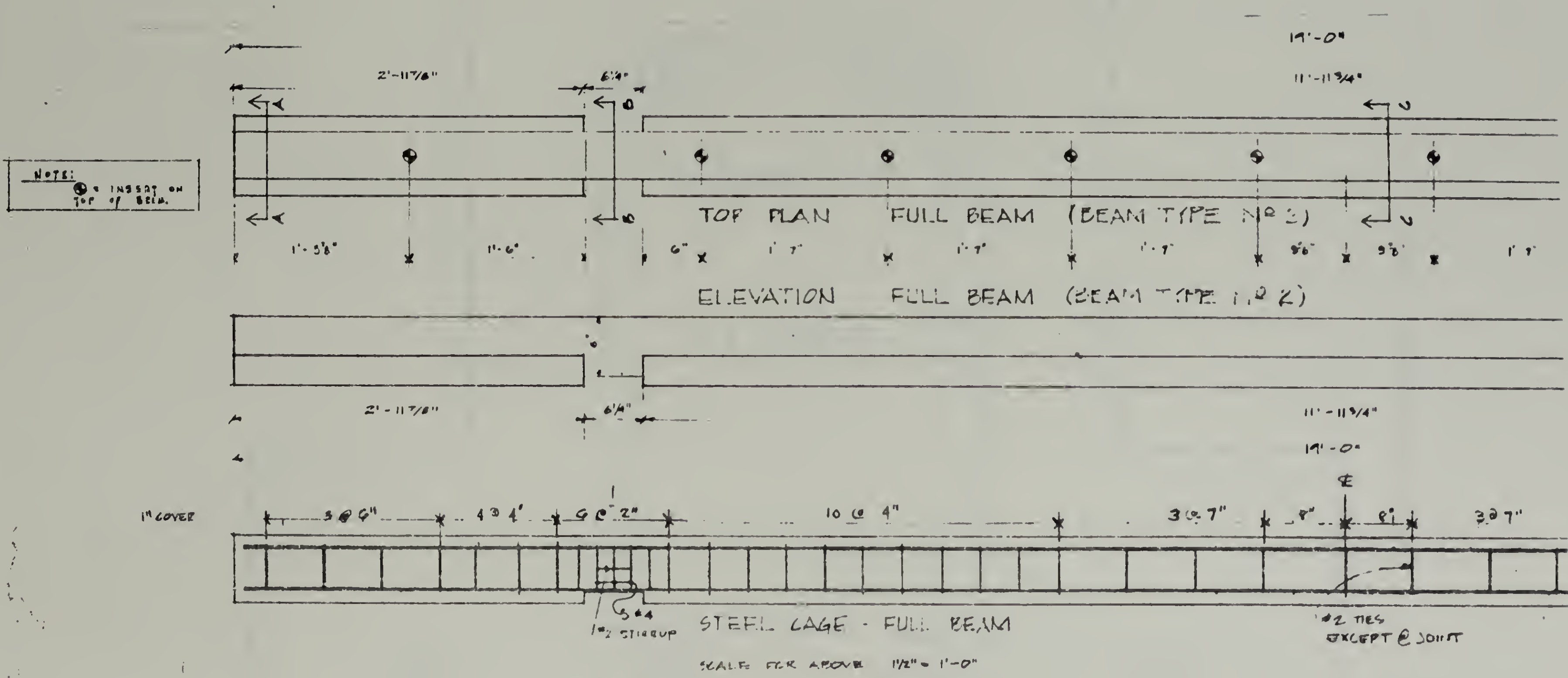


Fig. 5.6 Main beams reinforcement details

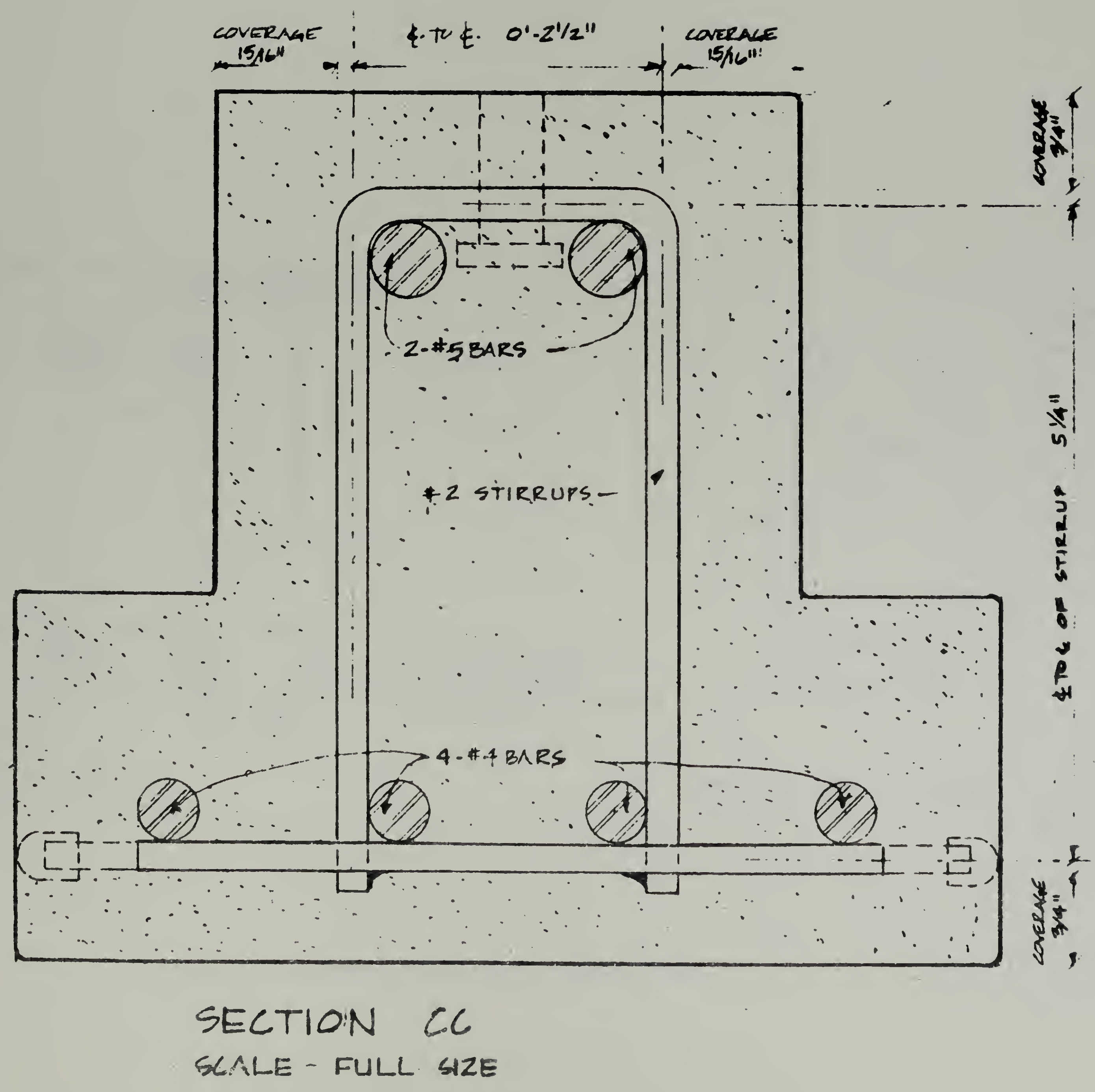


Fig. 5.7 Typical section of main beam

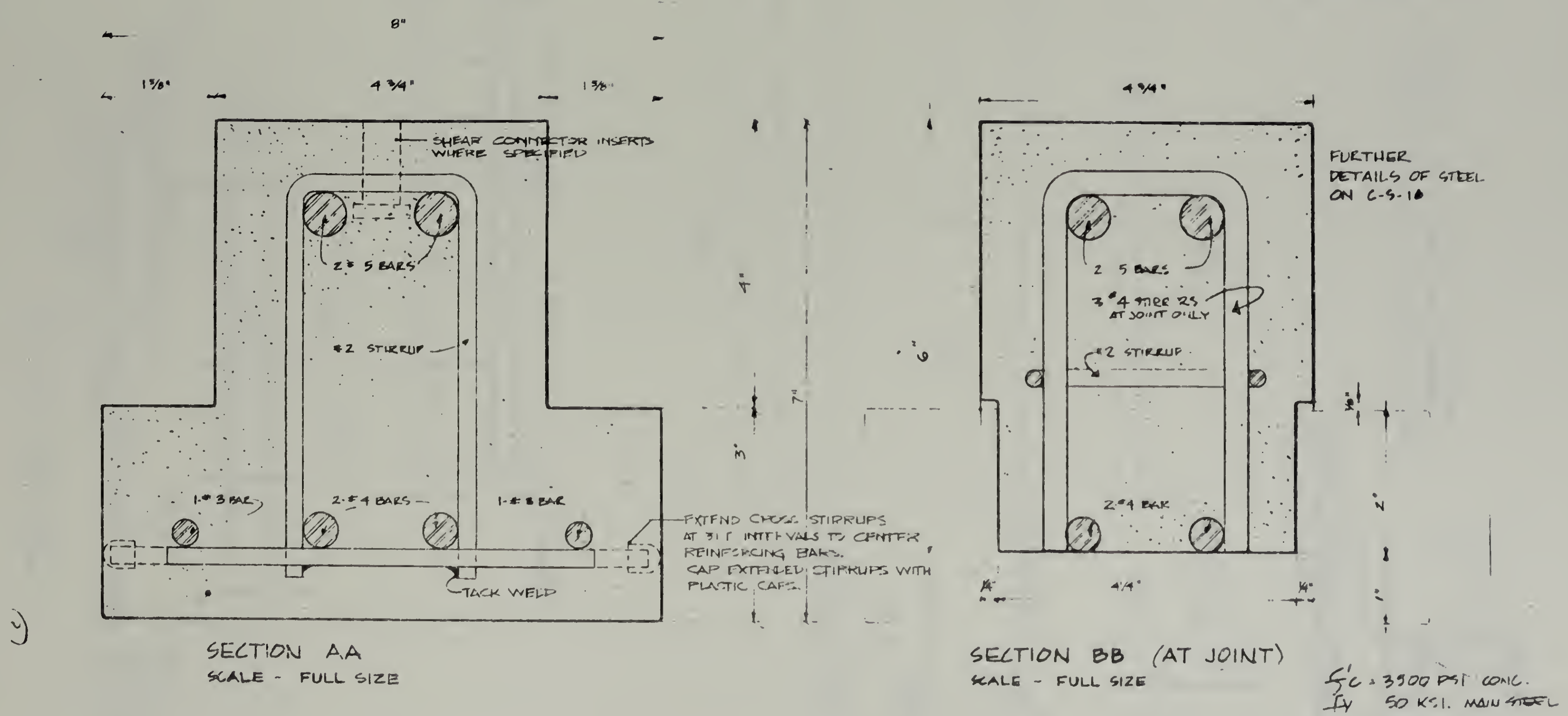


Fig. 5.8 Typical sections of main beams

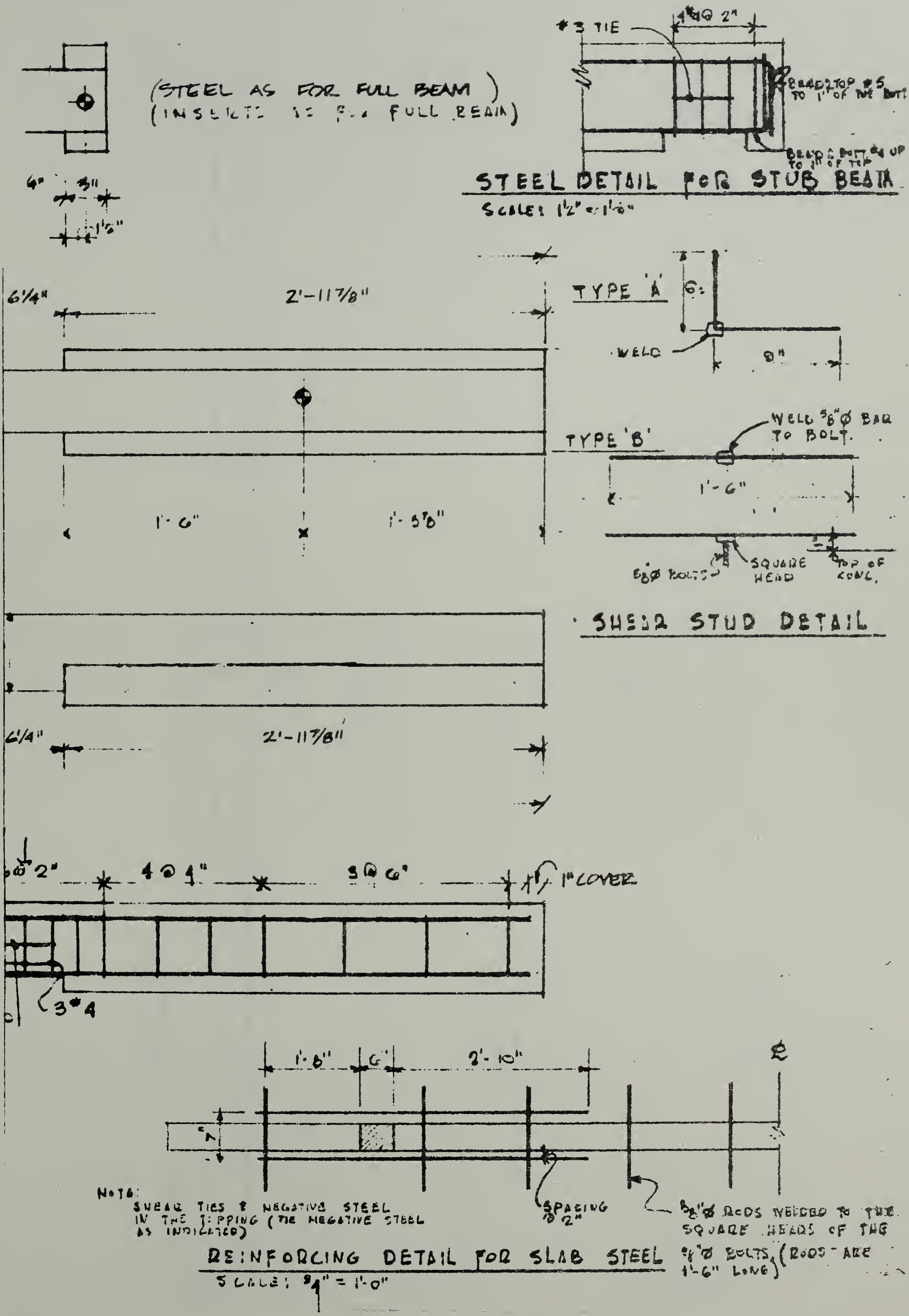


Fig. 5.9 Main beam

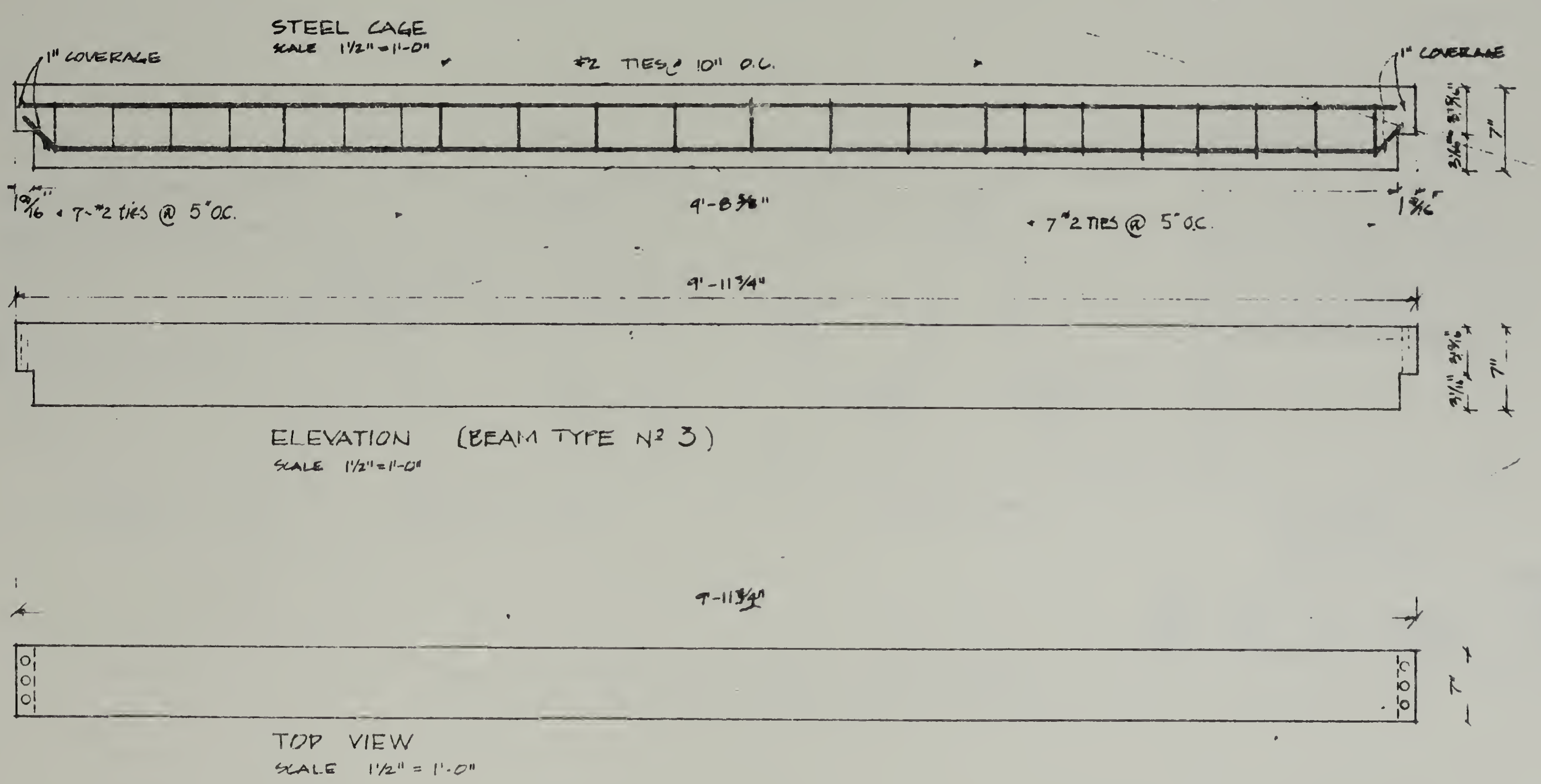


Fig. 5.10 Tie beam

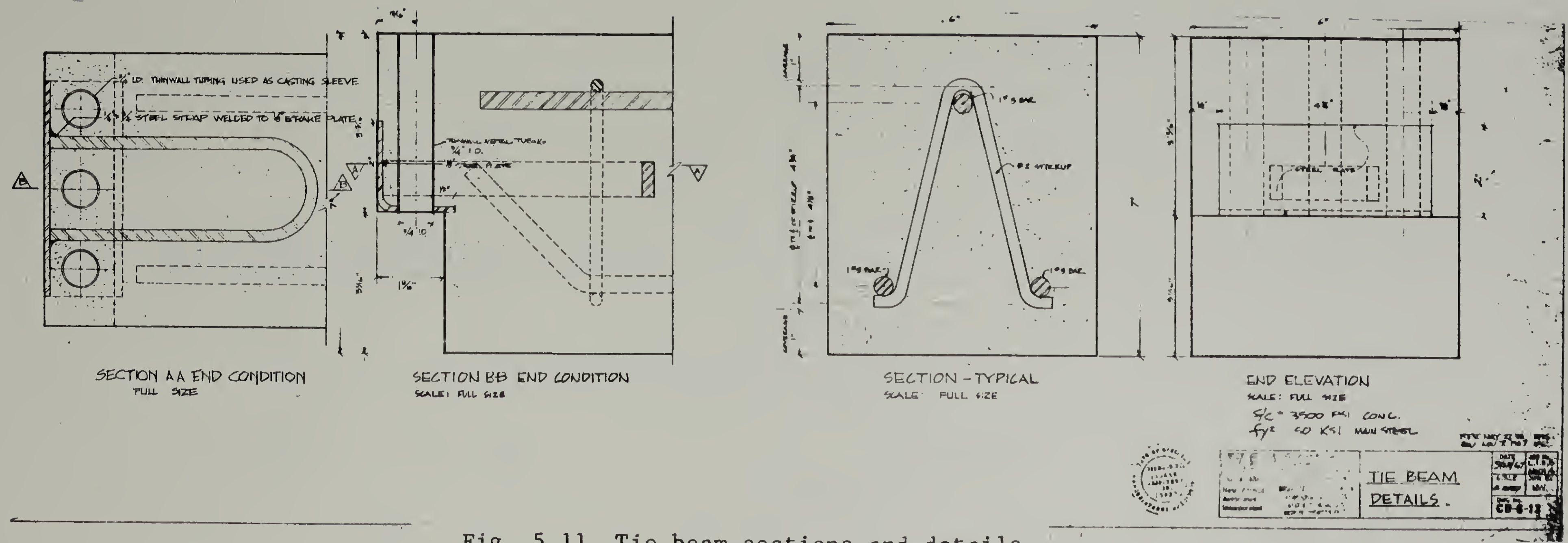


Fig. 5.11 Tie beam sections and details

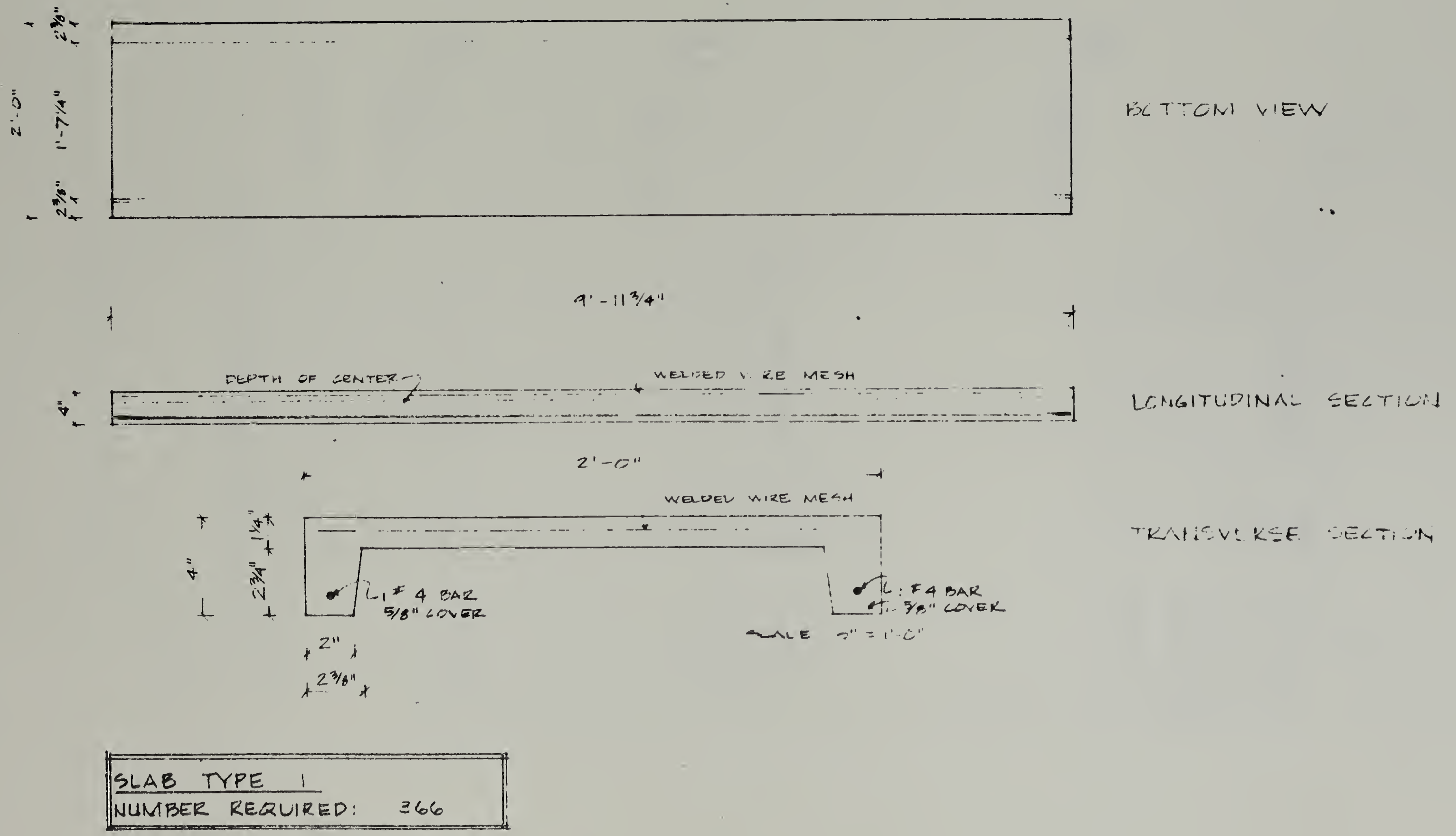


Fig. 5.12 Floor channel details

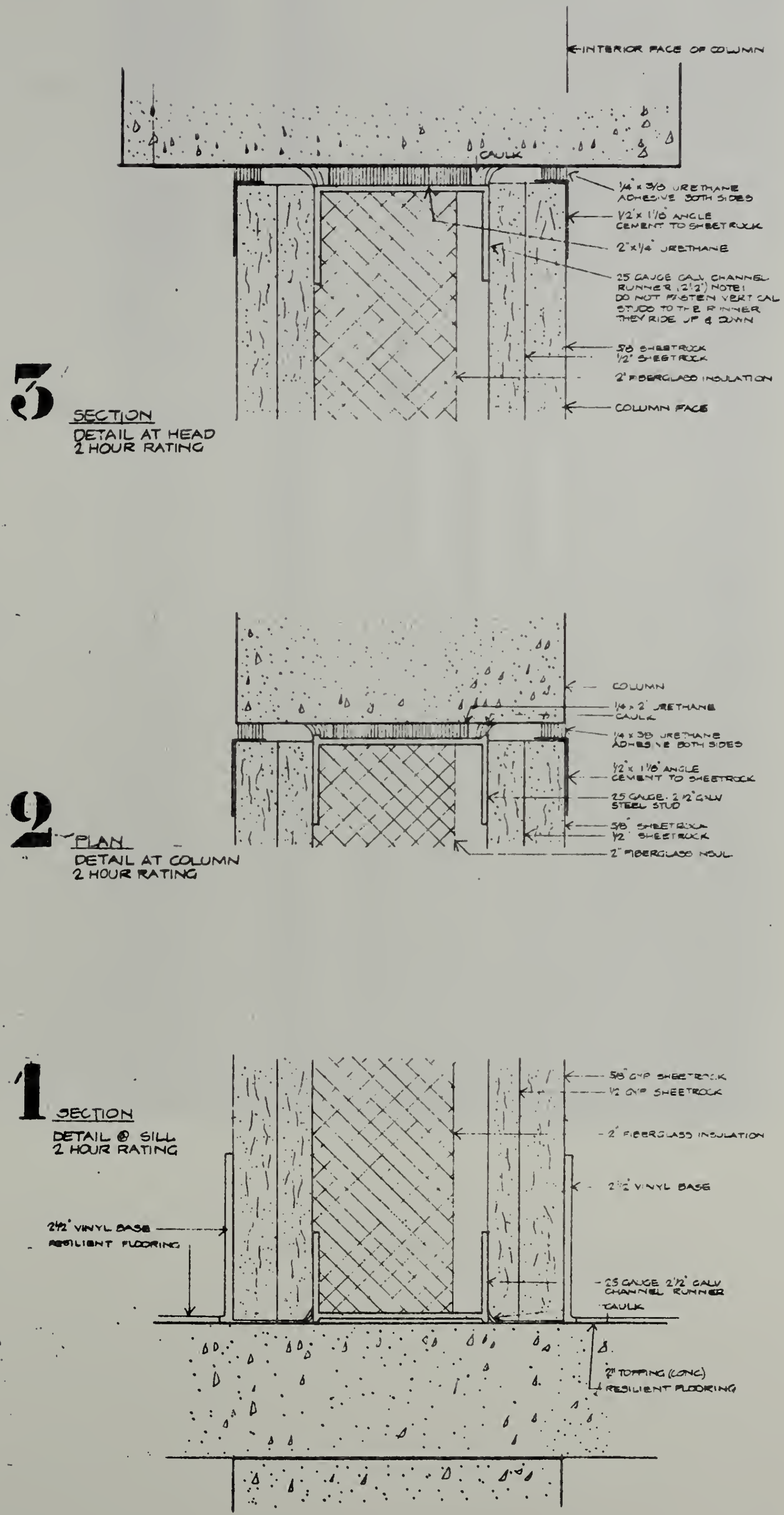


Fig. 5.13 Fire walls

Fig. 5.14 Firewall (partially dismantled)

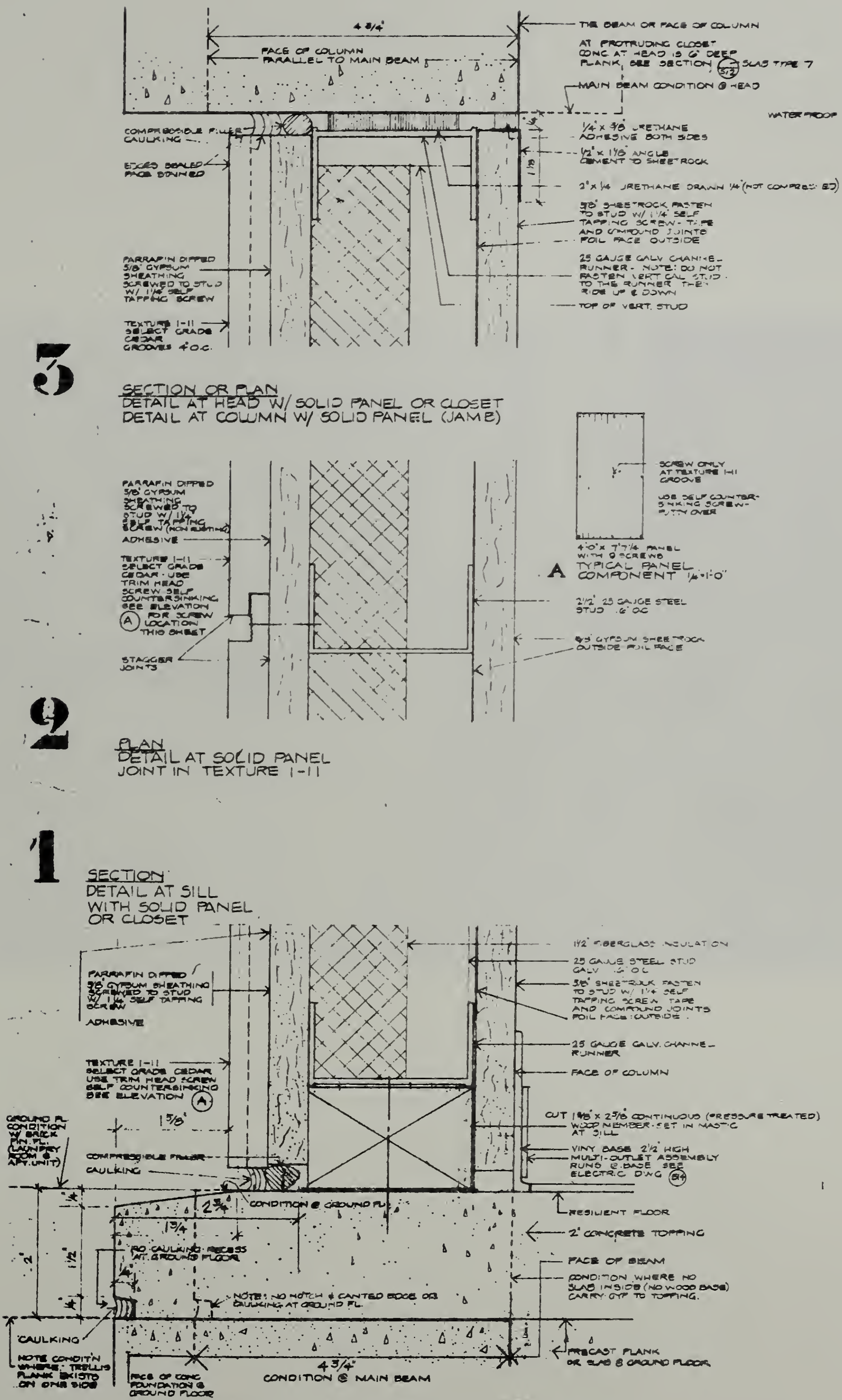


Fig. 5.15 Exterior wall details

Fig. 5.16 Interior "structicore" wall (partially dismantled)

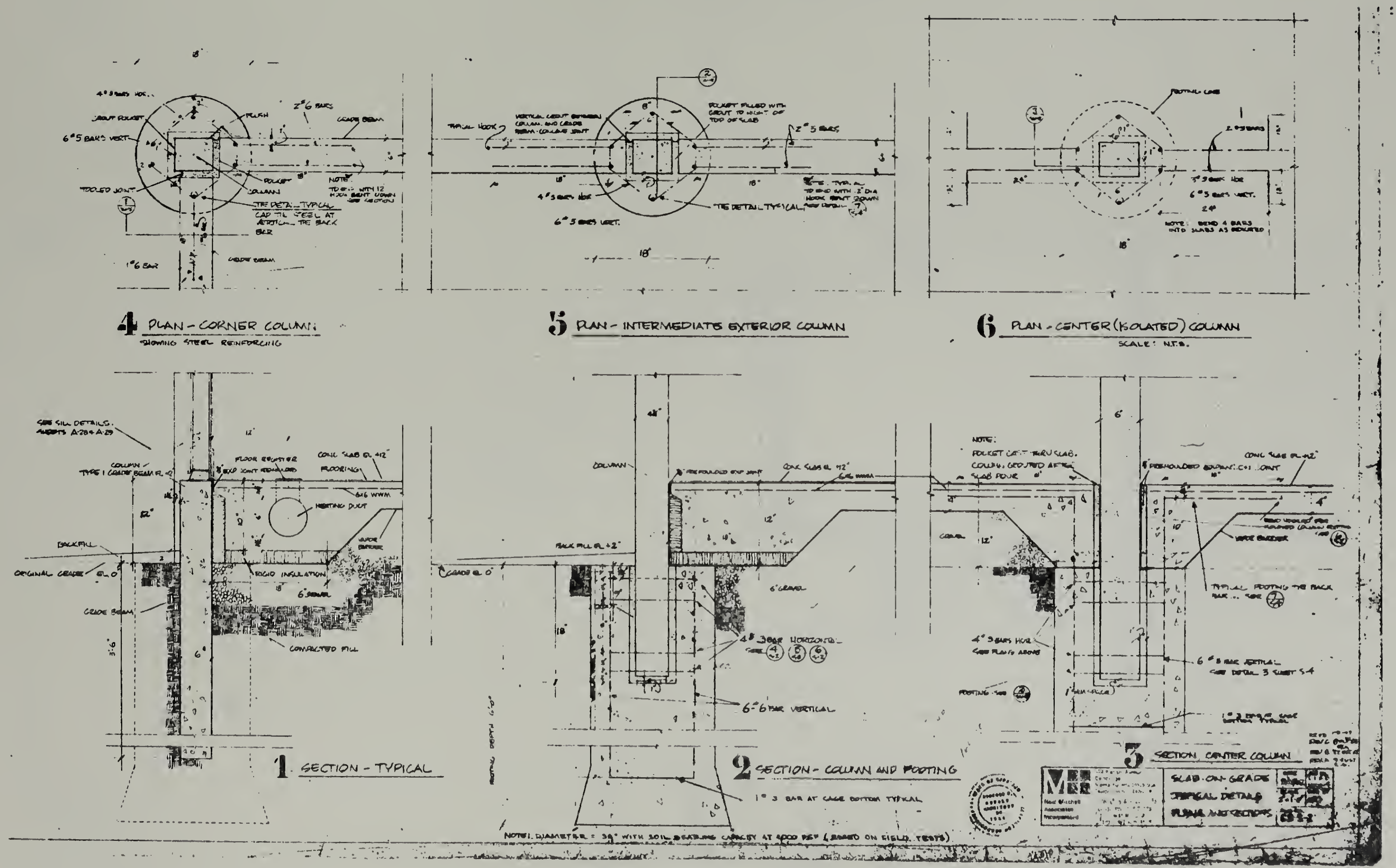


Fig. 5.17 Foundation plans

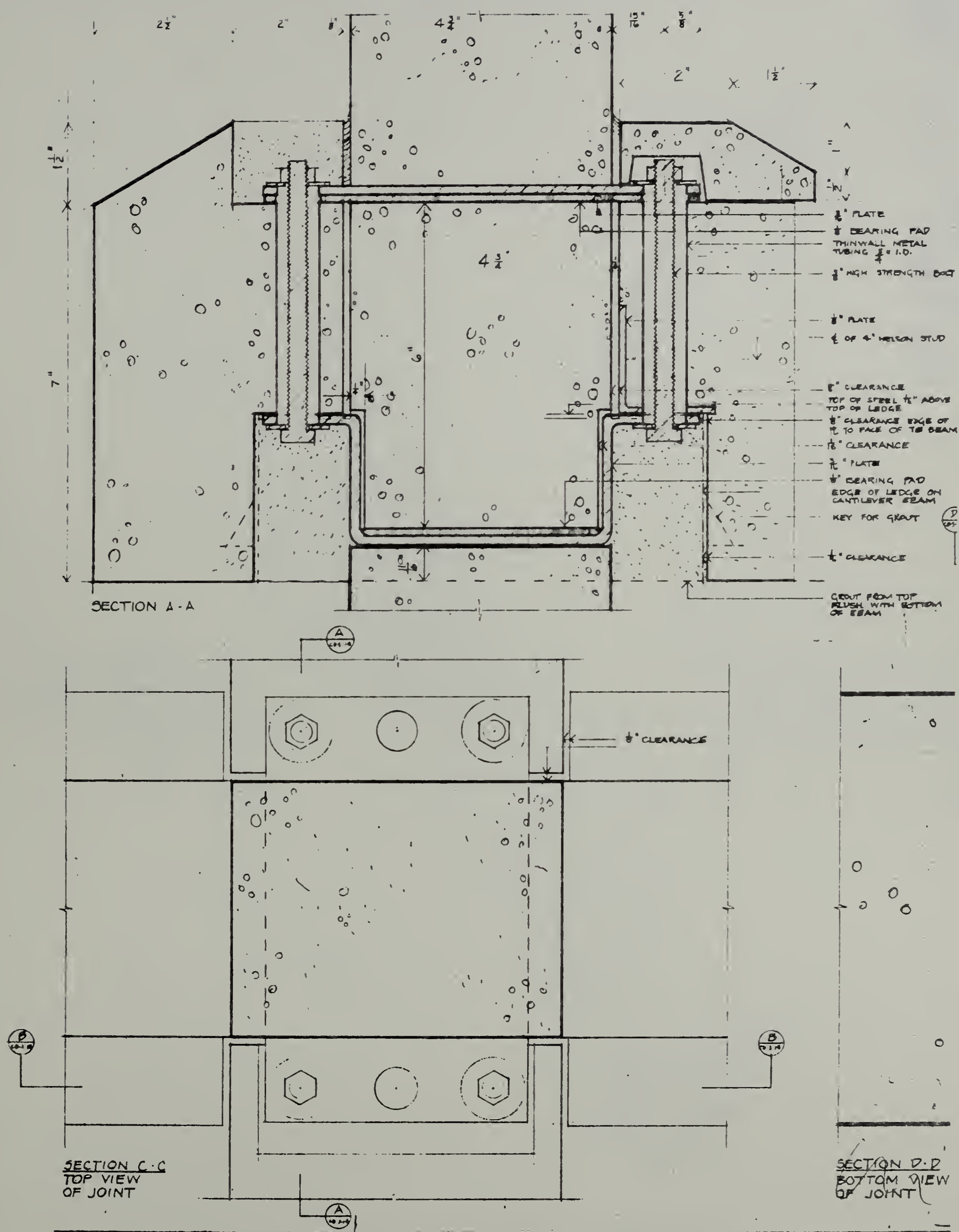


Fig. 5.18 Connection detail

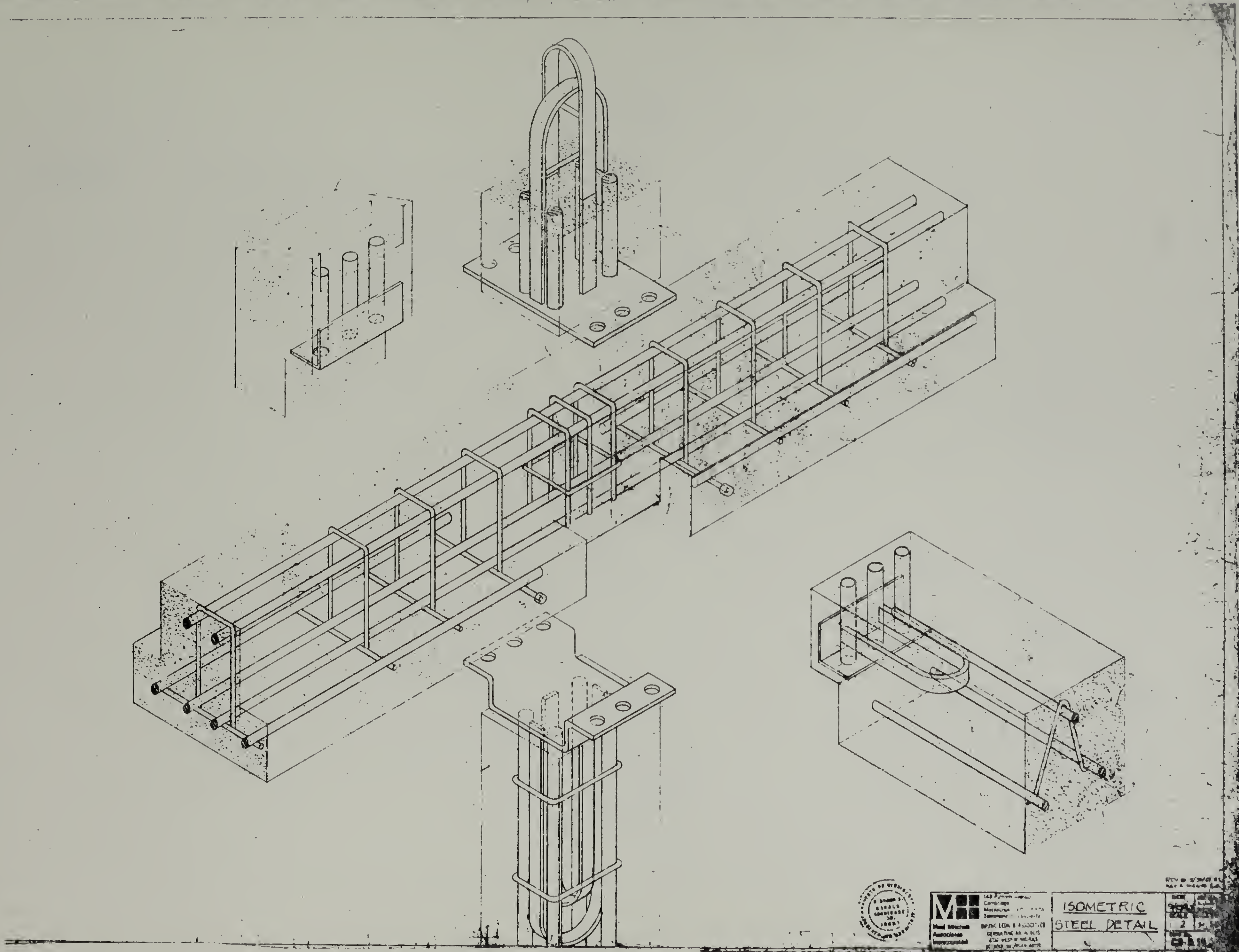


Fig. 5.19 Isometric view of precast-member connection reinforcement details

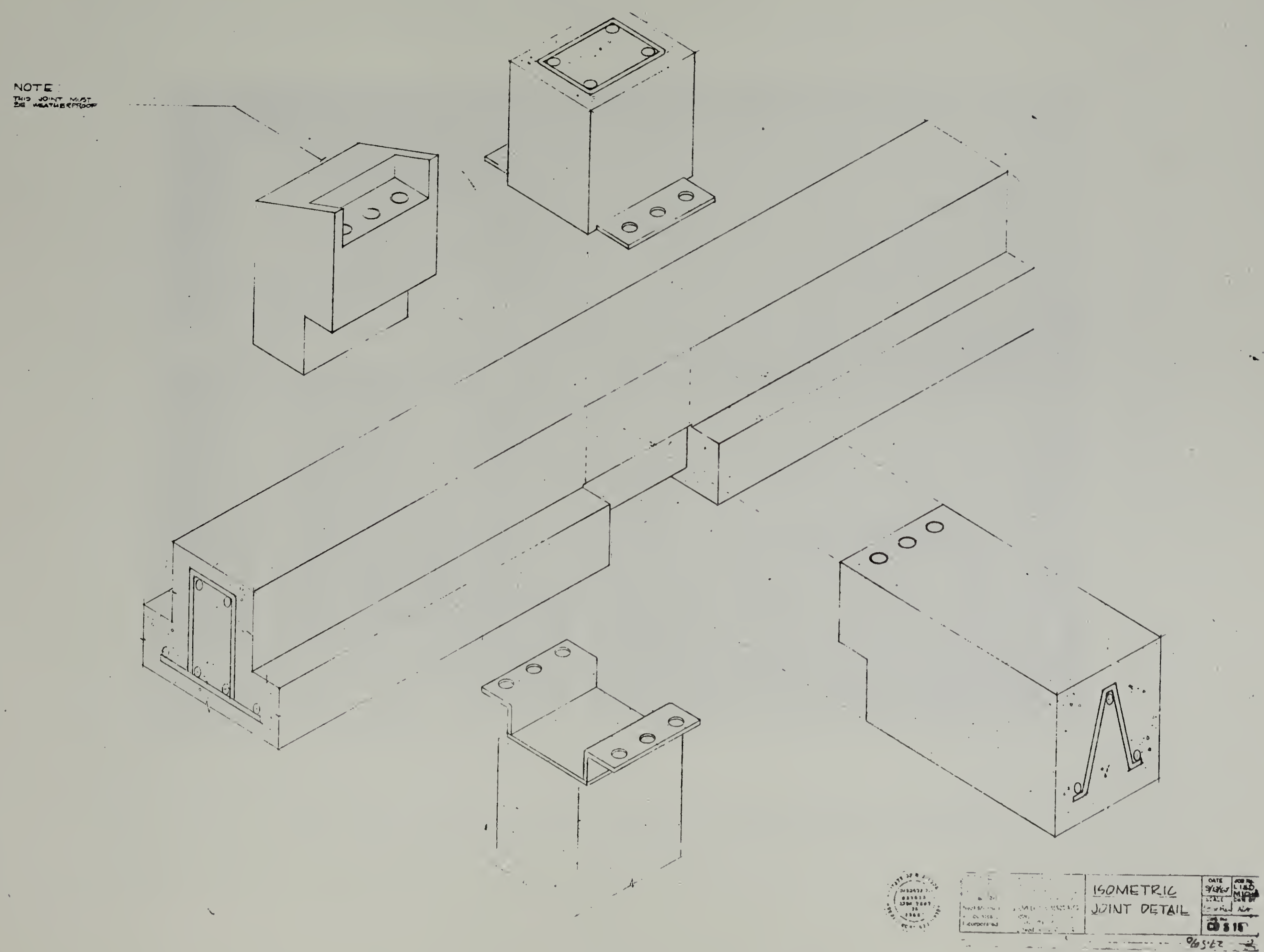


Fig. 5.20 Connections of precast members

Fig. 5.21 Test structure before the installation of walls

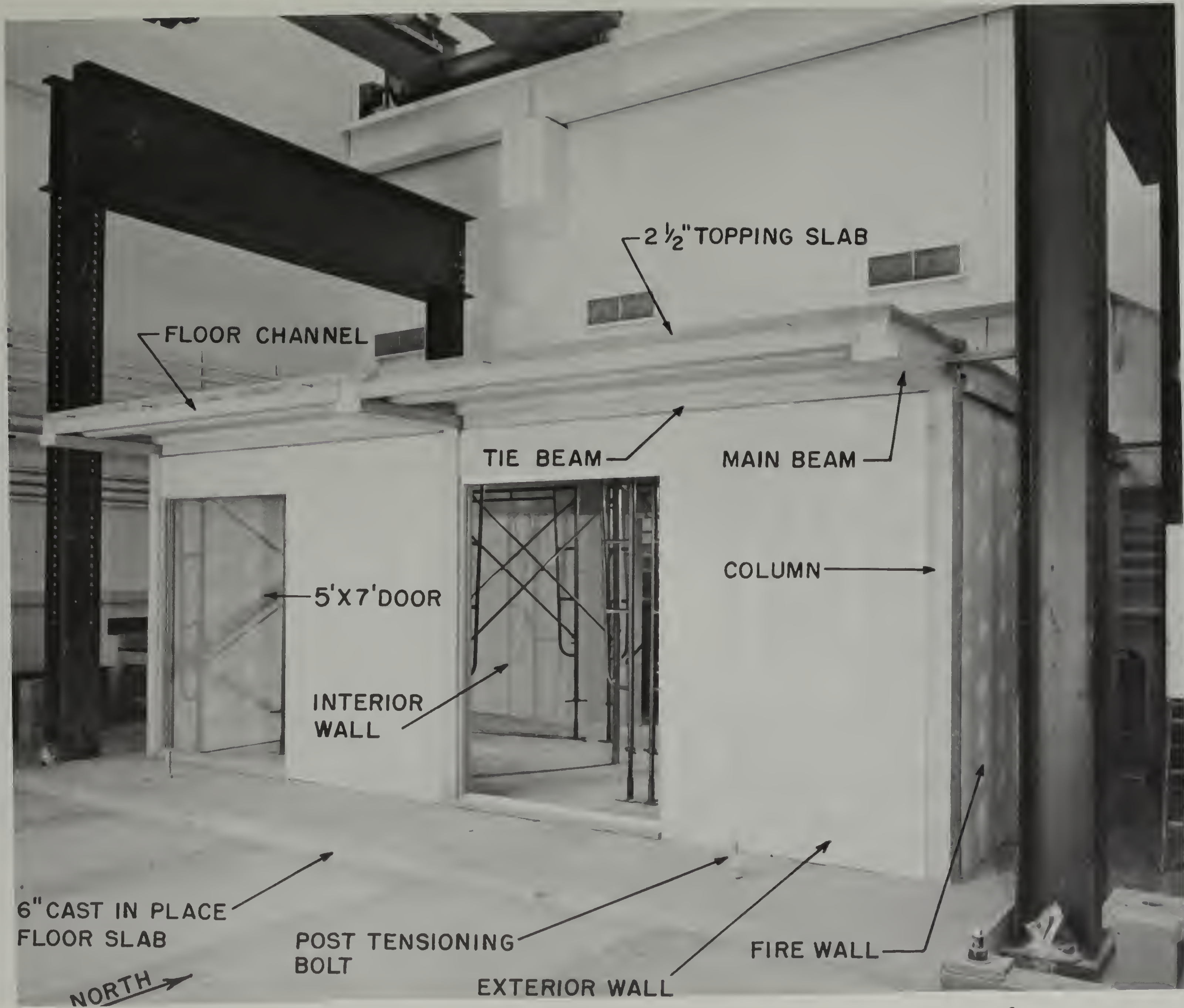


Fig. 5.22 The test structure after the installation of walls

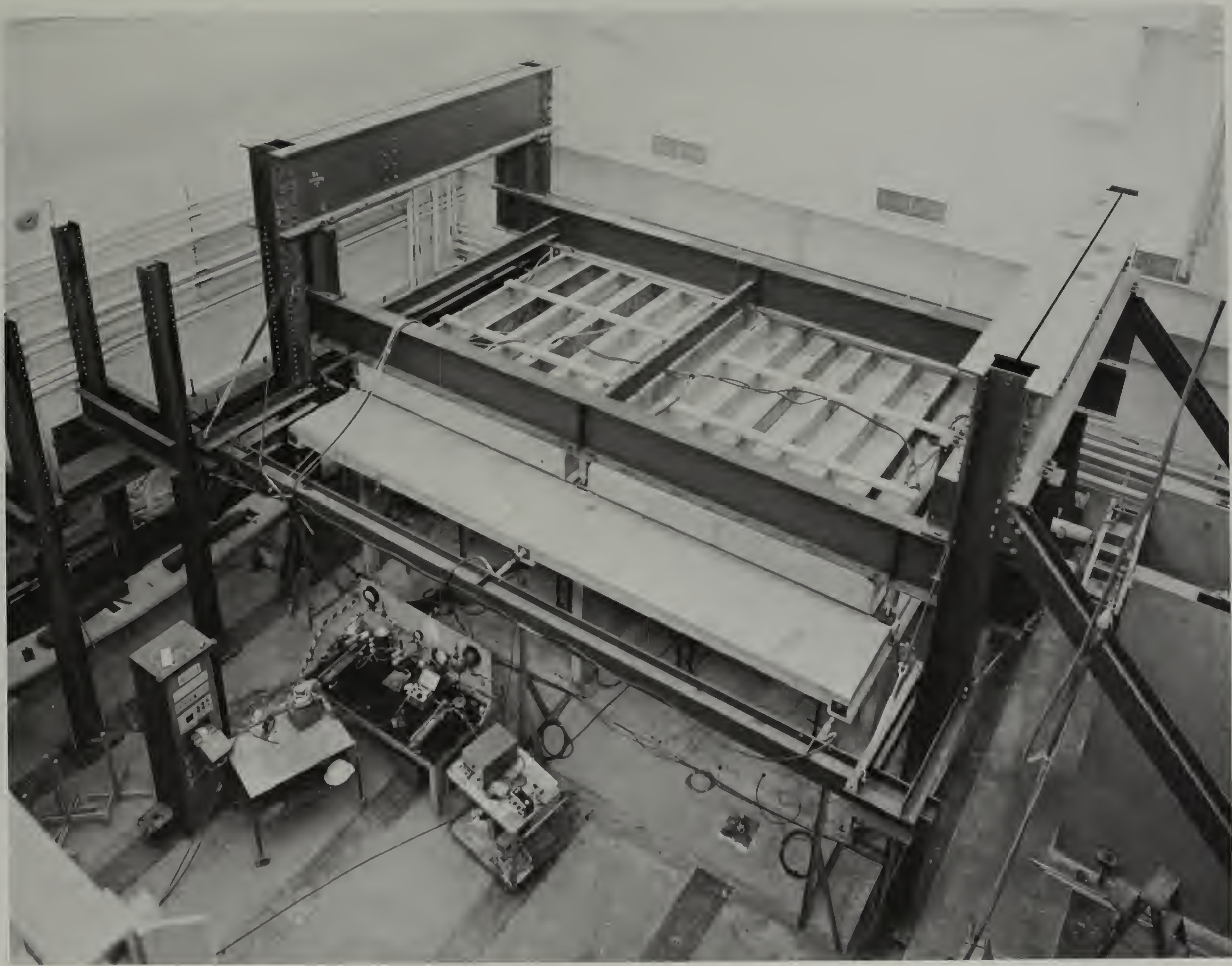


Fig. 5.23 The test structure with testing equipment installed (top view)

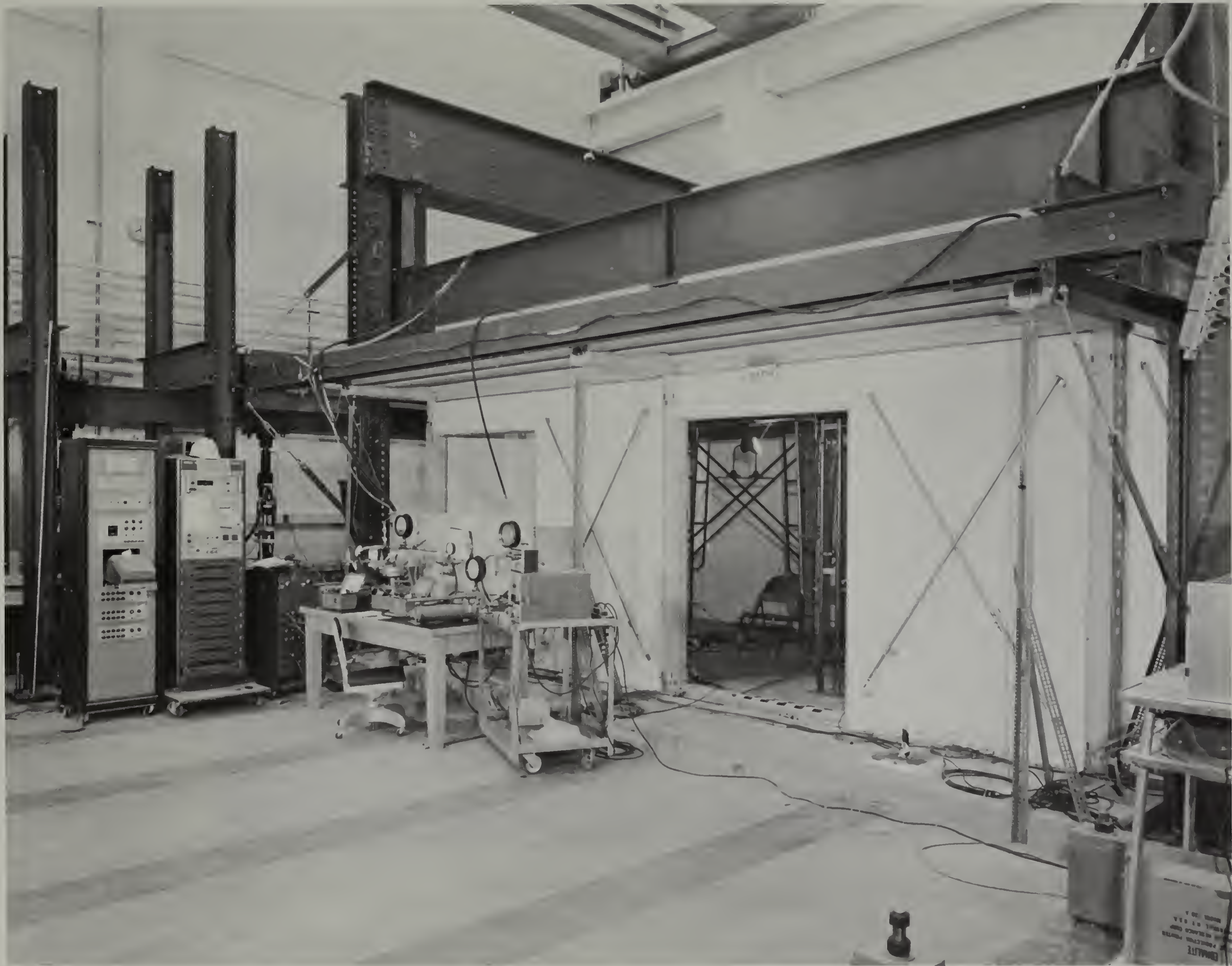


Fig. 5.24 Test structure with testing equipment installed (front view)

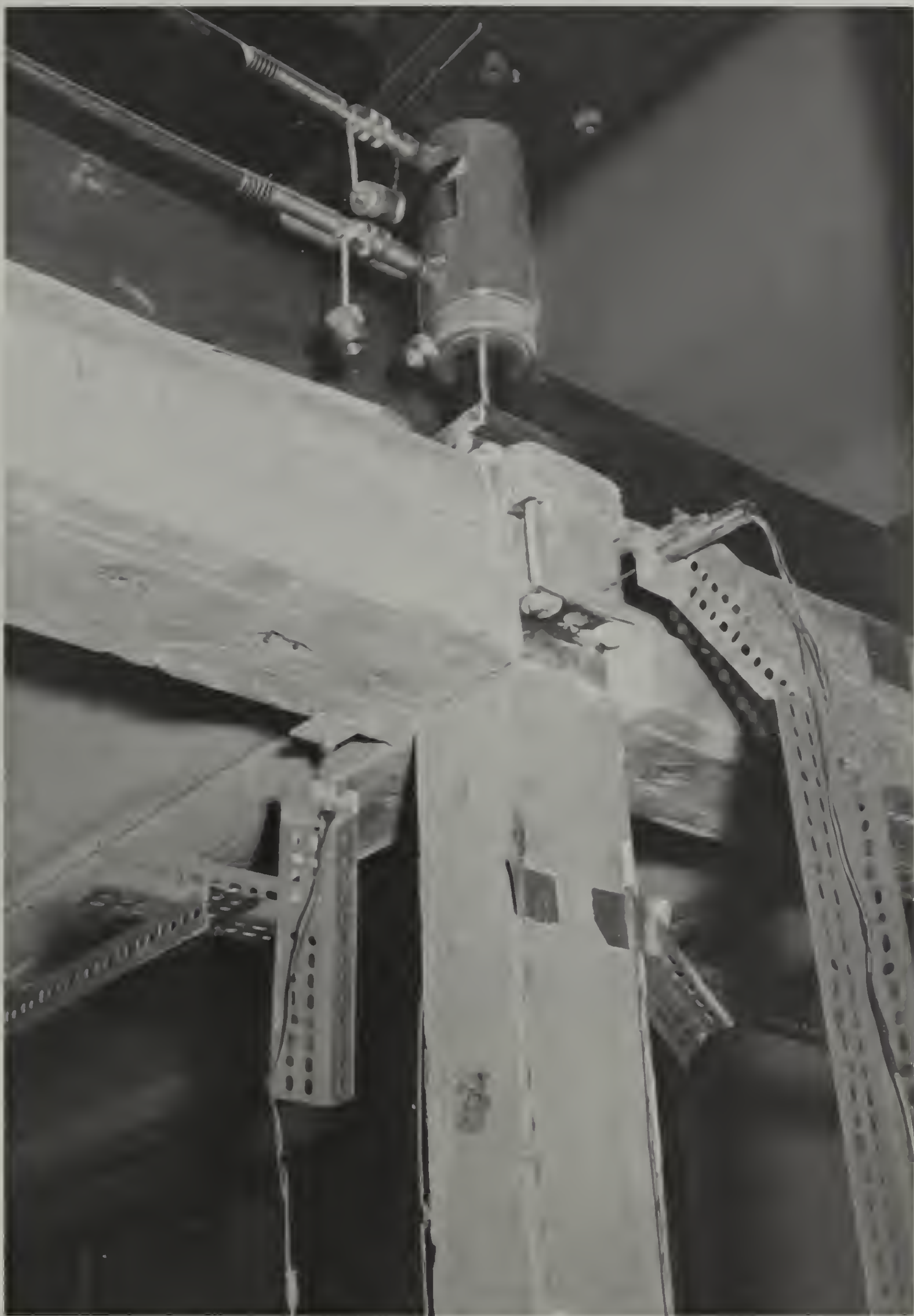


Fig. 5.25 Column load simulation

Fig. 5.26 Wind load simulation

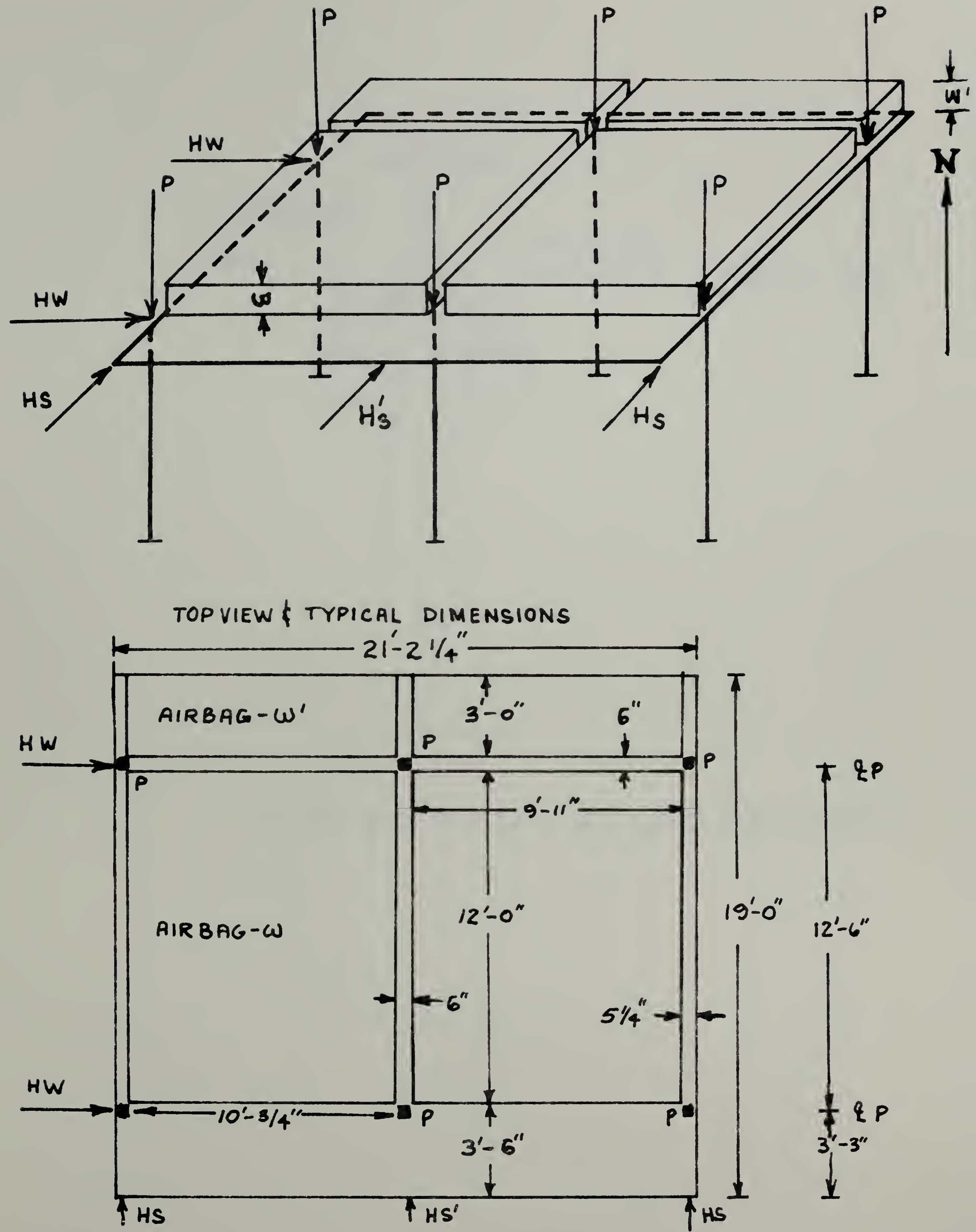


FIGURE 6.1 LOADING OF TEST STRUCTURE

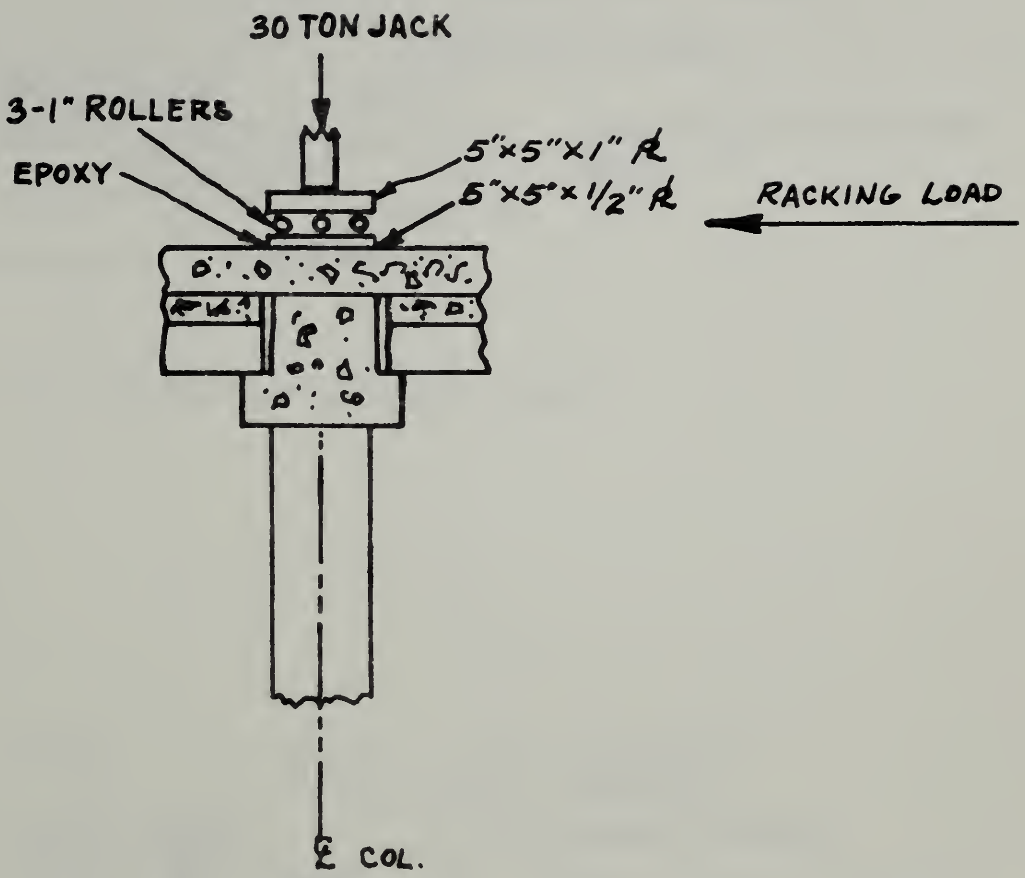


FIGURE 6.2 TYP. COLUMN LOAD SIMULATION

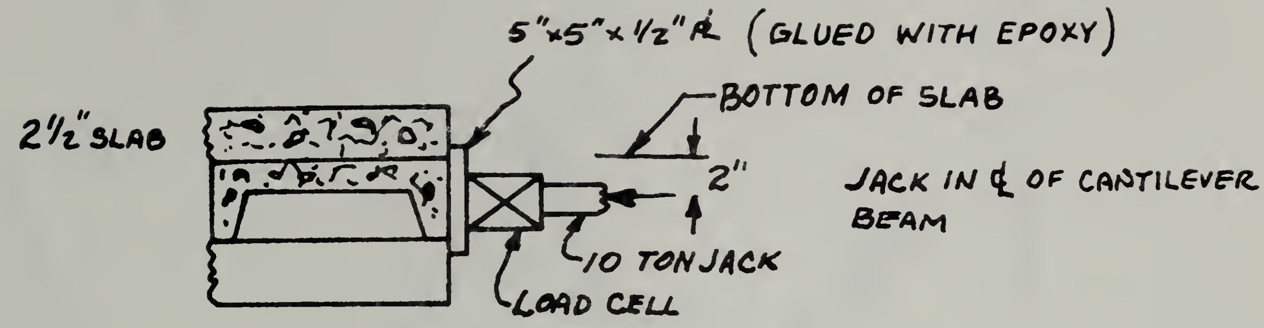


FIGURE 6.3 TYP. N→S RACKING LOAD

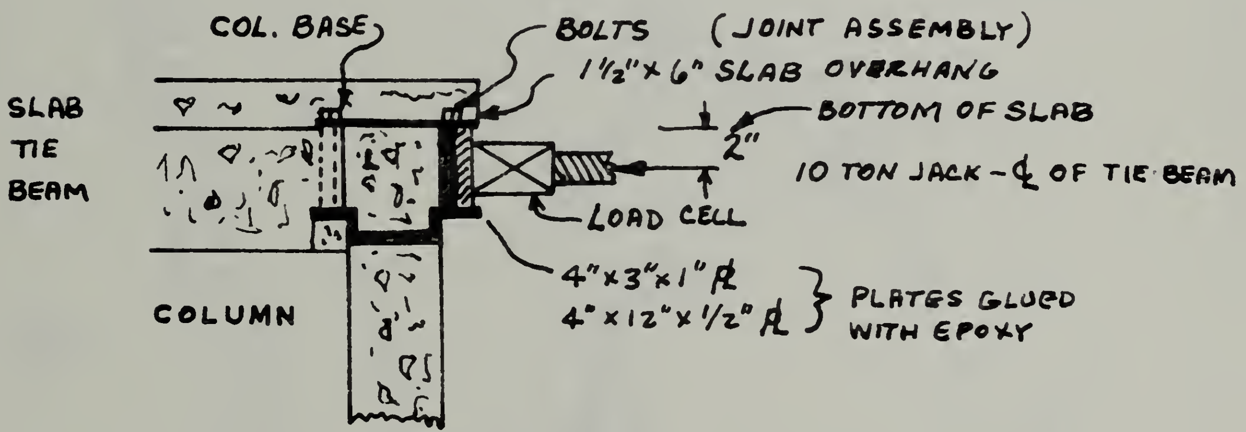


FIGURE 6.4 TYP. W→E RACKING LOAD

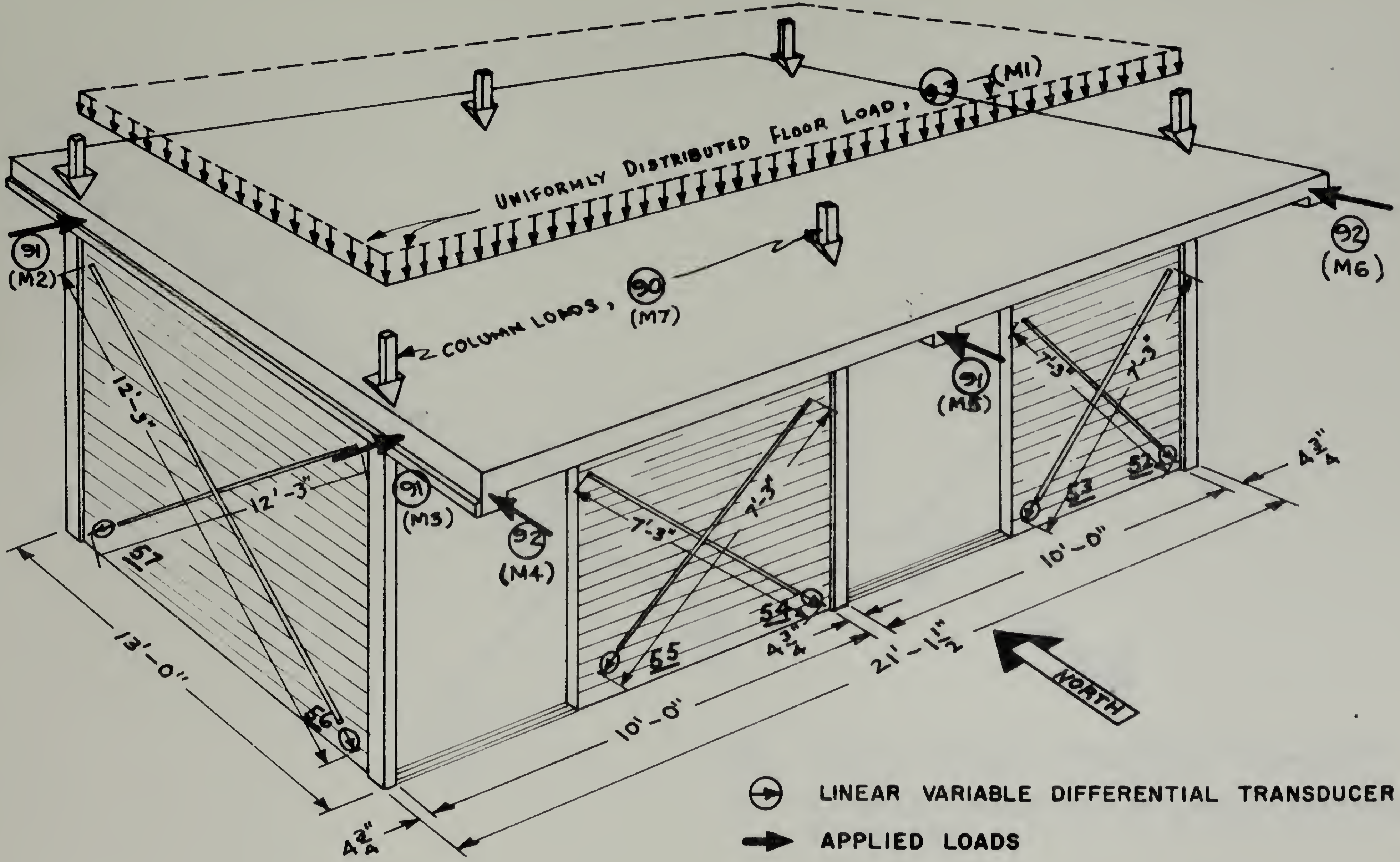


FIGURE NO. 7.1-INSTRUMENTATION LOCATION, NORTHEAST VIEW

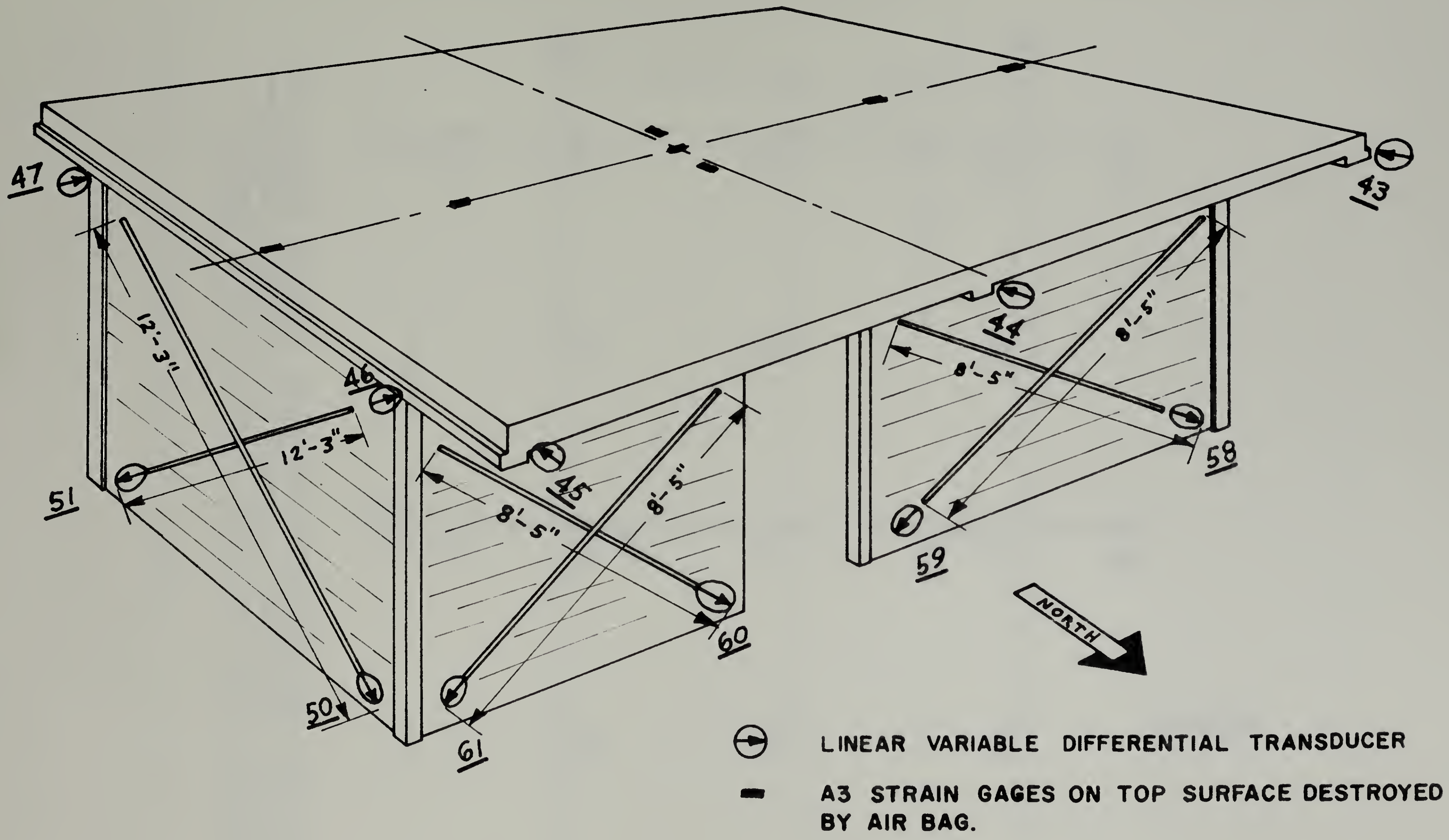


FIGURE NO. 7.2-INSTRUMENTATION LOCATION, SOUTHWEST VIEW

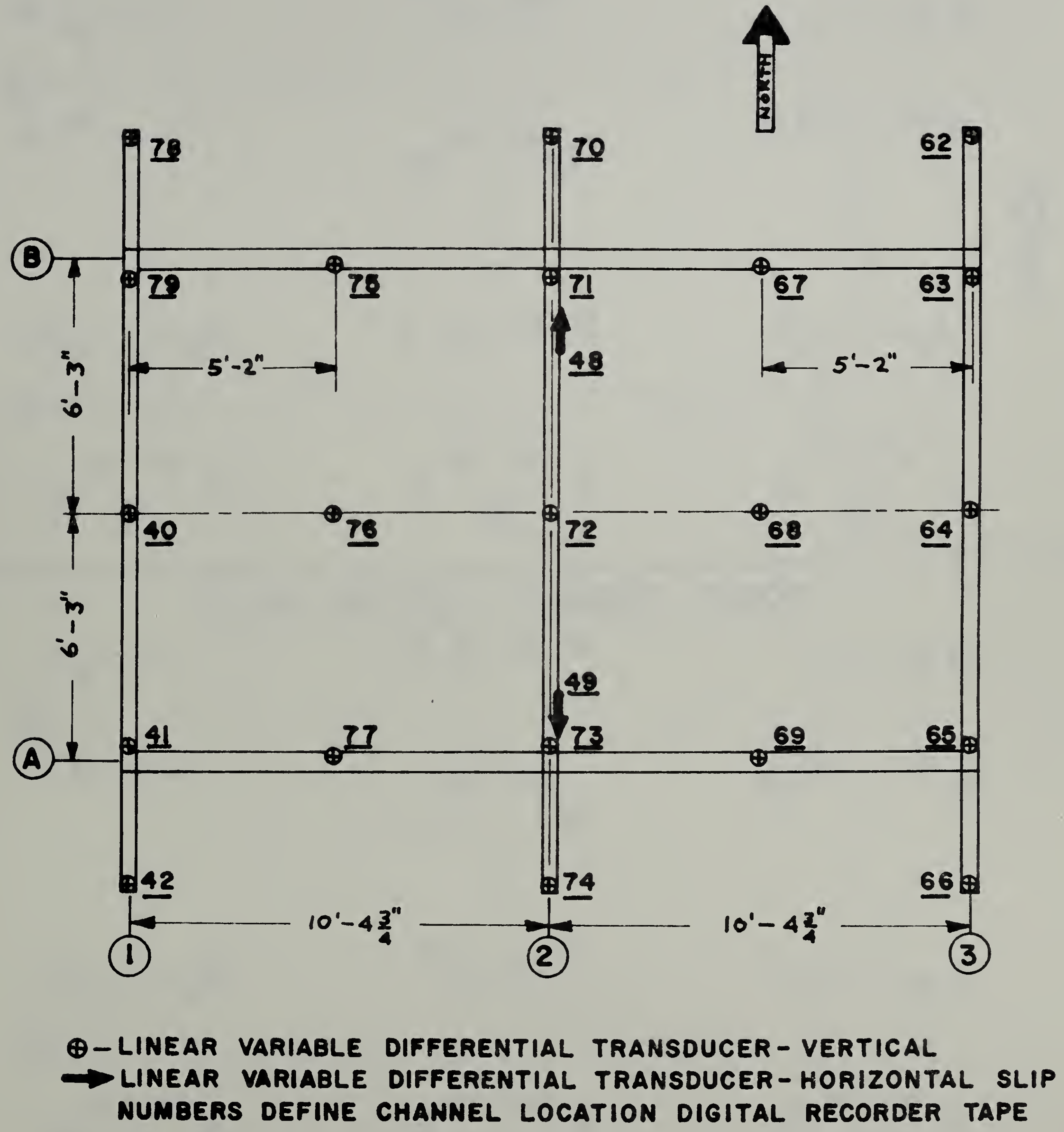
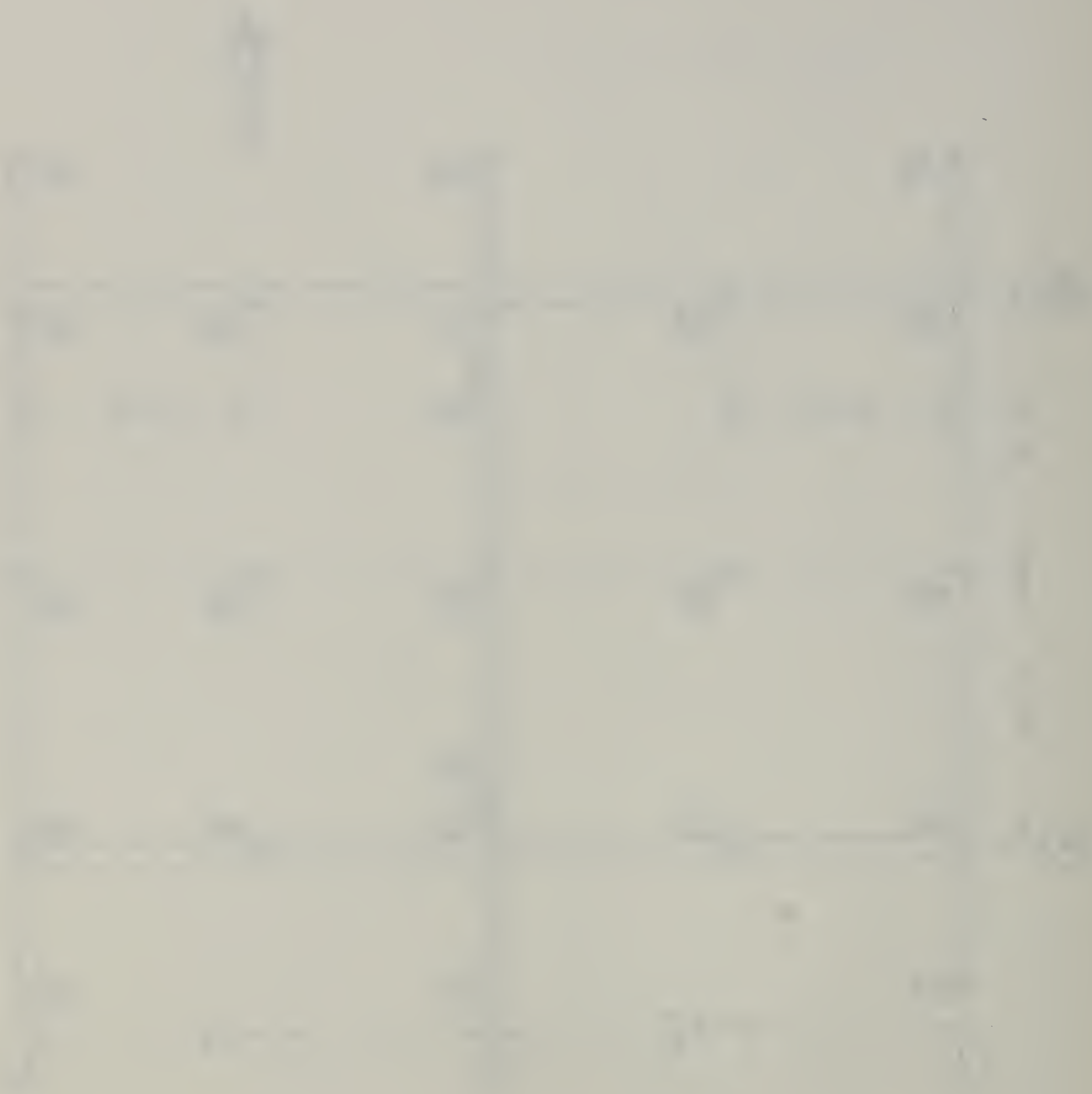


FIGURE NO. 7.3-VERTICAL DEFLECTION GAGES



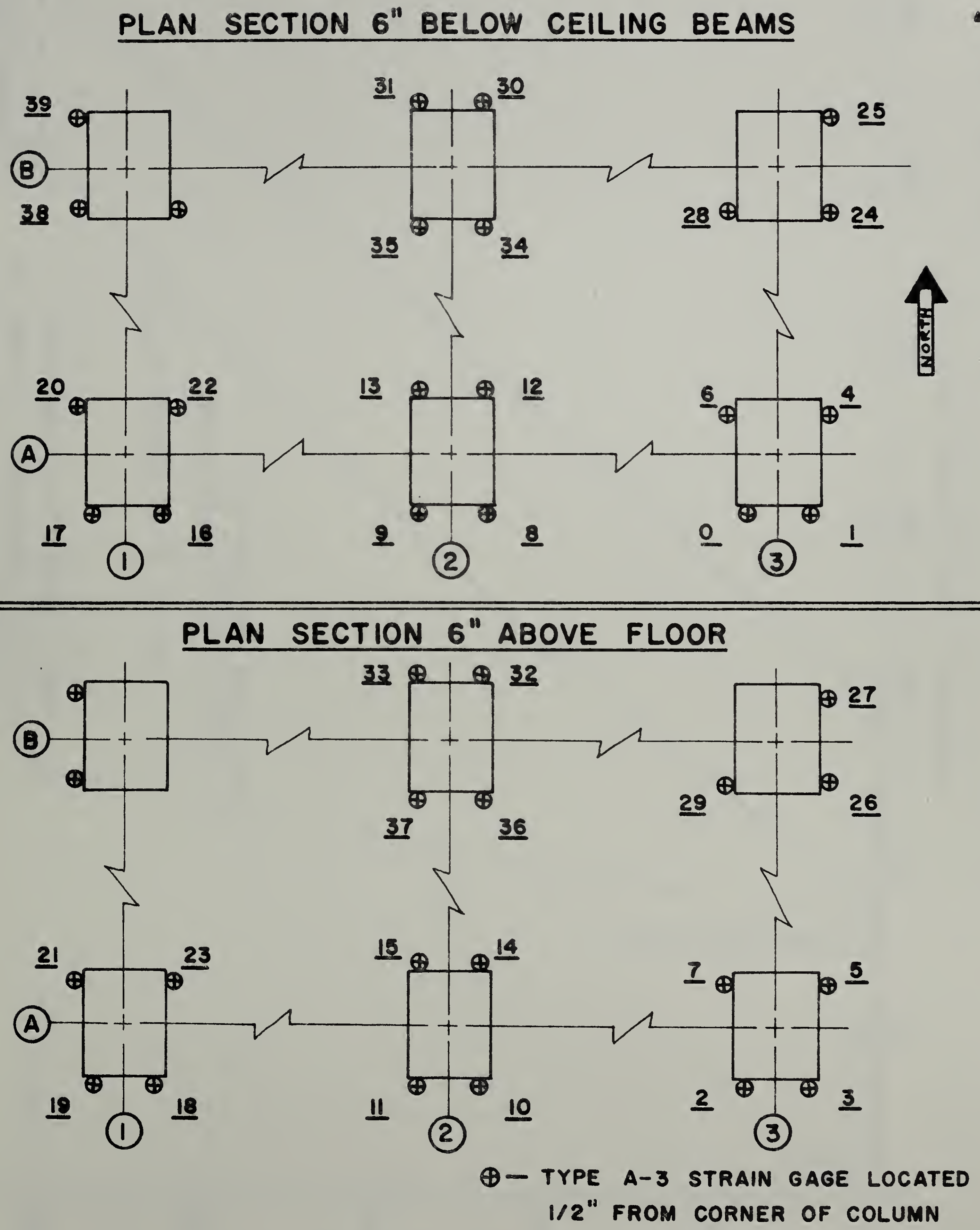


FIGURE NO. 7.4-COLUMN STRAIN GAGE LOCATION

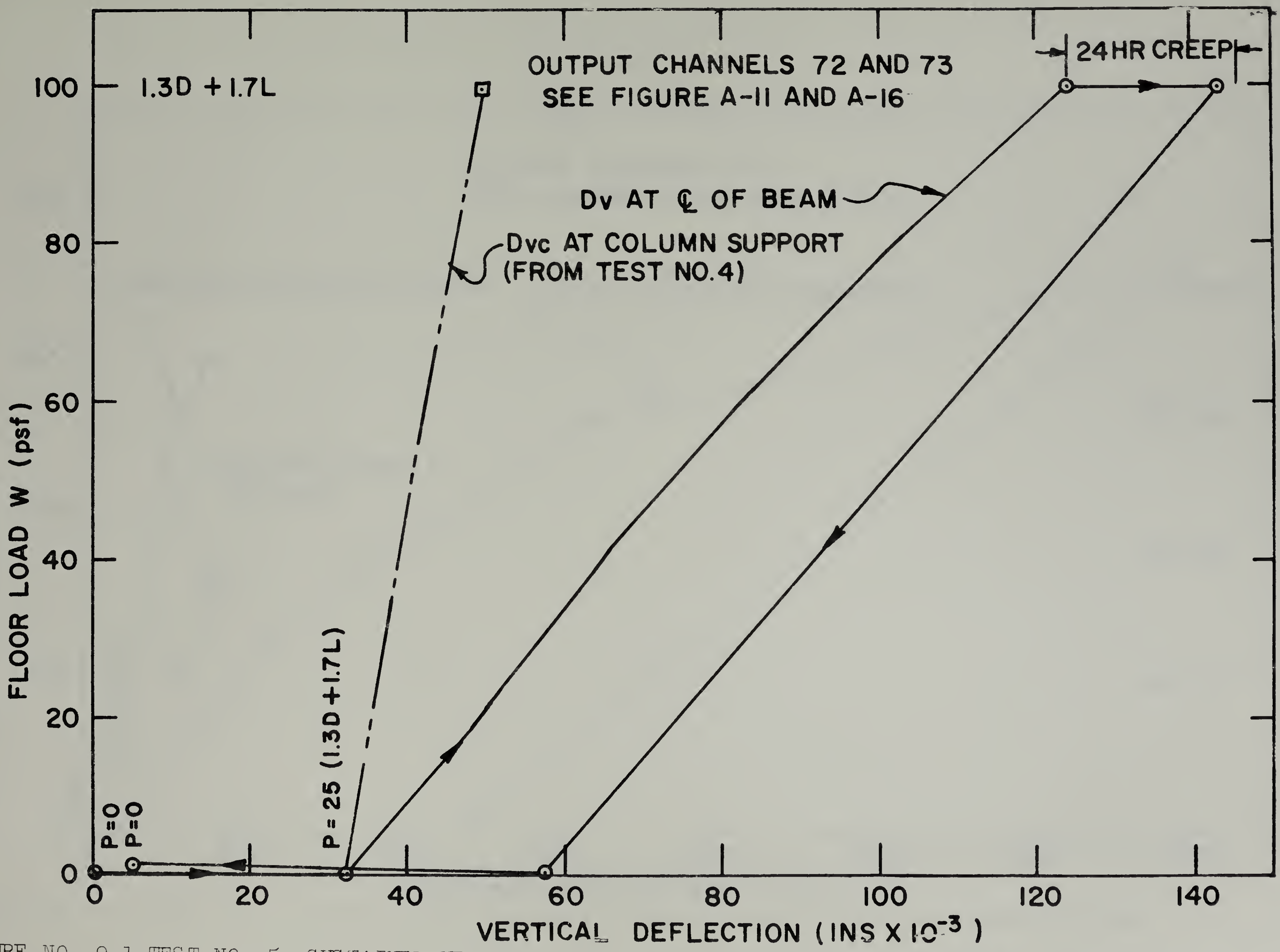


FIGURE NO. 9.1-TEST NO. 5, SUSTAINED VERTICAL LOAD VS MIDSPAN DEFLECTION OF CENTER MAIN BEAM

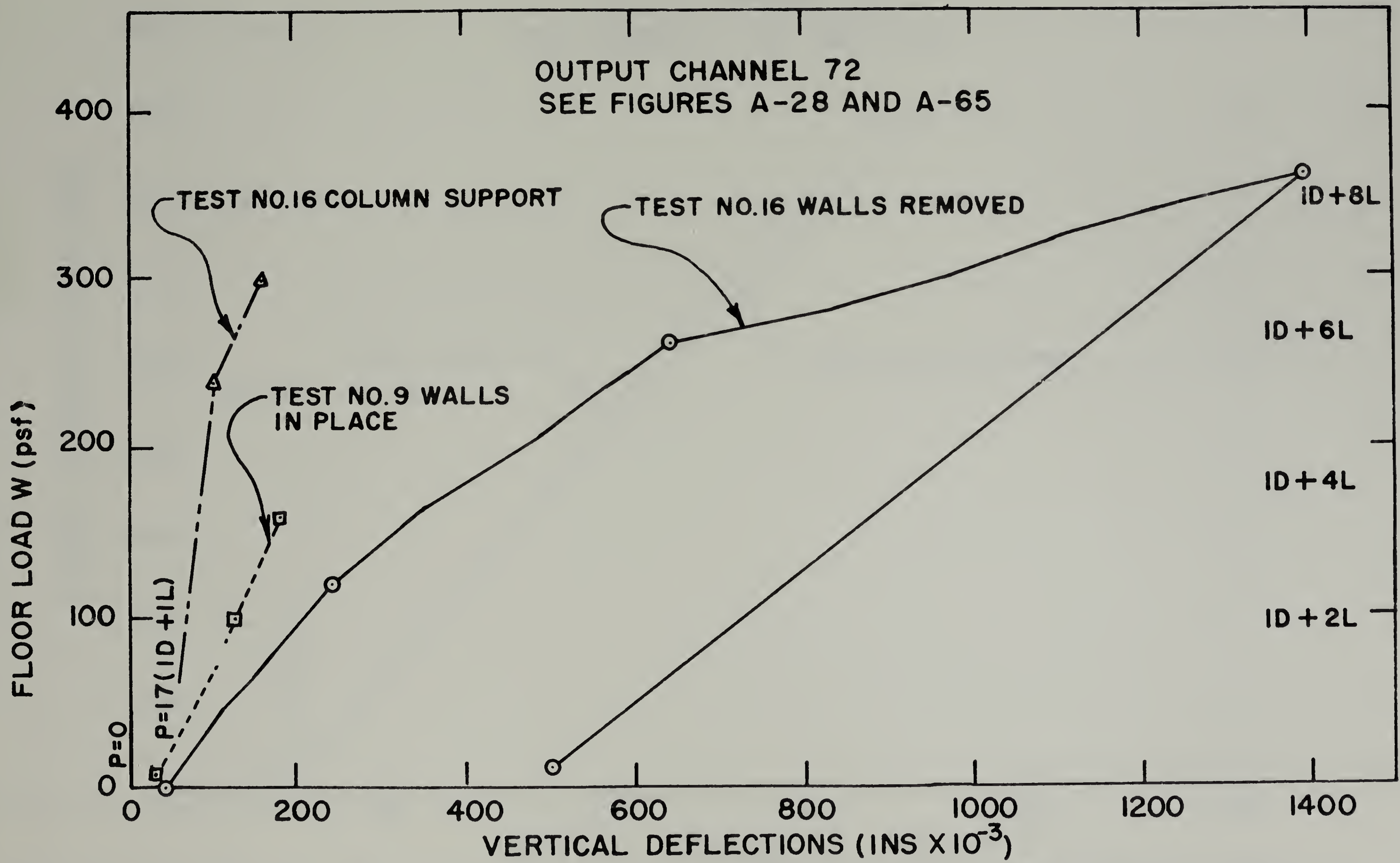


FIGURE NO. 9.2-TEST NO. 16, MAJOR FLOOR LOAD (W) VS MIDSPAN DEFLECTION OF CENTER MAIN BEAM

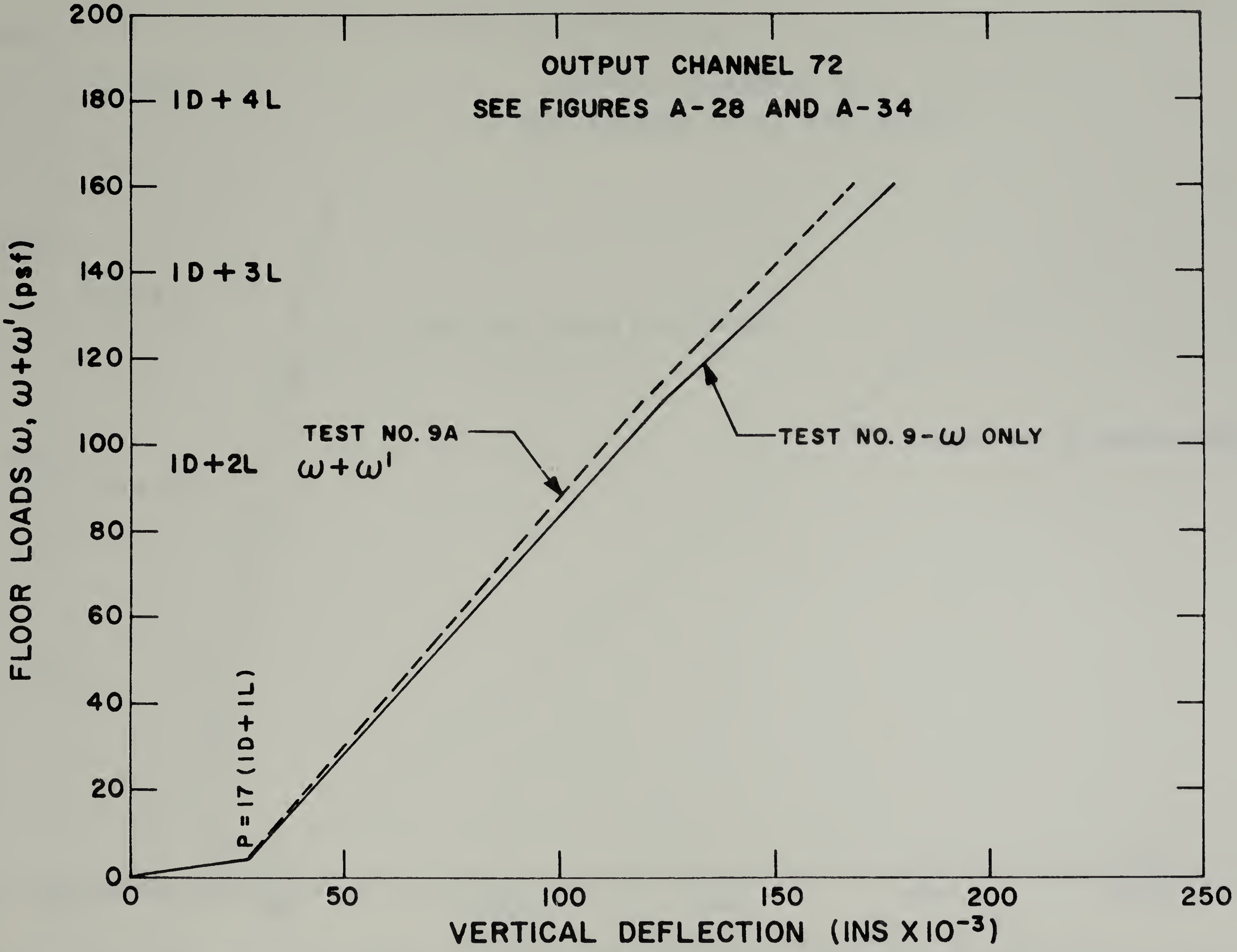


FIGURE NO. 9.3-INTERIOR FLOOR LOAD (W) AND TOTAL FLOOR LOAD (W+W) VS MIDSPAN DEFLECTION OF CENTER MAIN BEAM

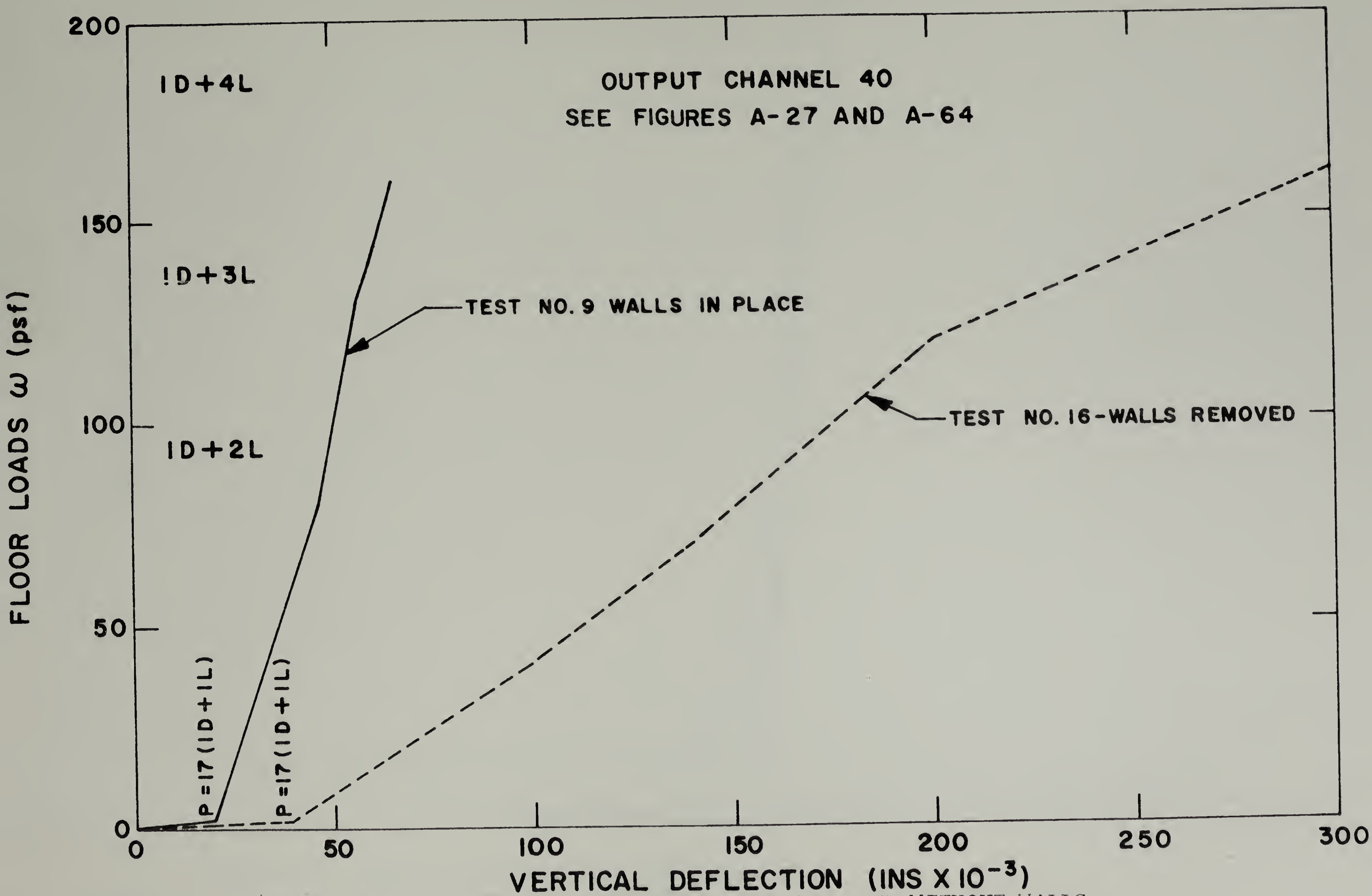


FIGURE NO. 9.4-MIDSPAN DEFLECTION OF WEST MAIN BEAM WITH AND WITHOUT WALLS

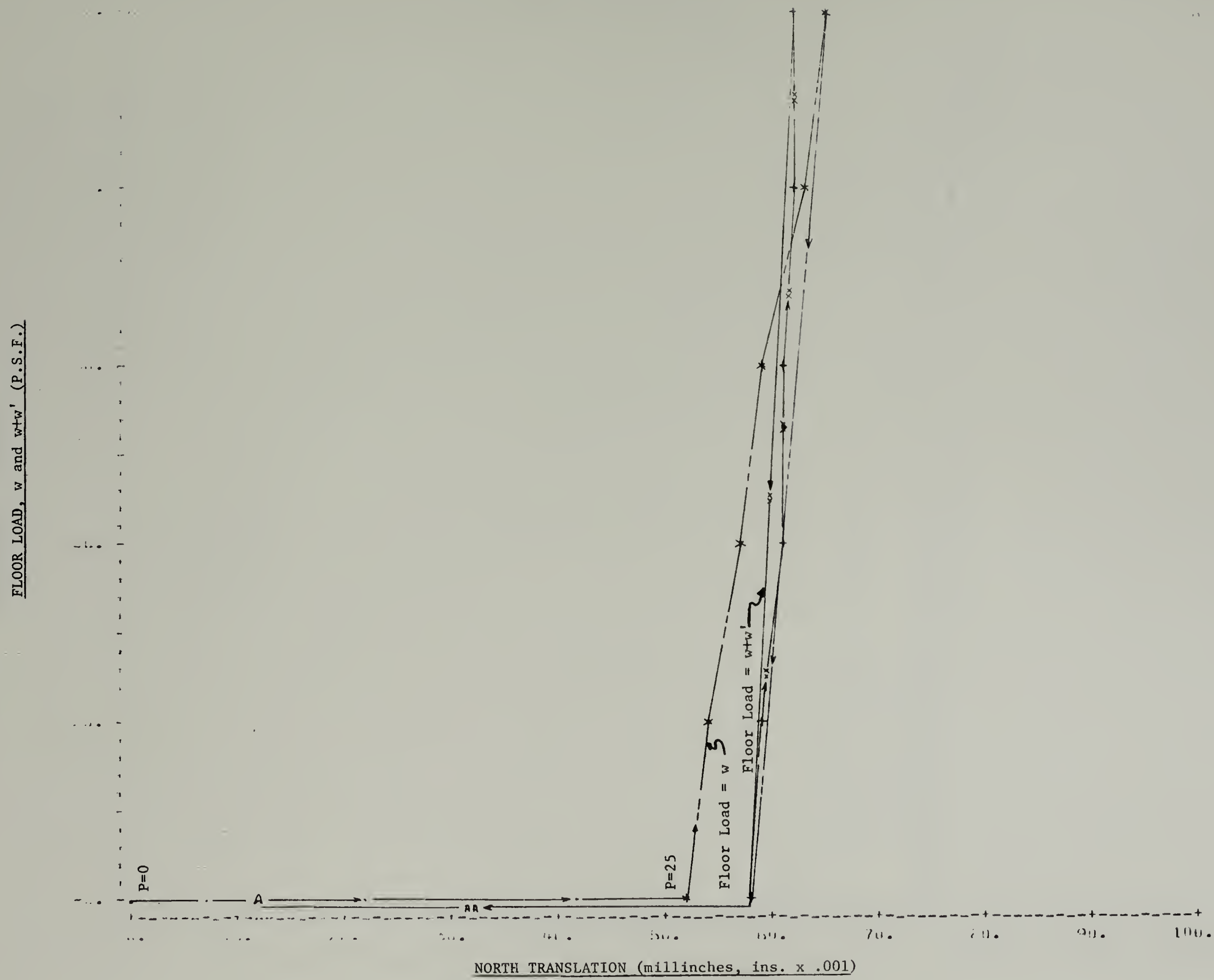


FIGURE NO. 9.5 - TEST NO. 12A and 13A, VERTICAL LOAD VS. NORTH TRANSLATION

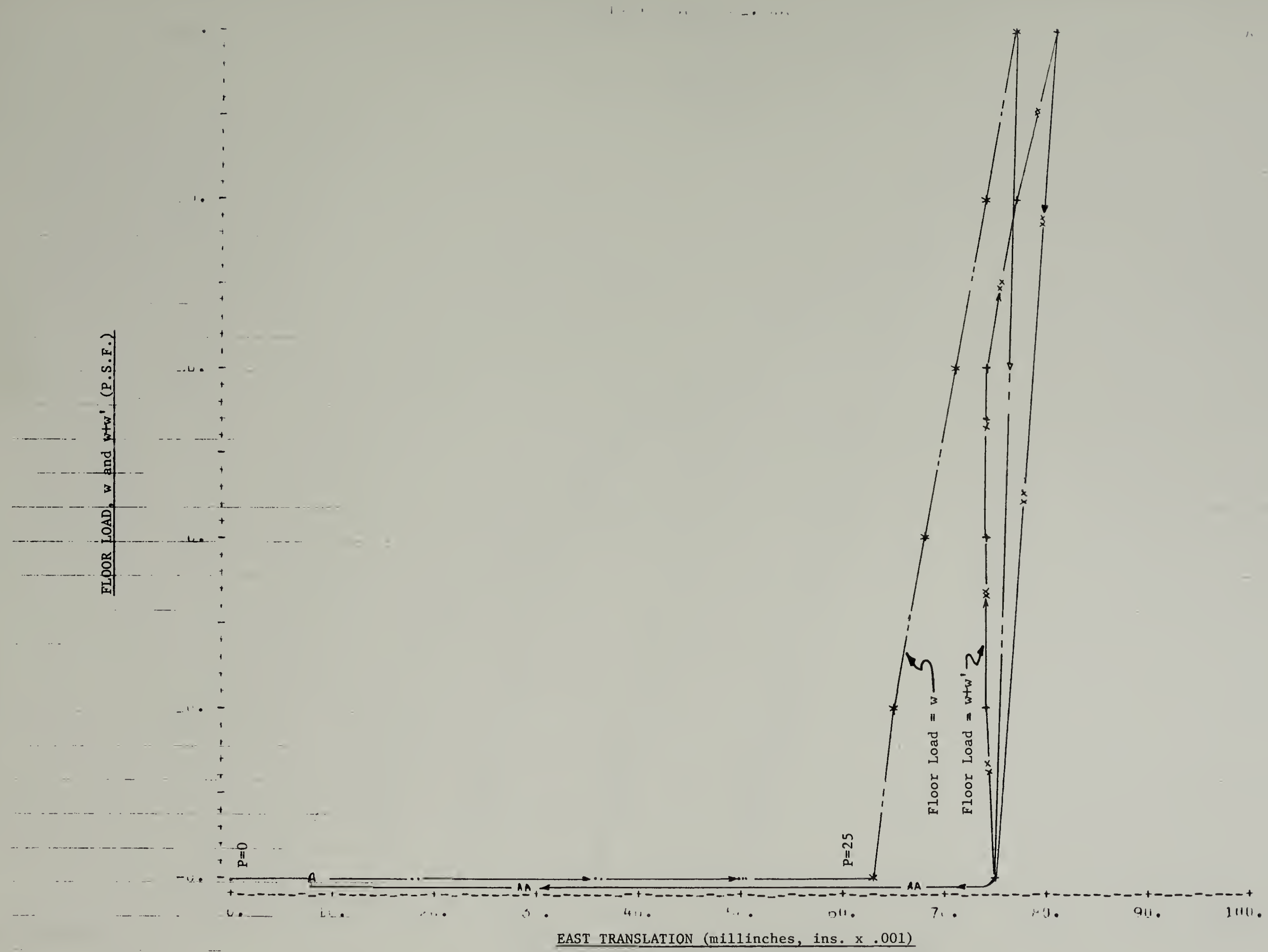


FIGURE NO. 9.6 - TEST NO. 12 & 13, VERTICAL LOAD VS. EAST TRANSLATION

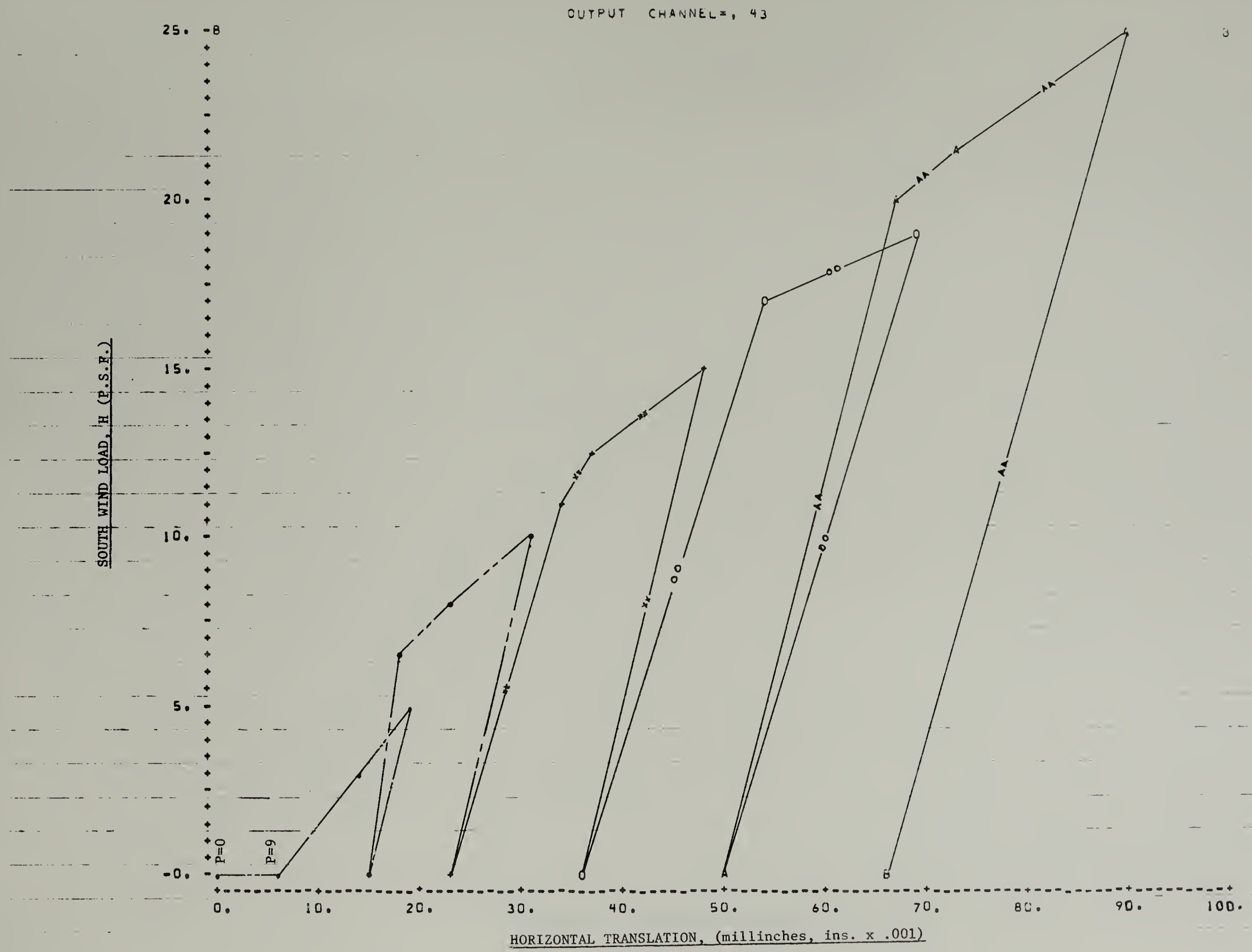


FIGURE NO. 9.7 - TEST NO. 2, SOUTH WIND LOAD VS. TRANSLATION

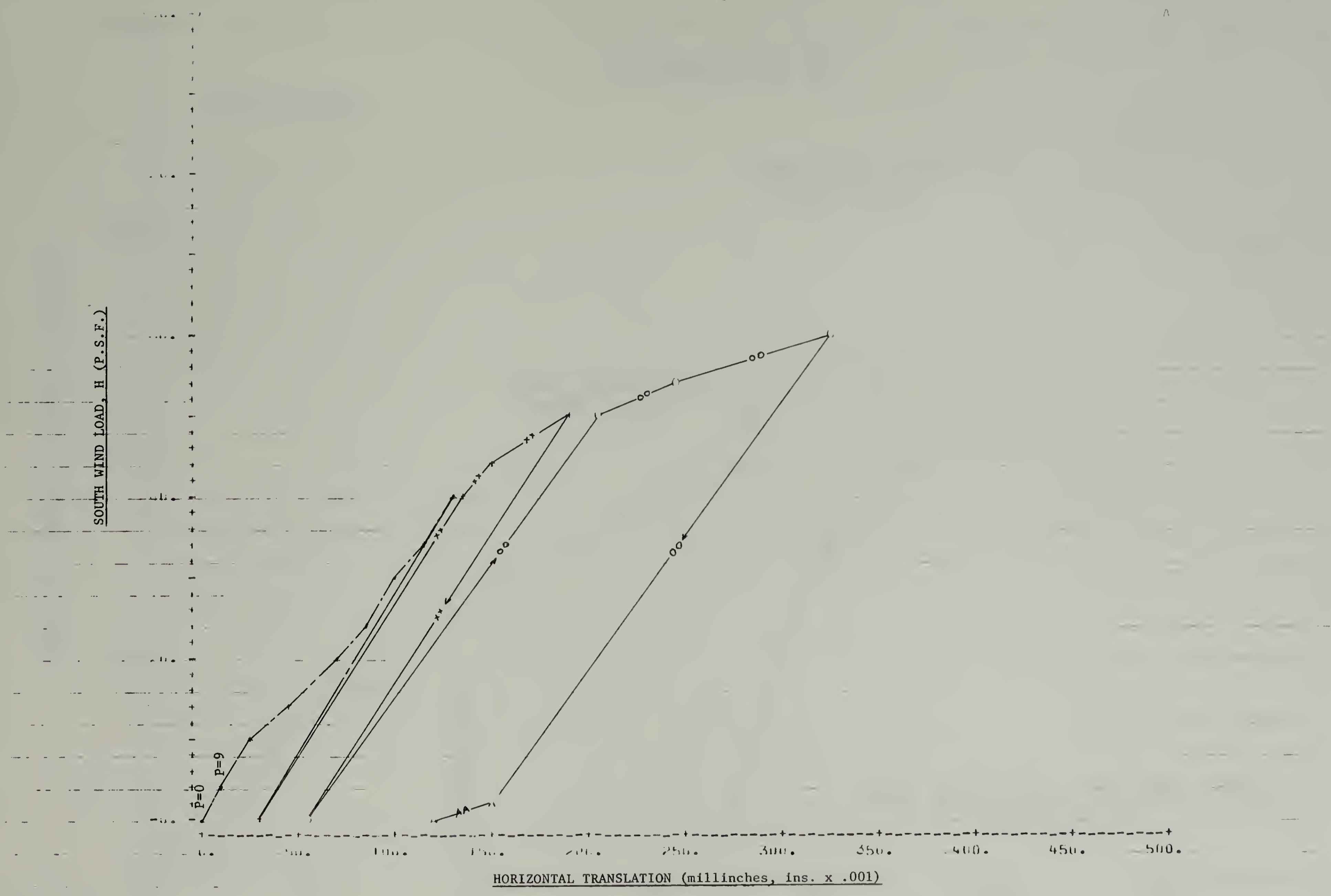


FIGURE NO. 9.8 - TEST NO. 10, SOUTH WIND LOAD VS. TRANSLATION

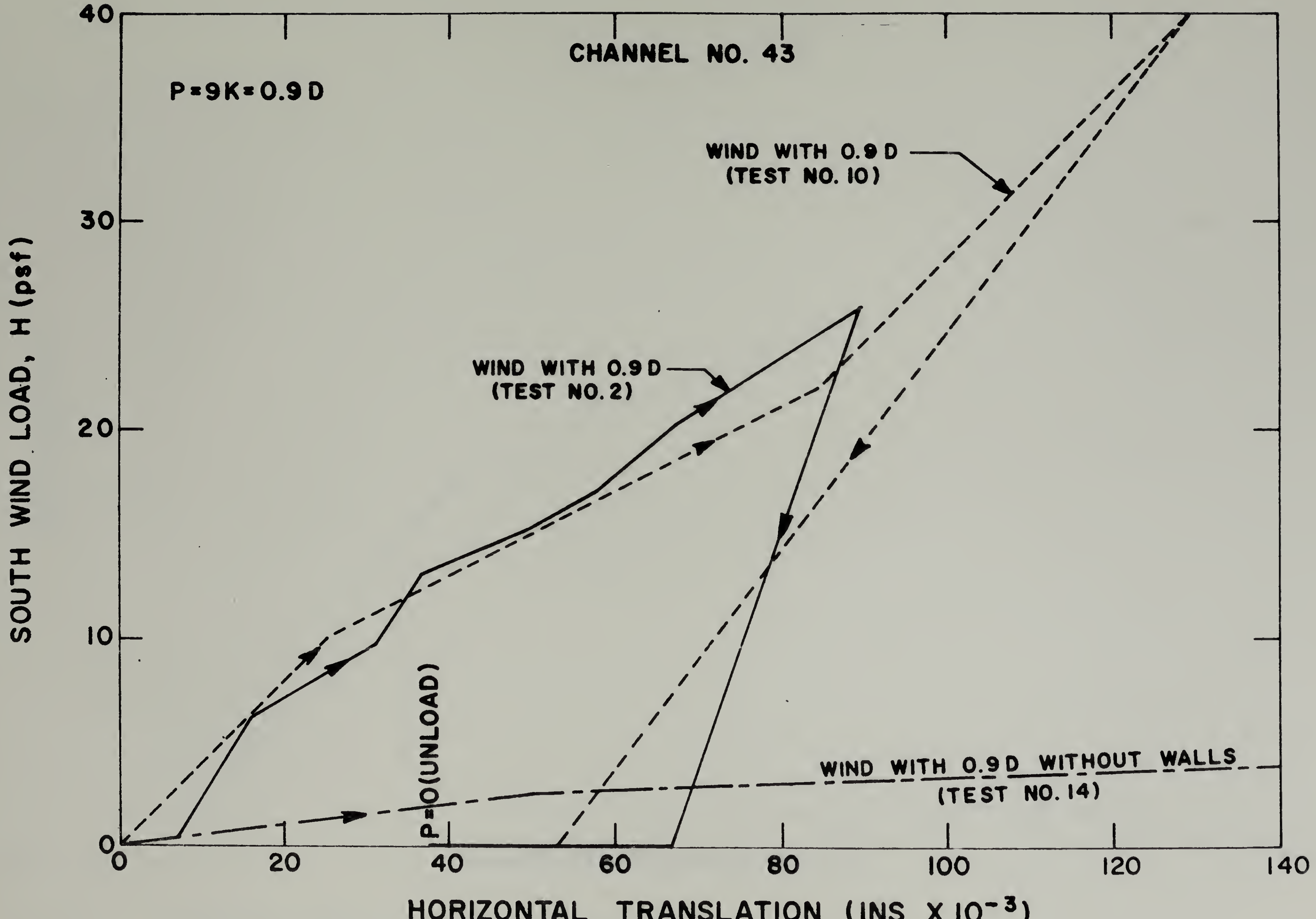
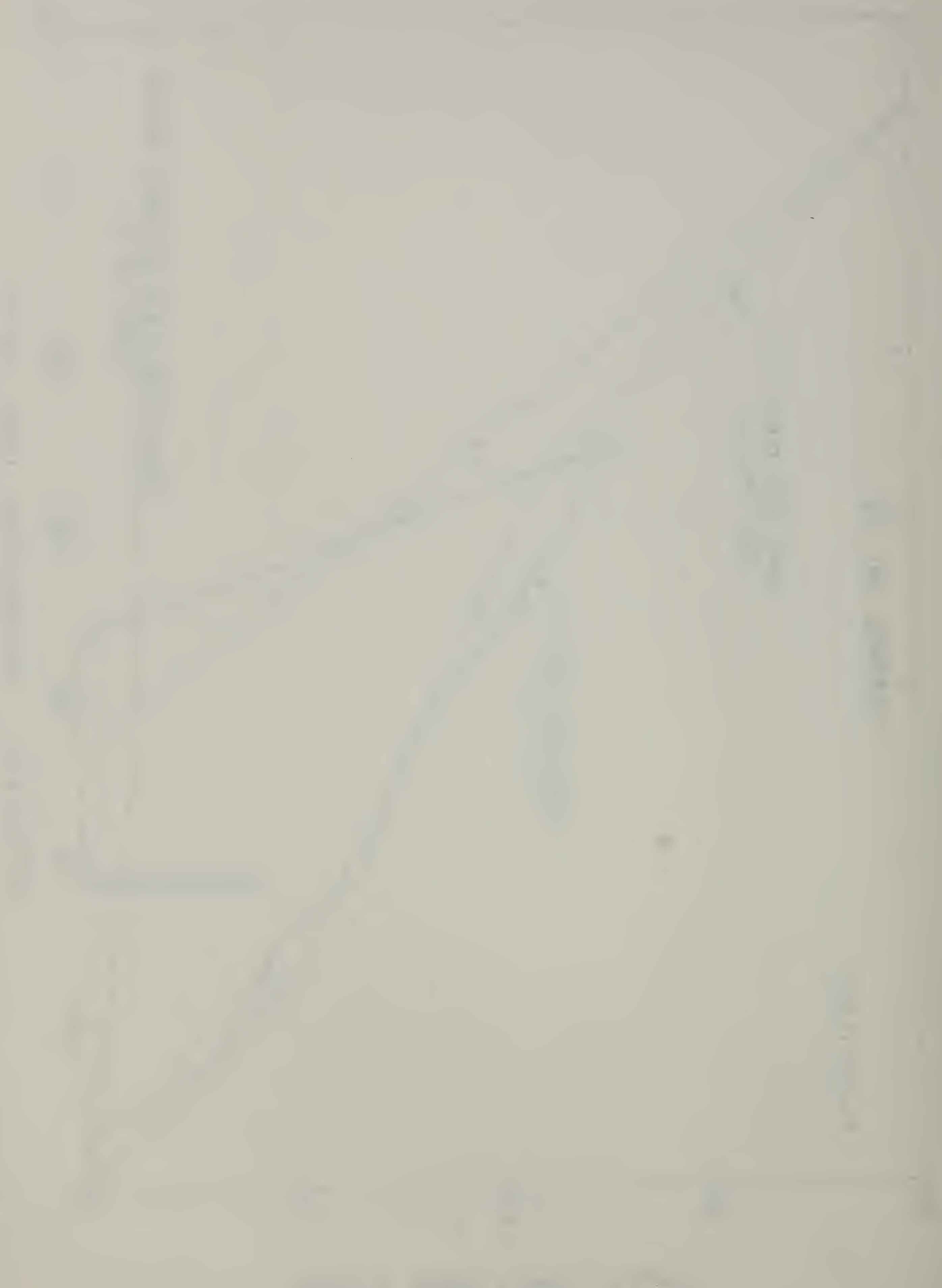


FIGURE NO. 9.9-N-S HORIZONTAL TRANSLATION OF STRUCTURE WITH AND WITHOUT WALLS



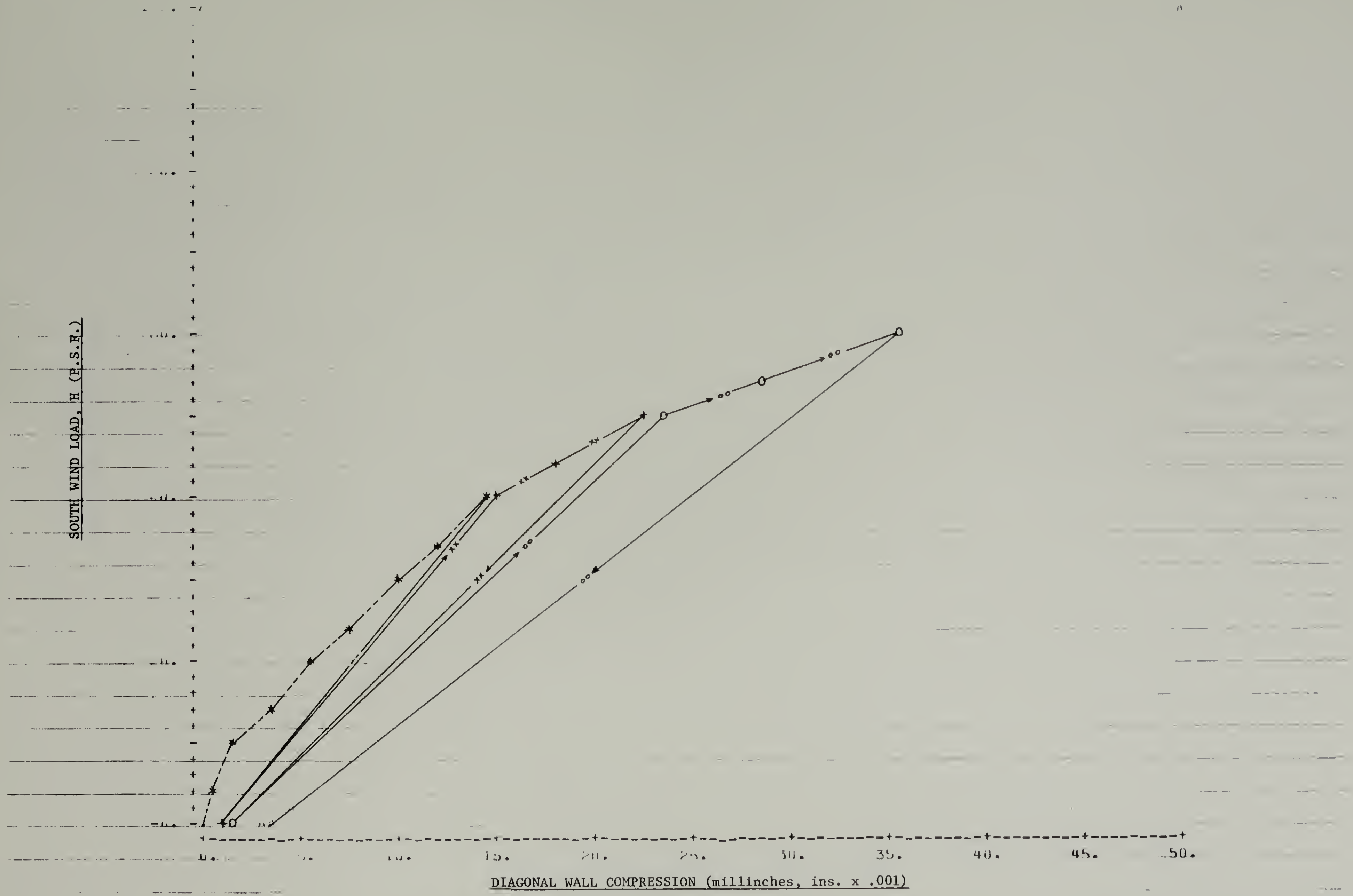


FIGURE NO. 9.10 - TEST NO. 10, SOUTH WIND LOAD VS. WALL COMPRESSION

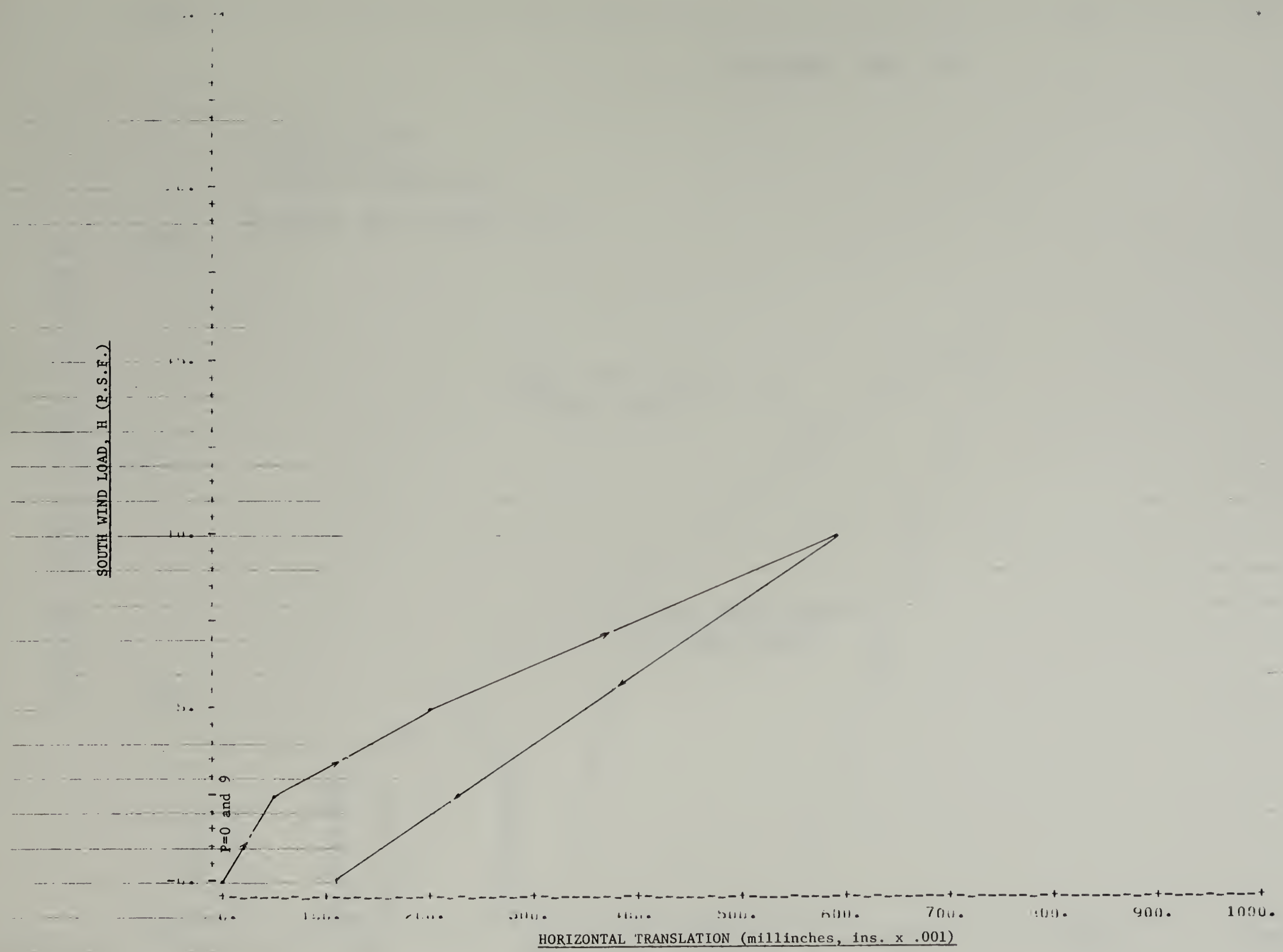


FIGURE NO. **9.11** TEST NO. 14, SOUTH WIND LOAD VS. TRANSLATION

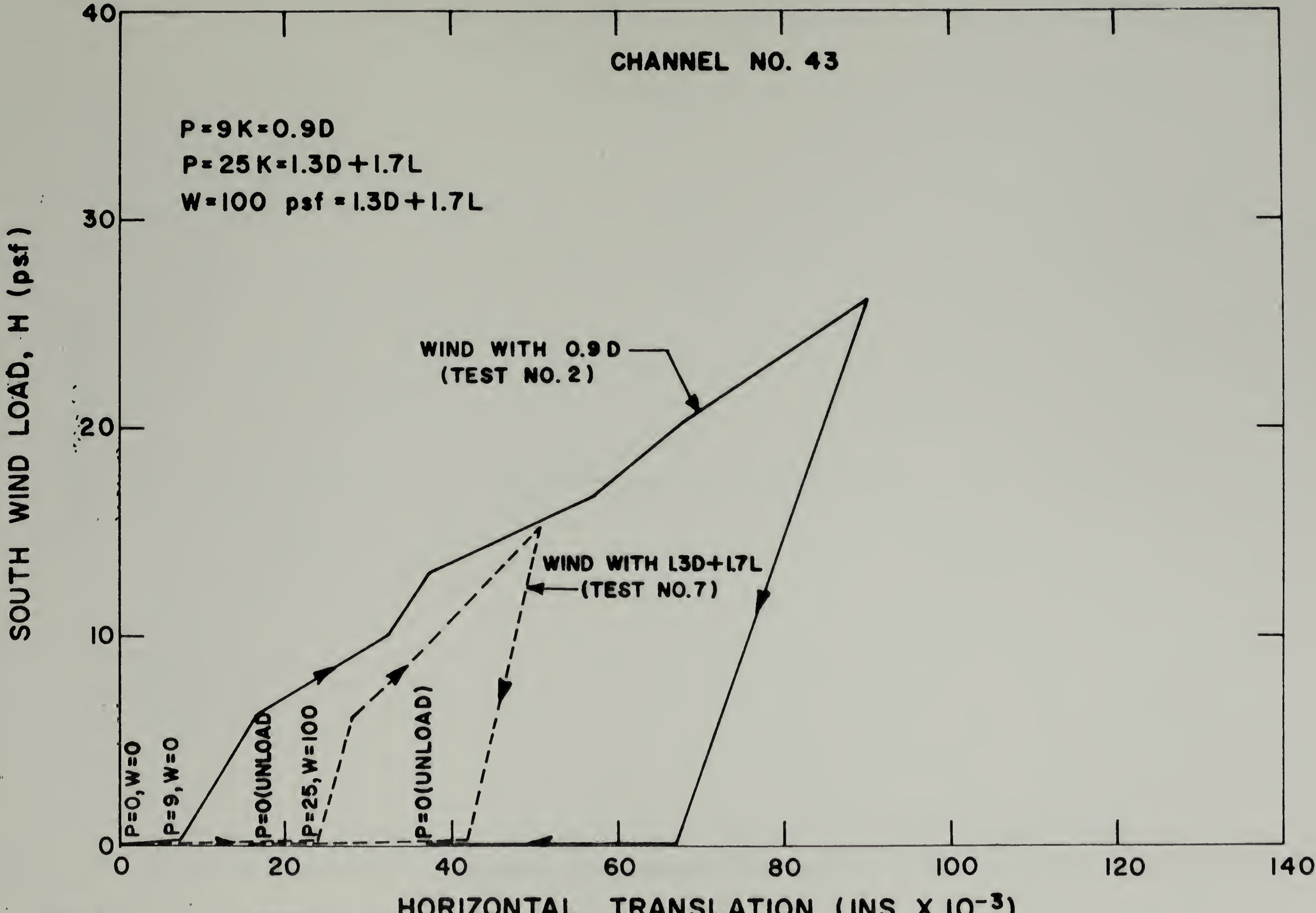


FIGURE NO. 9.12-EFFECT OF VERTICAL LOADS ON N-S HORIZONTAL TRANSLATION

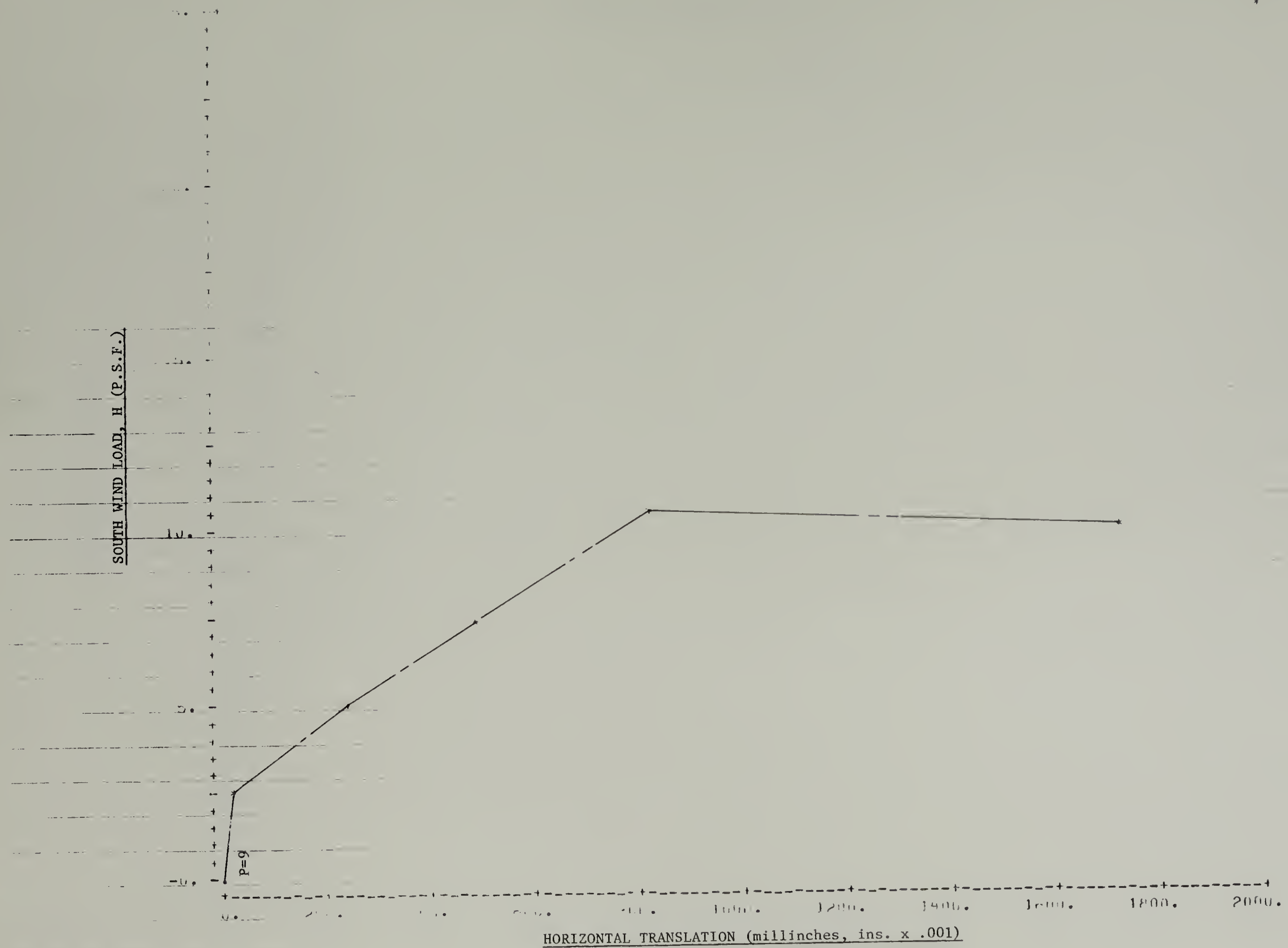


FIGURE NO. 9.13 - TEST NO. 18, SOUTH WIND LOAD VS. TRANSLATION

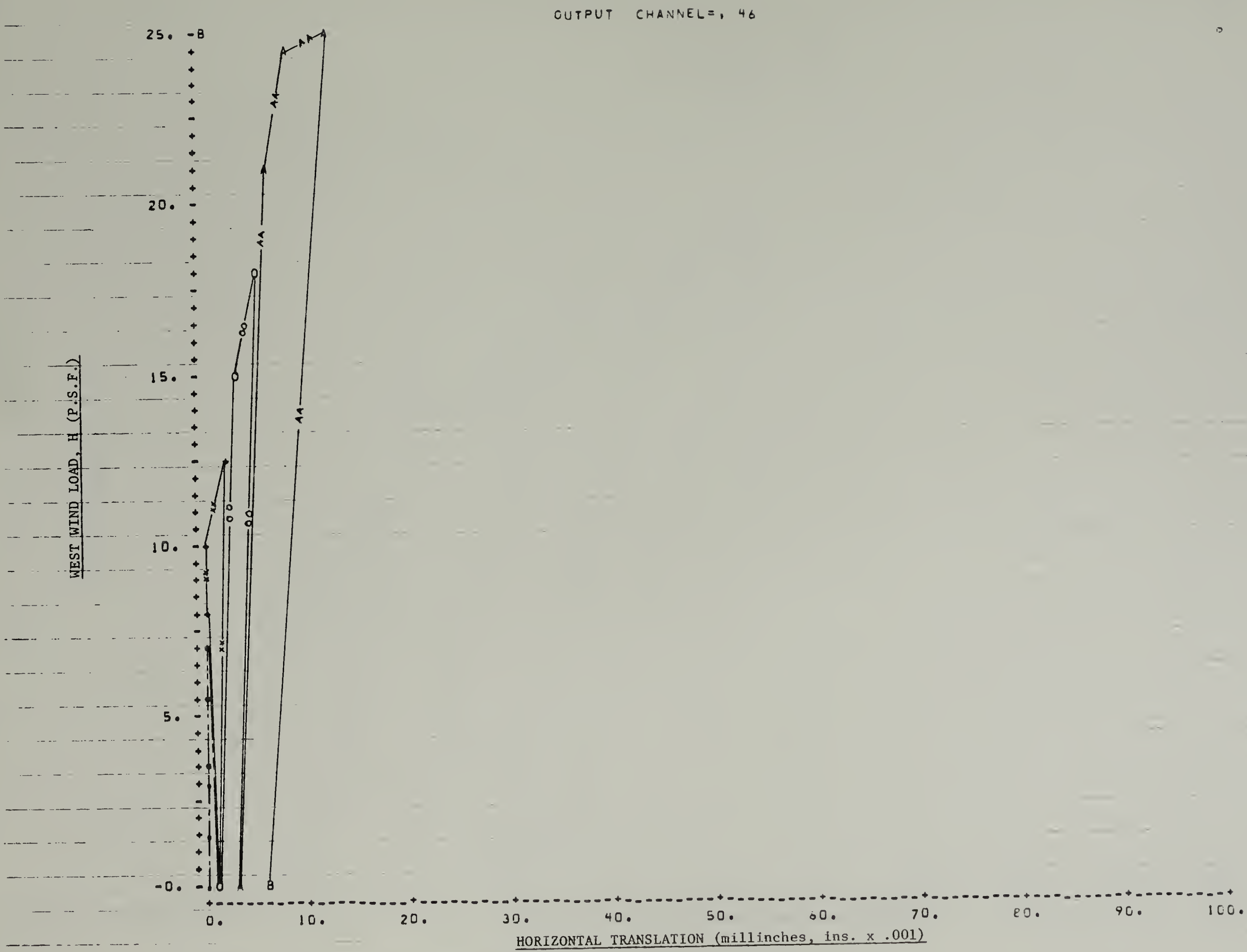


FIGURE NO. 9.14 - TEST NO. 3, WEST WIND LOAD VS. TRANSLATION

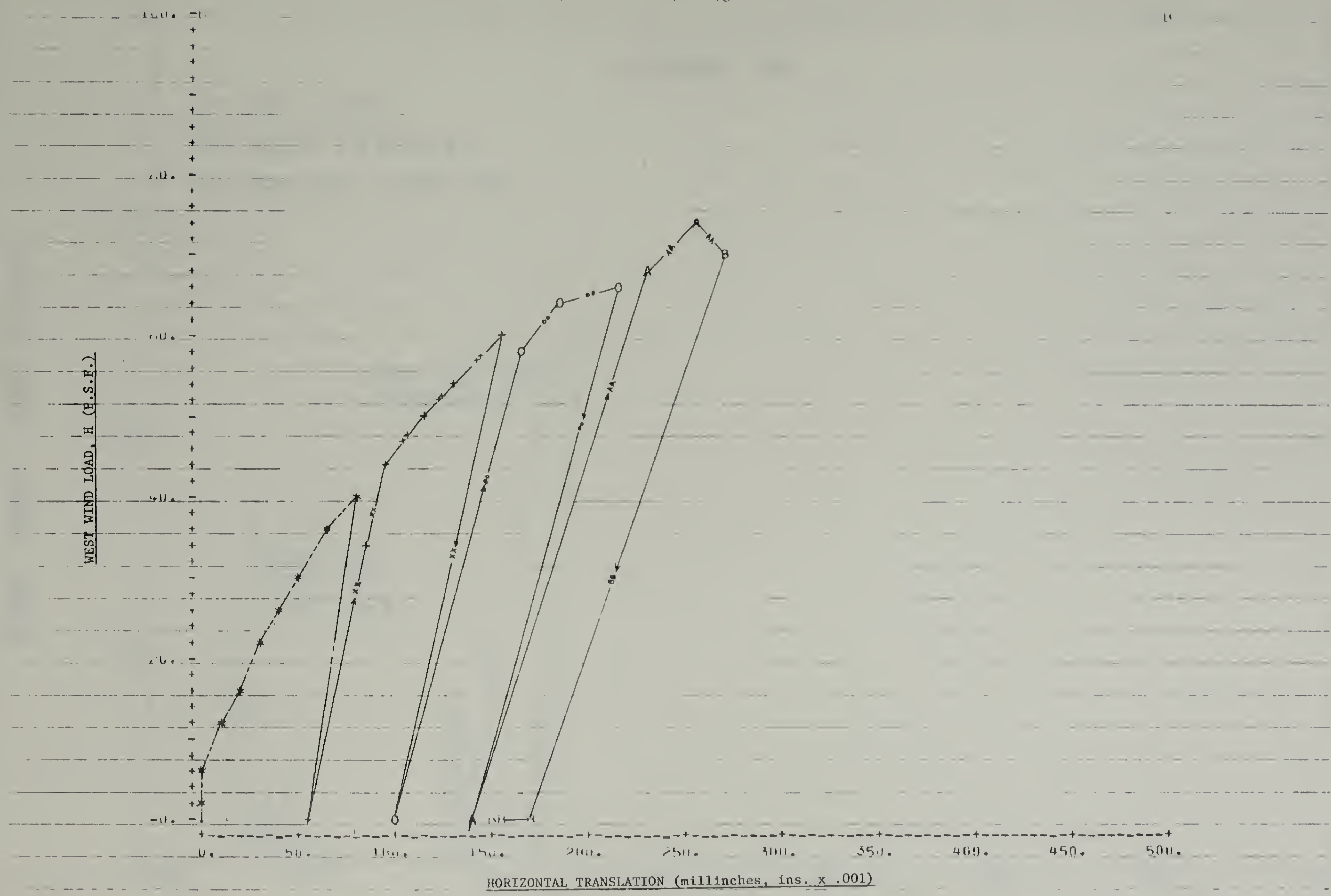


FIGURE NO. 2.15 - TEST NO. 11, WEST WIND LOAD VS. TRANSLATION

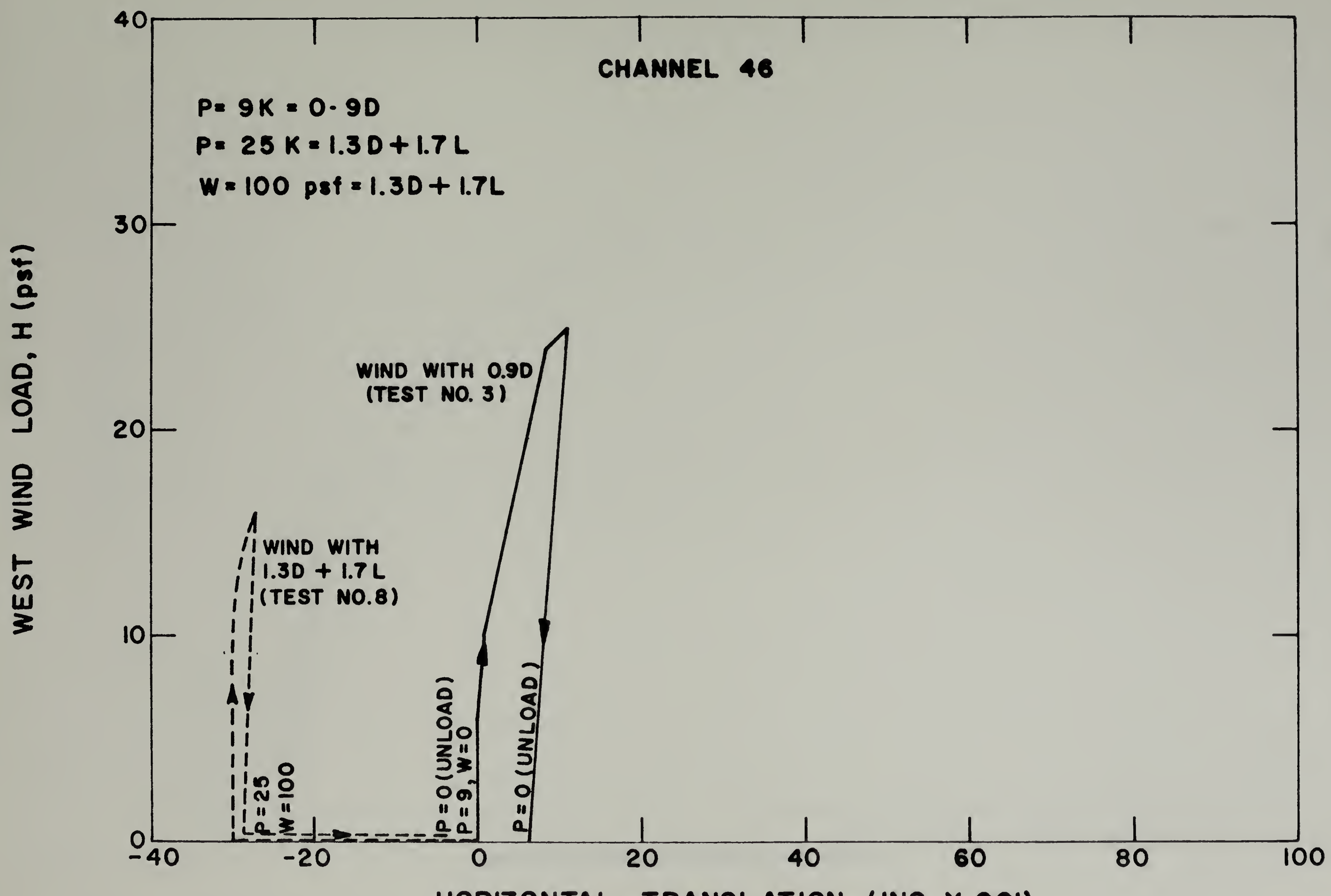


FIGURE NO. 9.16-EFFECT OF VERTICAL LOADS ON E-W HORIZONTAL TRANSLATION

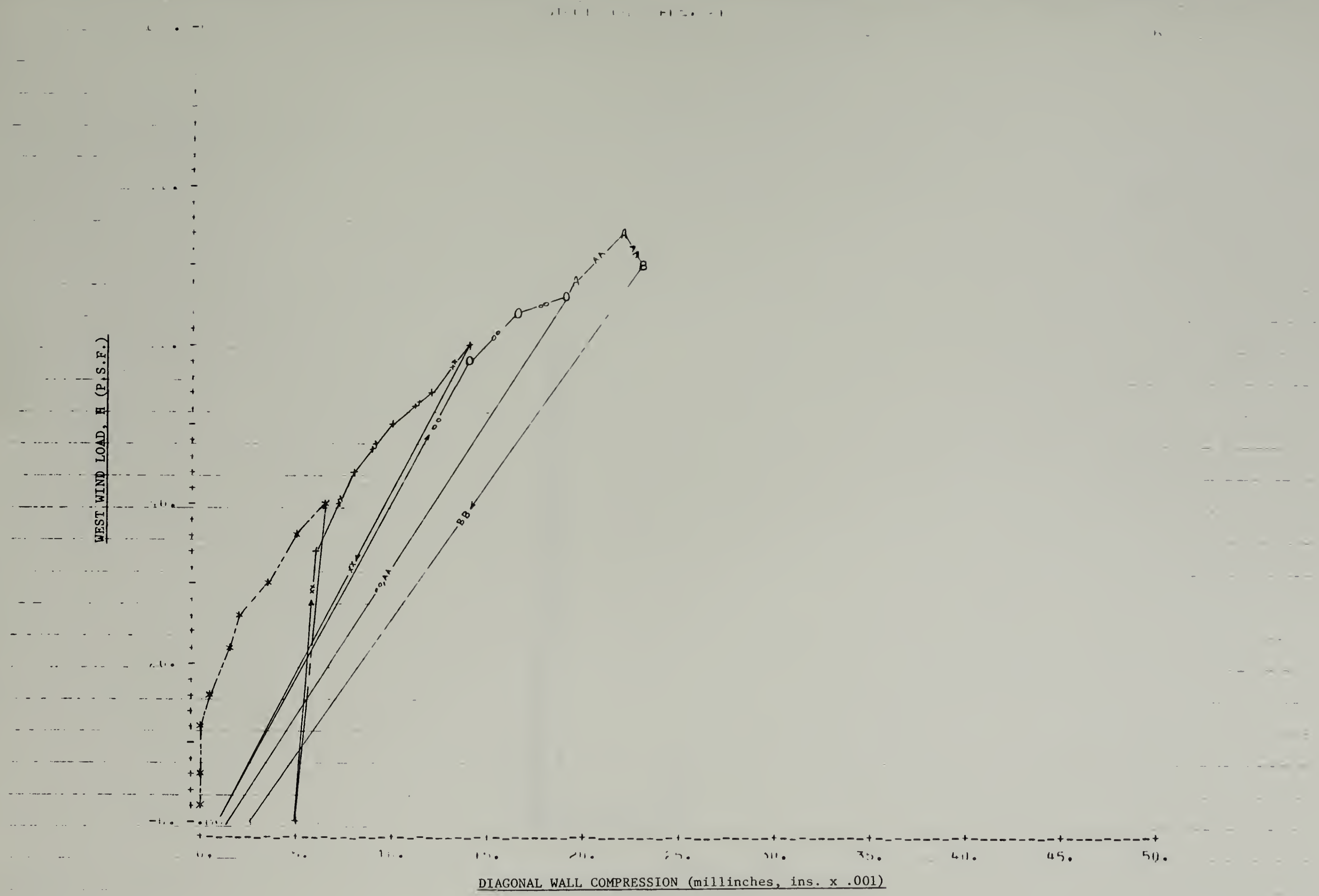


FIGURE NO. 9.17 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

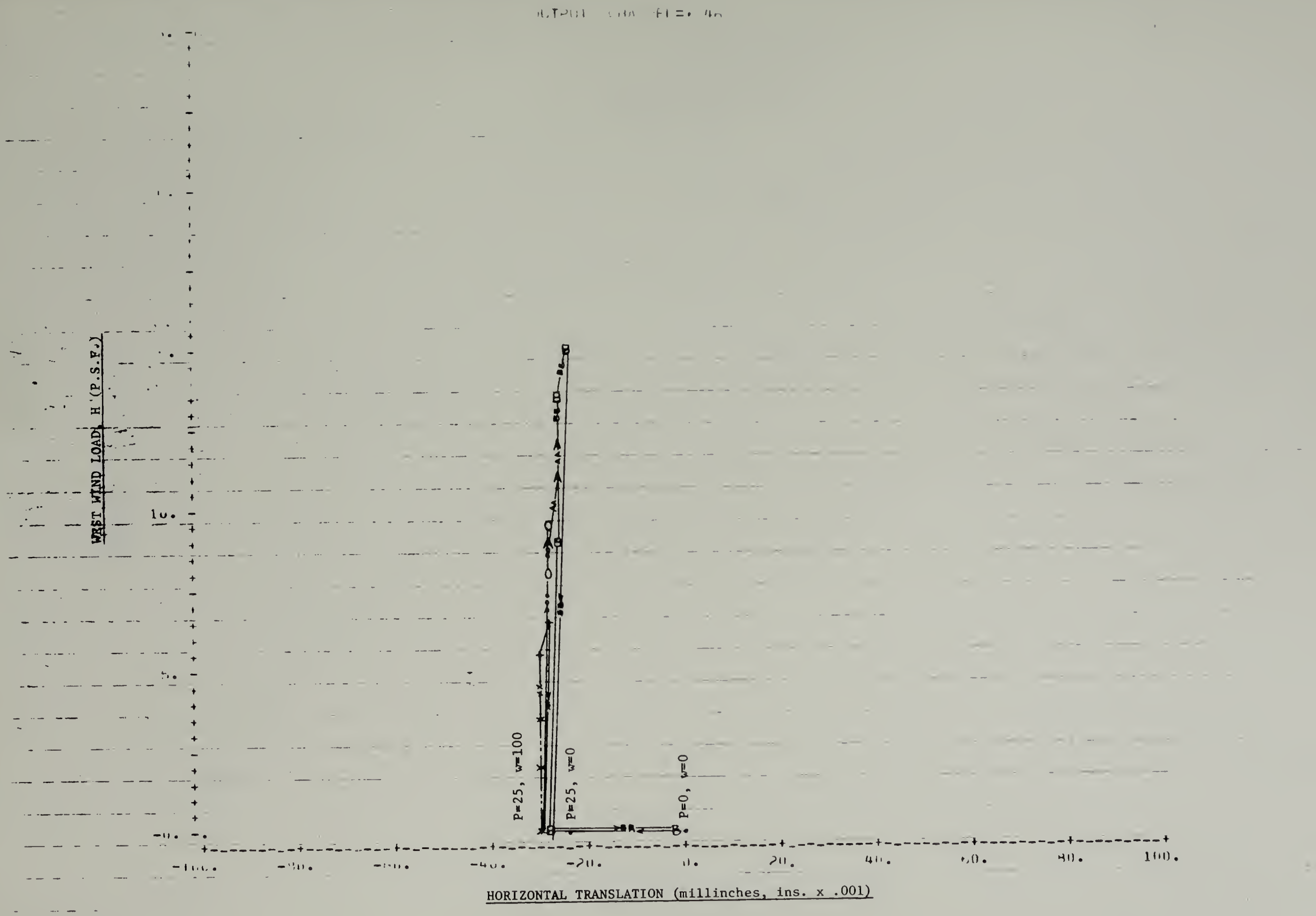


FIGURE NO. 9.18 - TEST NO. 8, WEST WIND LOAD VS. TRANSLATION

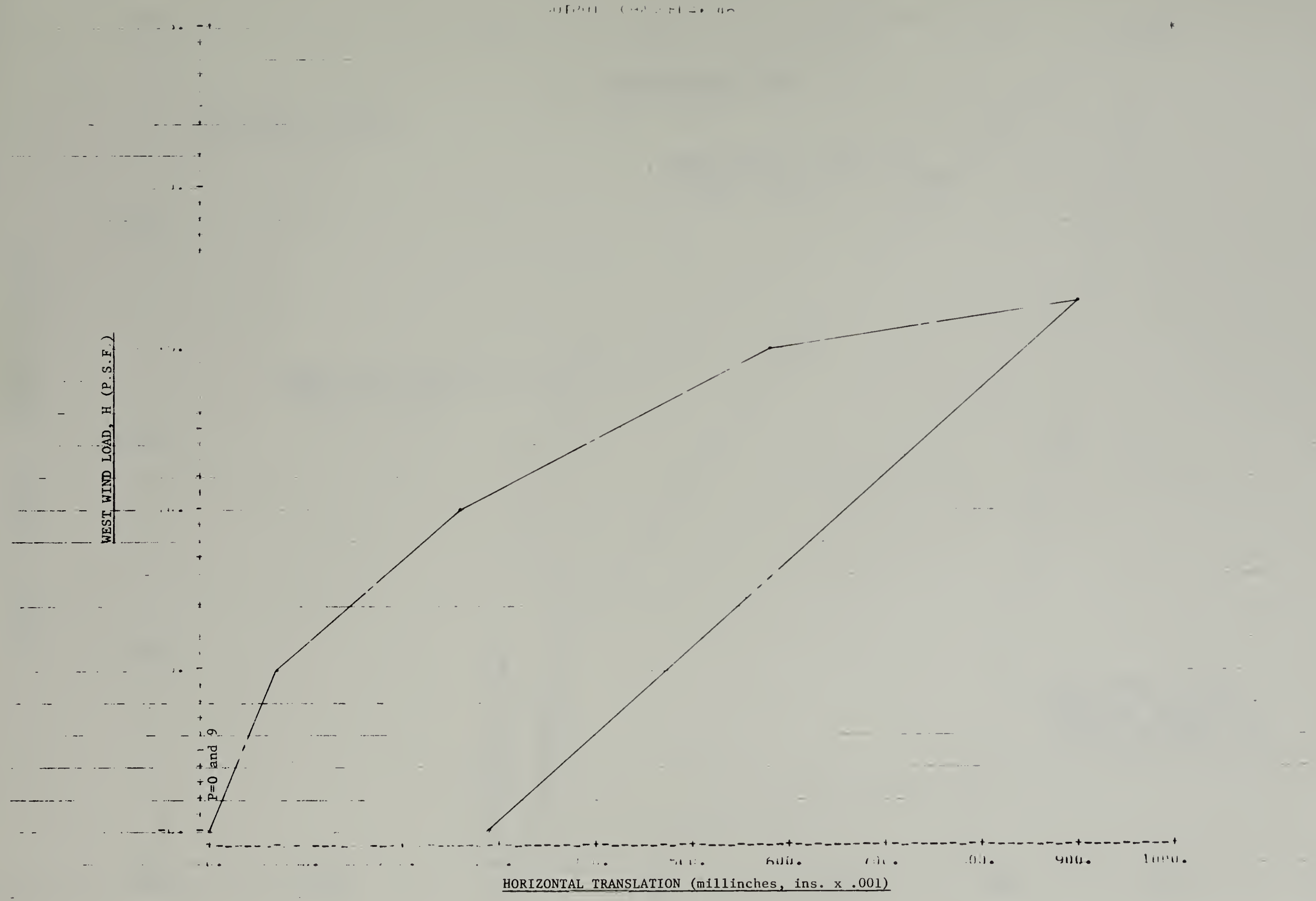


FIGURE NO. 9.19 - TEST NO. 15, WEST WIND LOAD VS. TRANSLATION

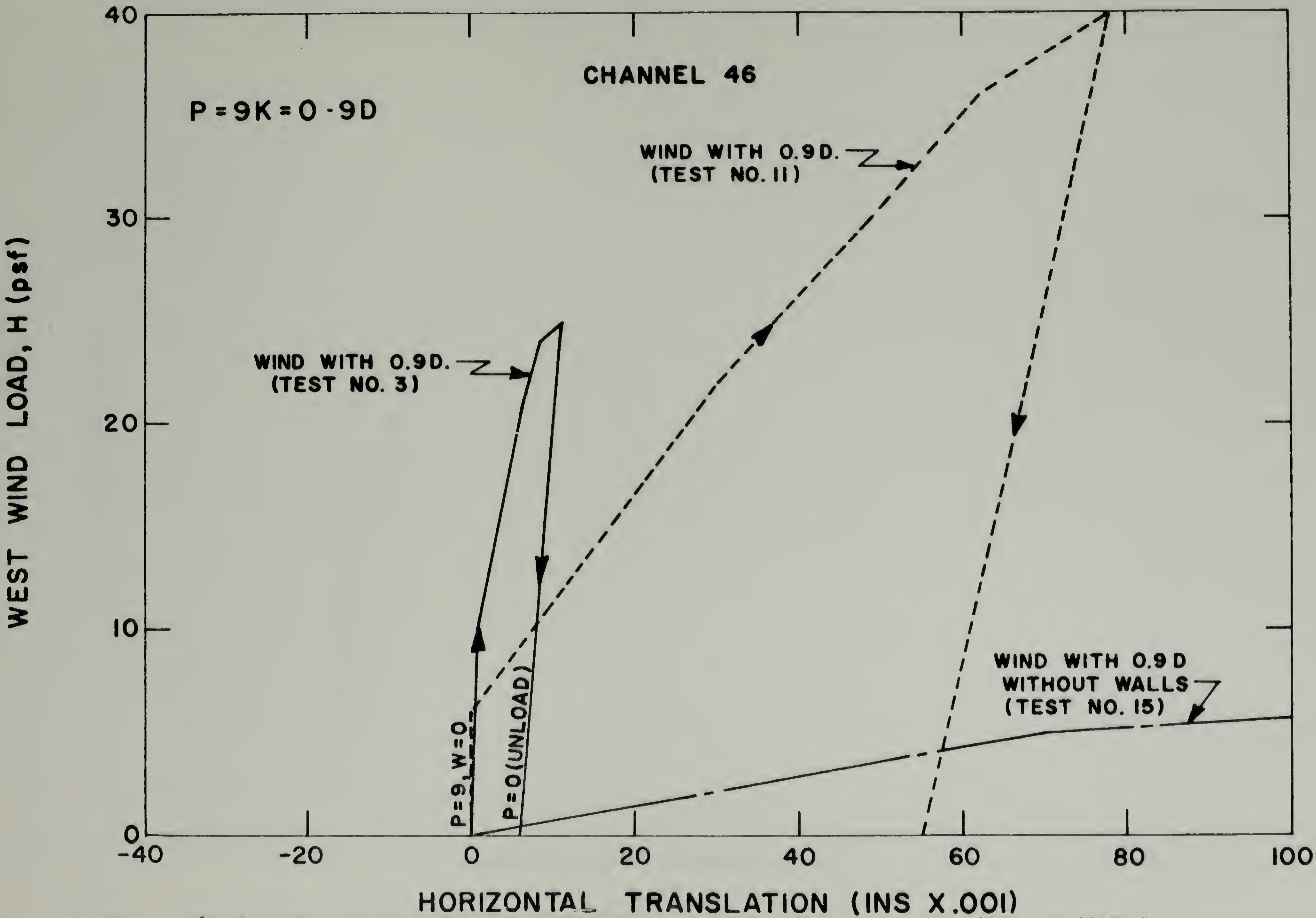


FIGURE NO. 9.20-E-W HORIZONTAL TRANSLATION OF STRUCTURE WITH AND WITHOUT WALLS

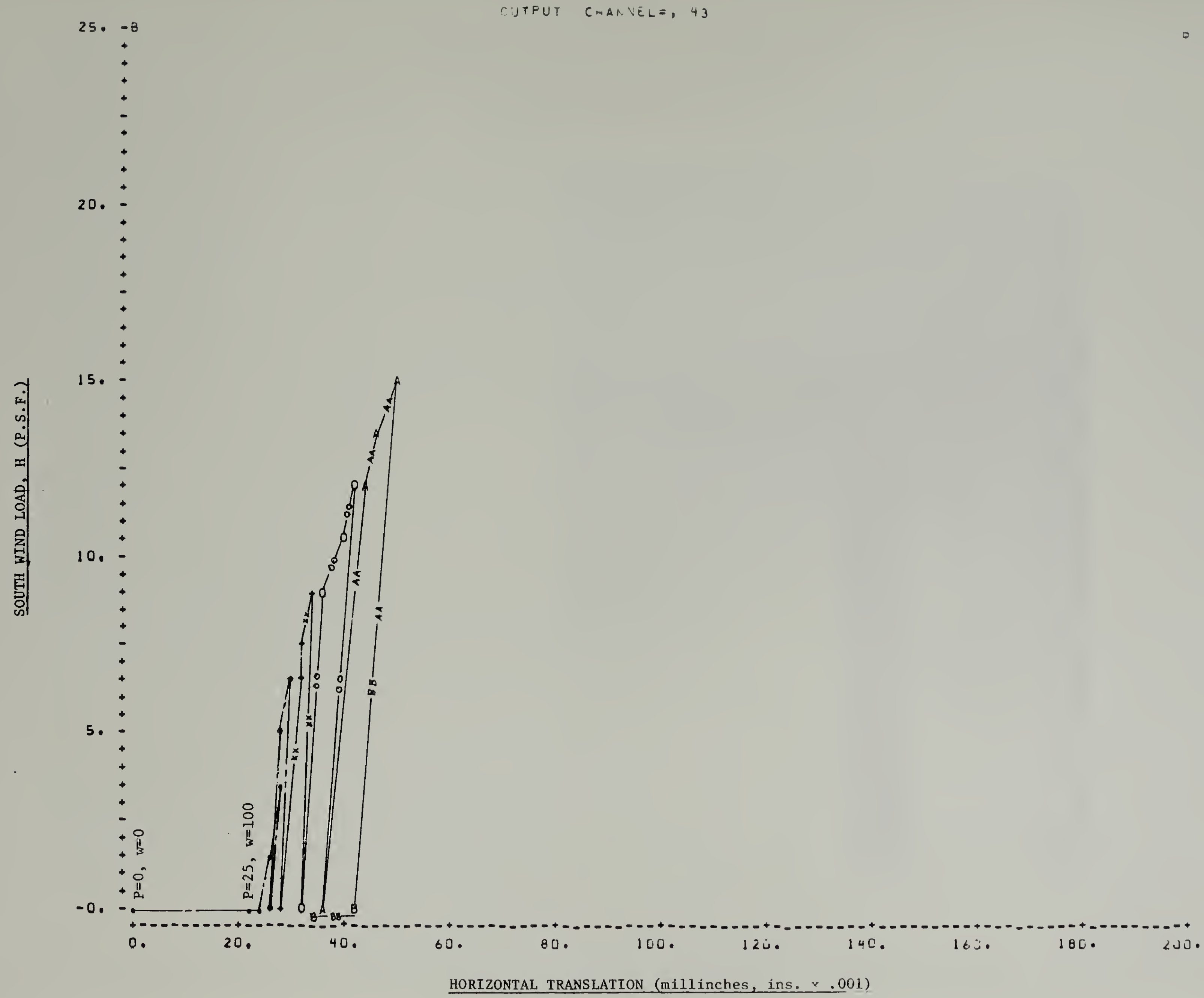


FIGURE NO. 9.21 - TEST NO. 7, SOUTH WIND LOAD VS. TRANSLATION

Fig. 9.22 Drywall crack near ceiling on interior side of east wall

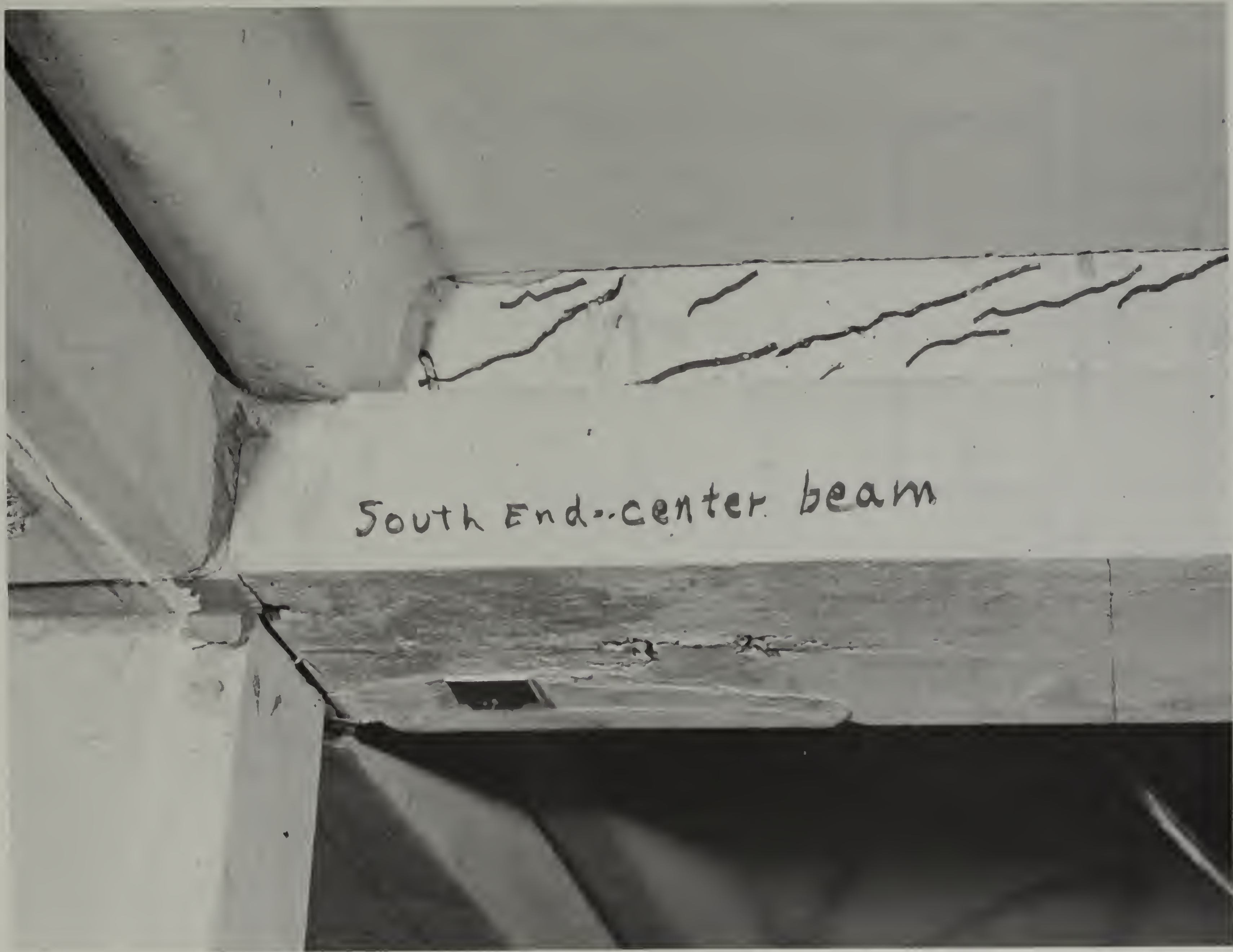


Fig. 9.23 Diagonel tension cracking at W=270 PST
(test No. 16)

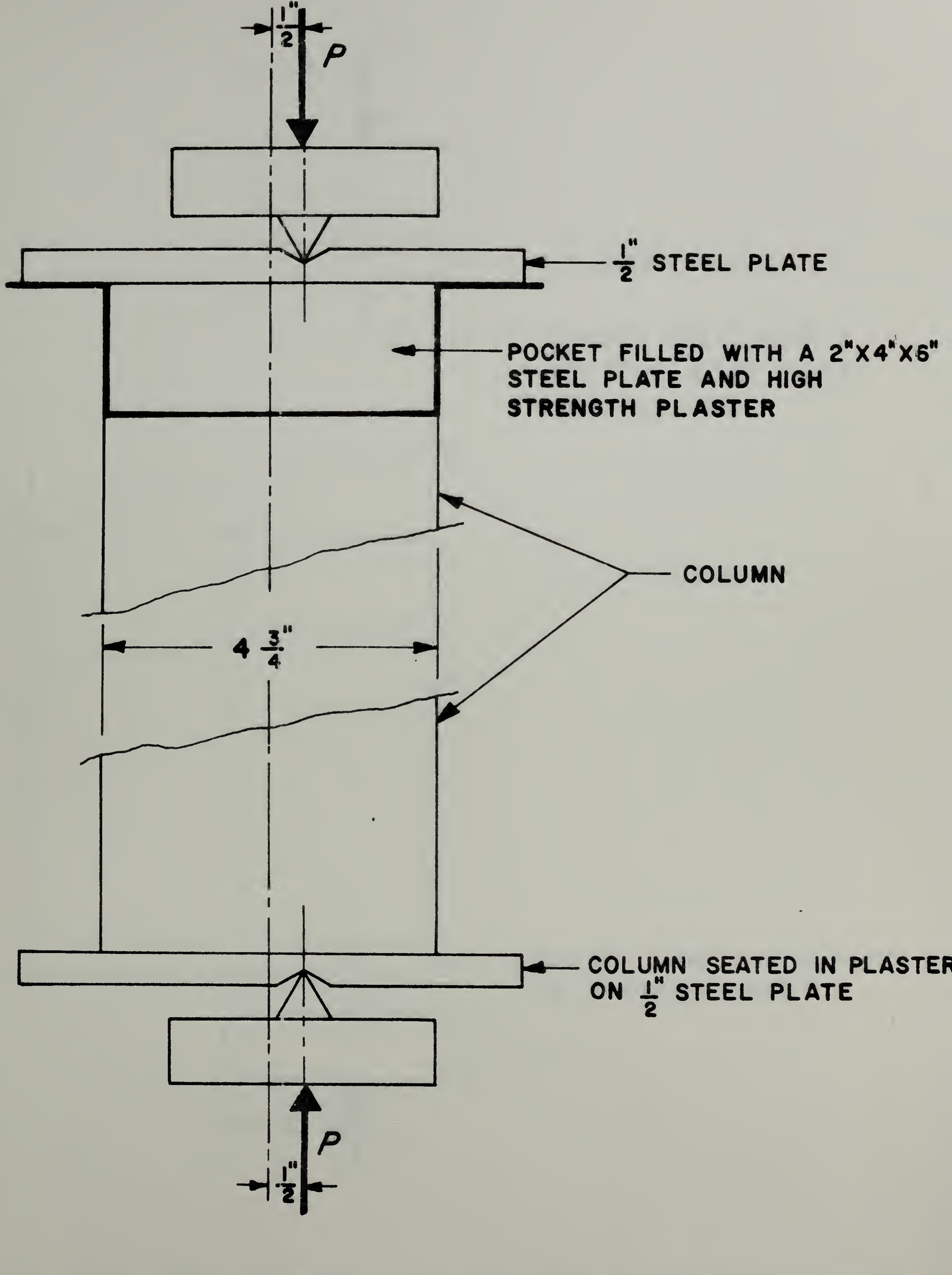


FIG. 10.1 TEST METHOD FOR COLUMN TESTS (Minor Axis)

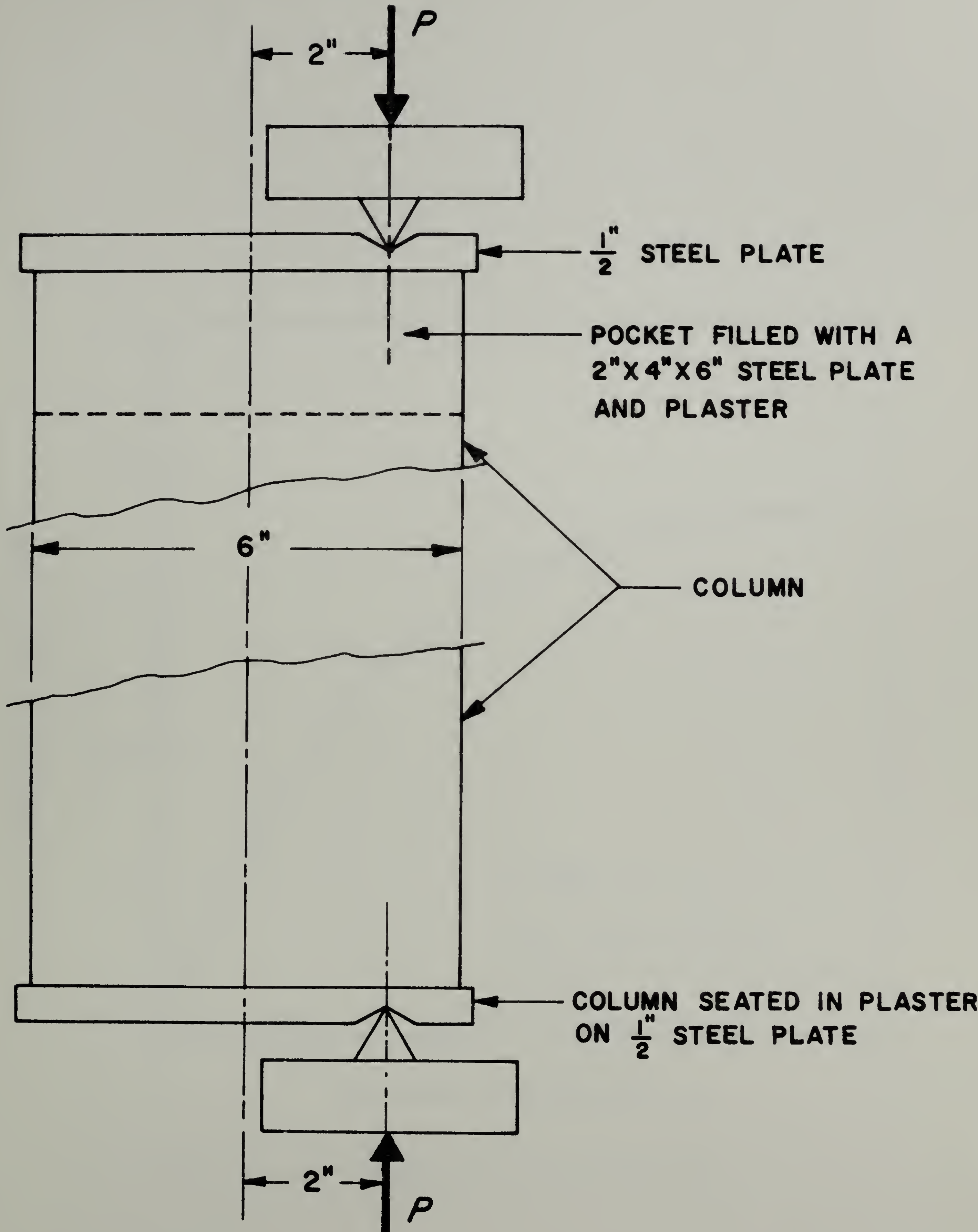


FIG. 10.2 TEST METHOD FOR COLUMN TESTS (Major Axis)

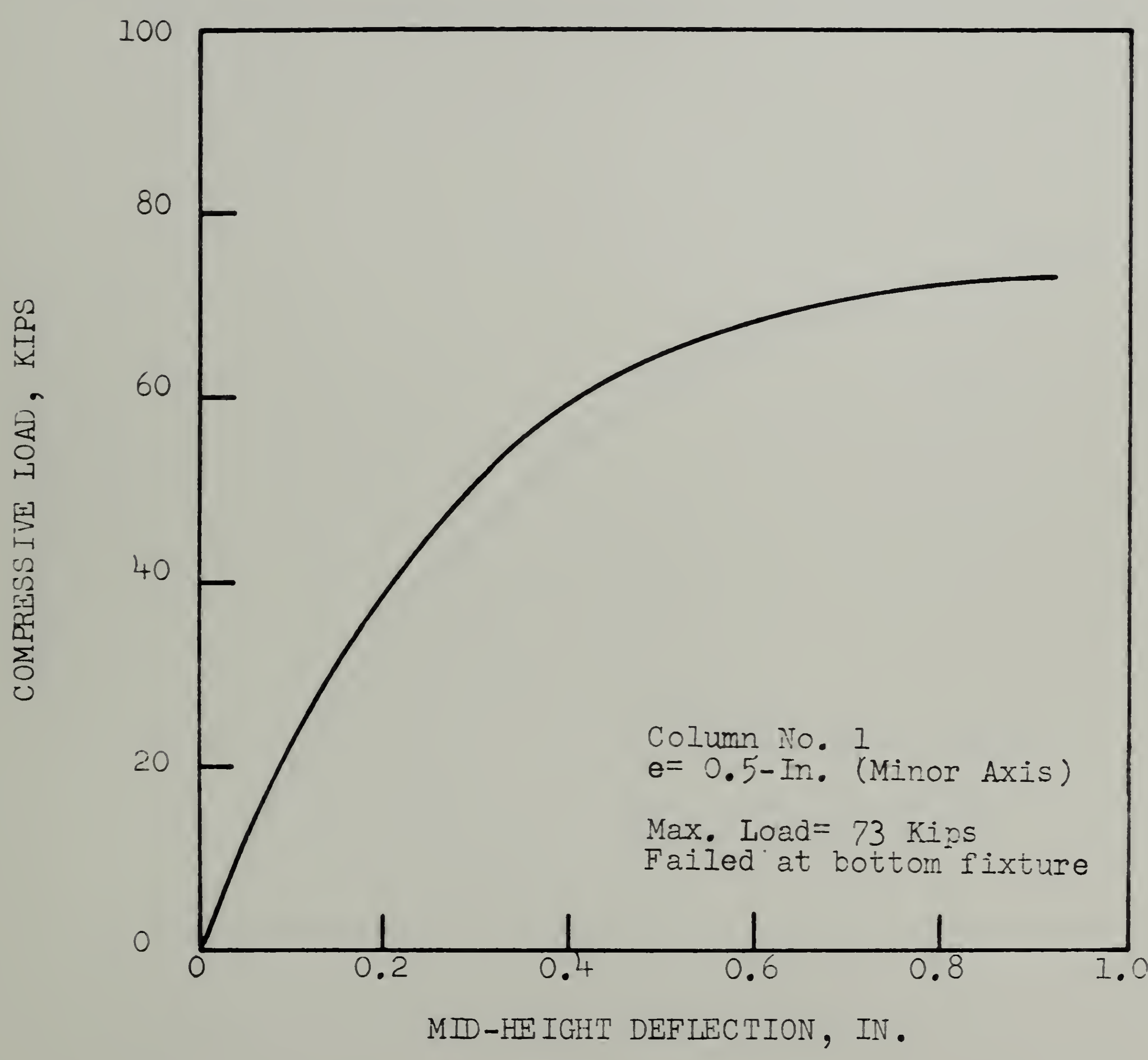


FIG. 10.3 SHORT-TERM TEST ON COLUMN NO. 1

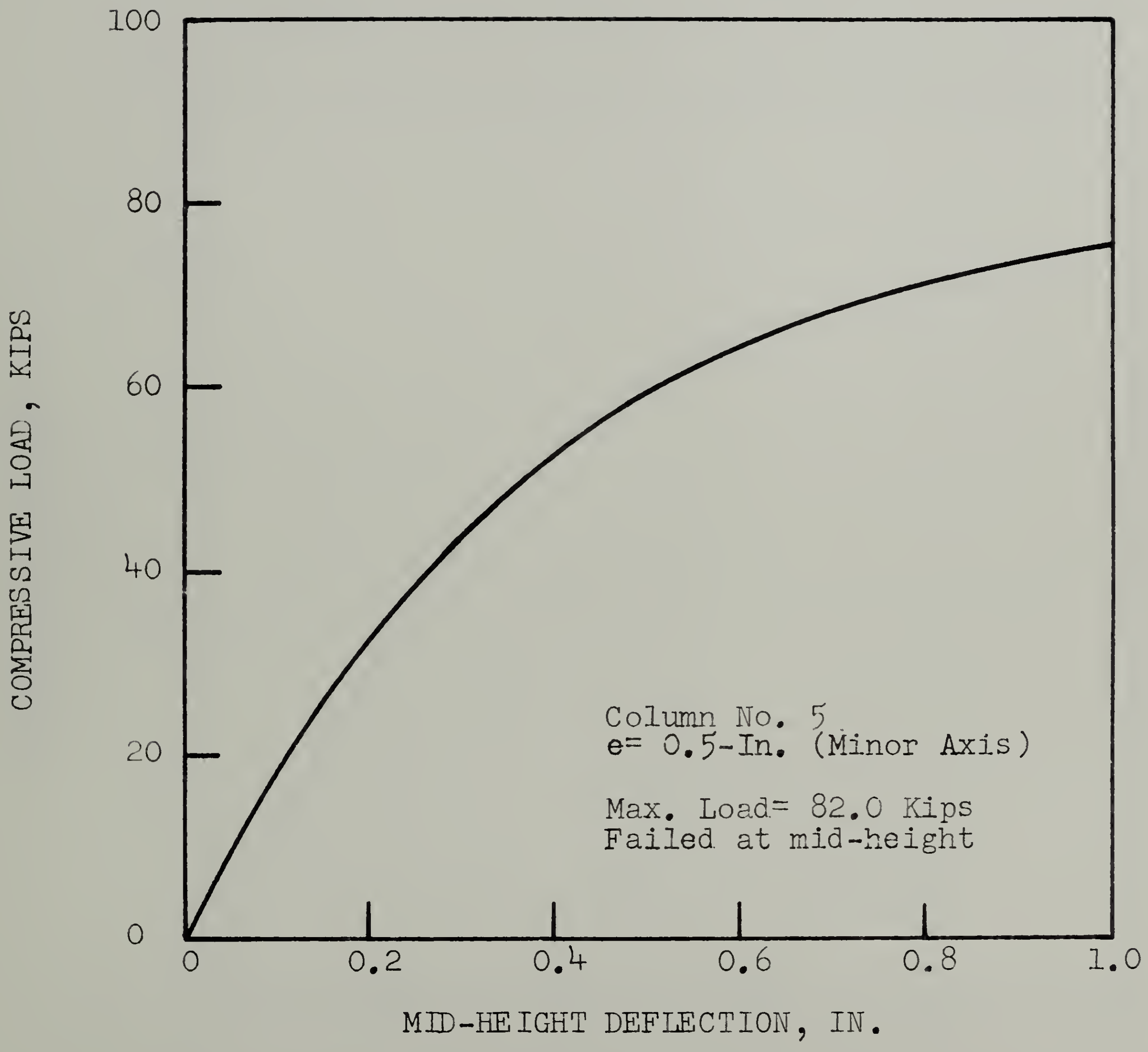


FIG. 10.4 SHORT-TERM TEST ON COLUMN NO. 5

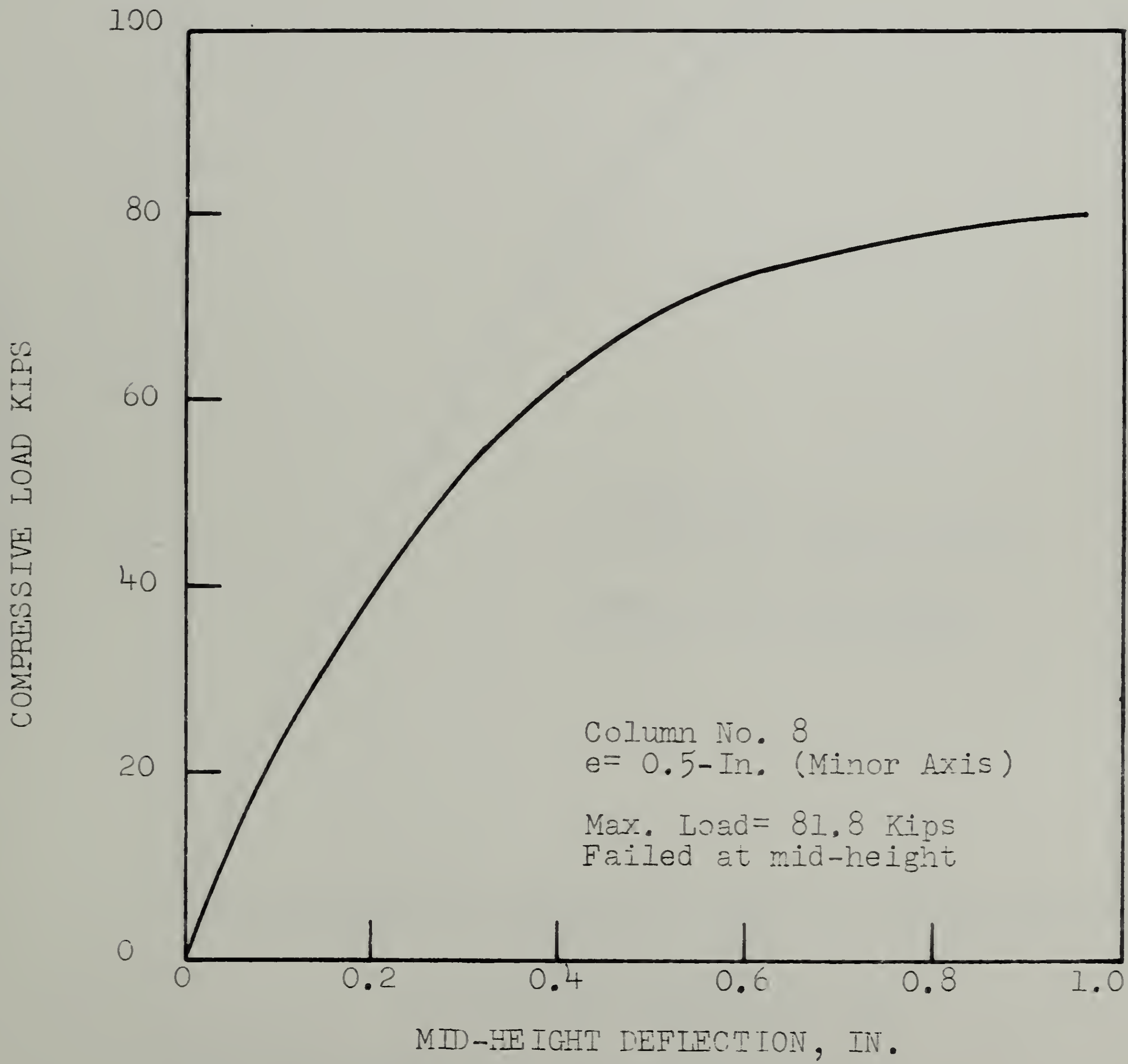


FIG. 10.5 SHORT-TERM TEST ON COLUMN NO. 8

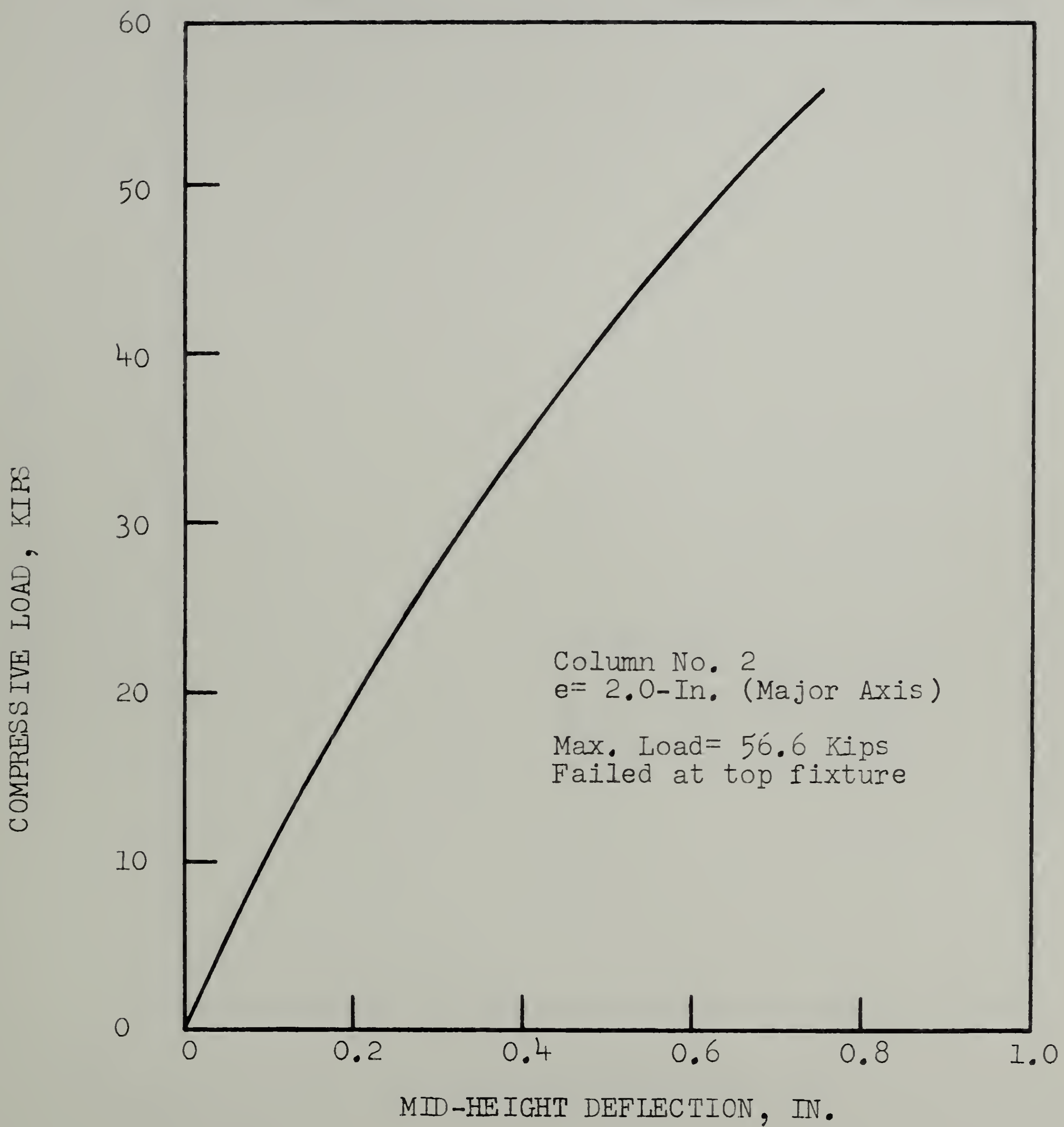


FIG. 10.6 SHORT-TERM TEST ON COLUMN NO. 2

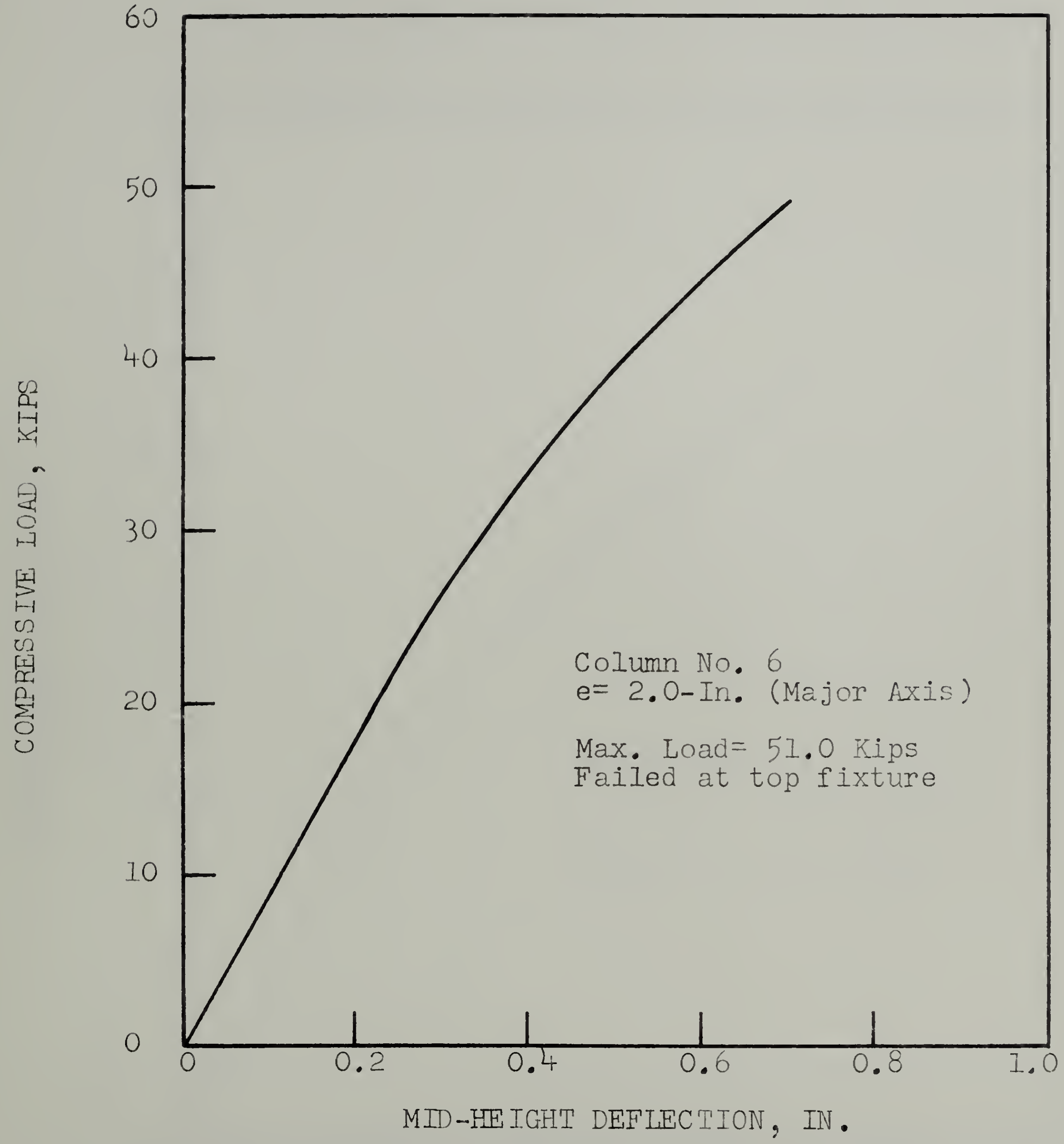


FIG. 10.7 SHORT-TERM TEST ON COLUMN NO. 6

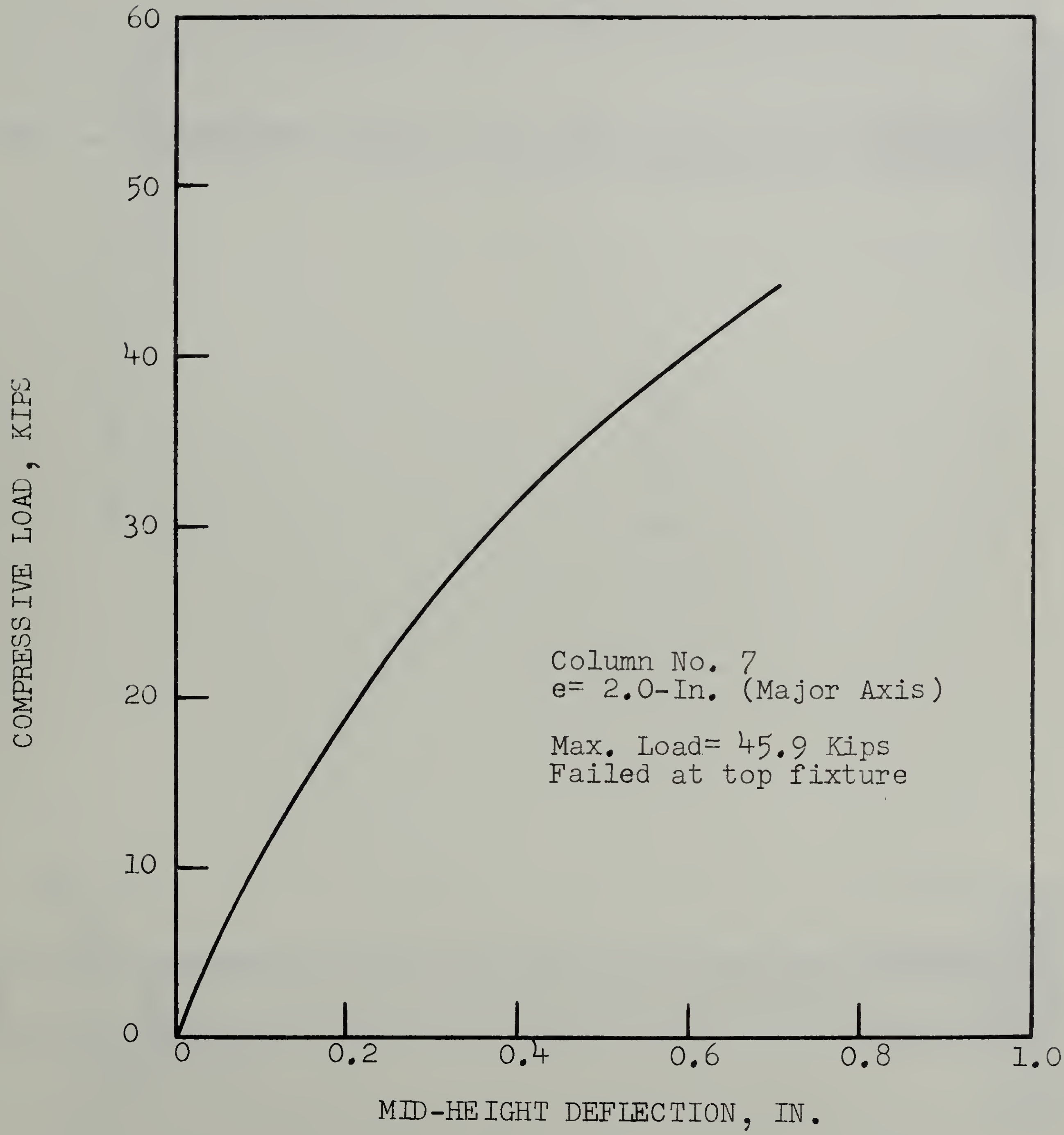


FIG. 10.8 SHORT-TERM TEST ON COLUMN NO. 7

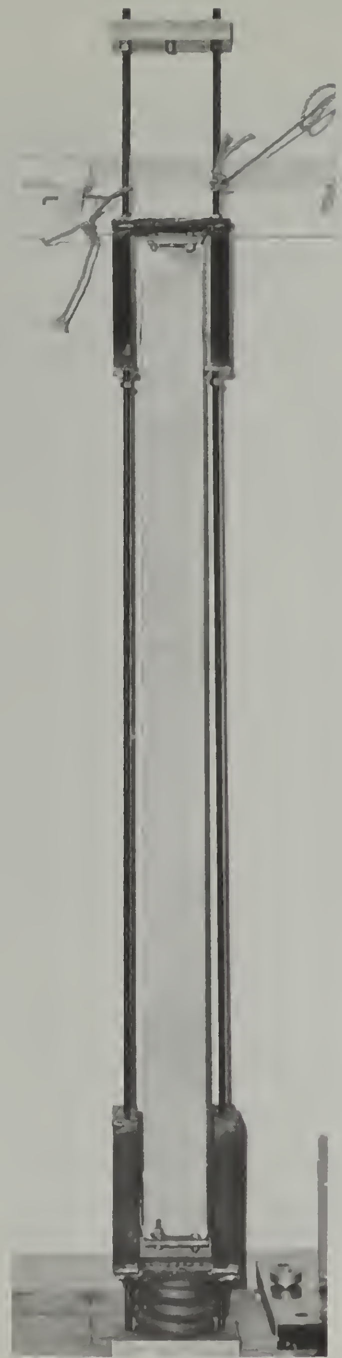


Fig. 10.9 Creep loading frame for column No. 3 minor axis bending

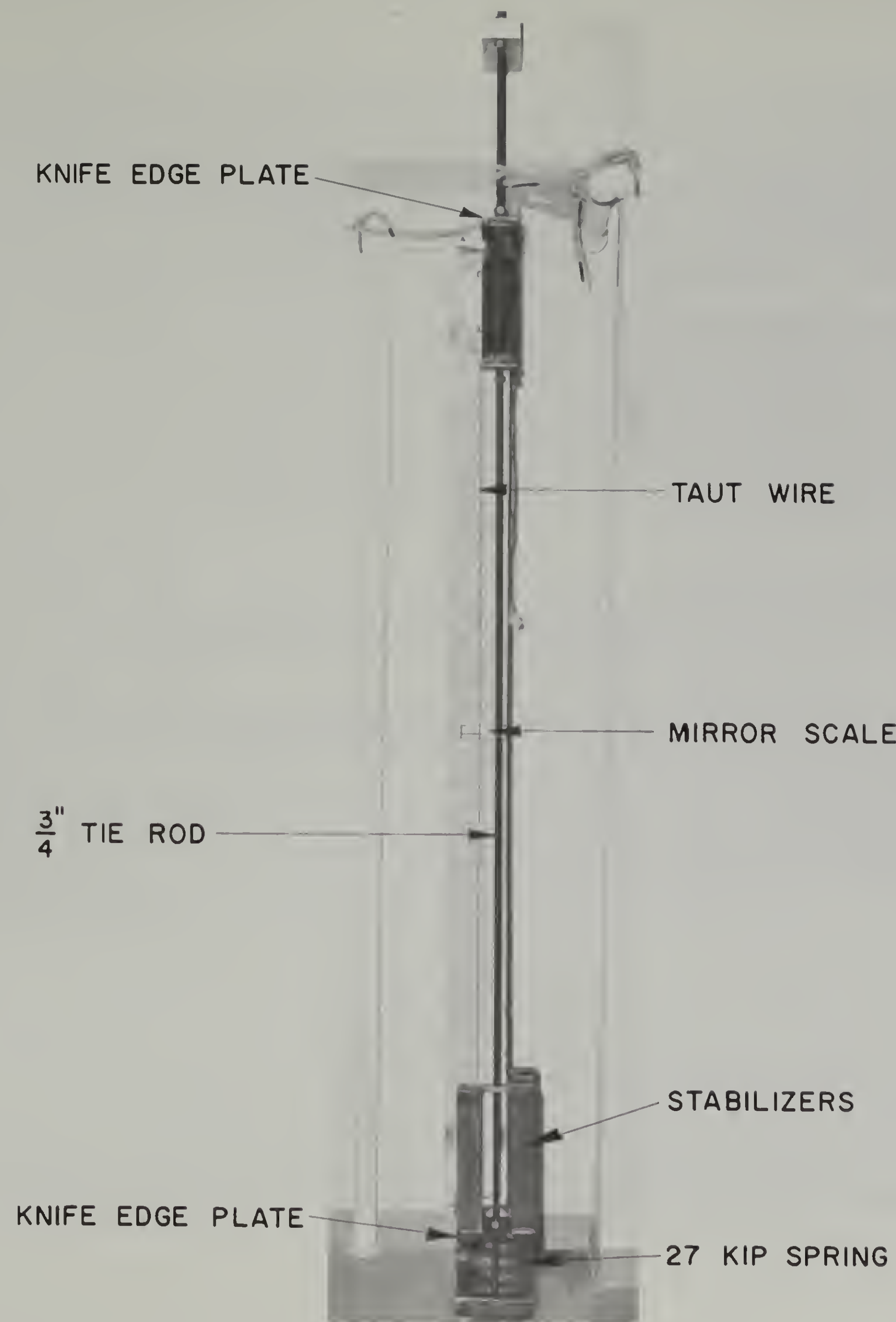


Fig. 10.10 Creep loading frame for column No. 4 major axis bending

Fig. 10.11 Detail of base for column No. 4 creep frame

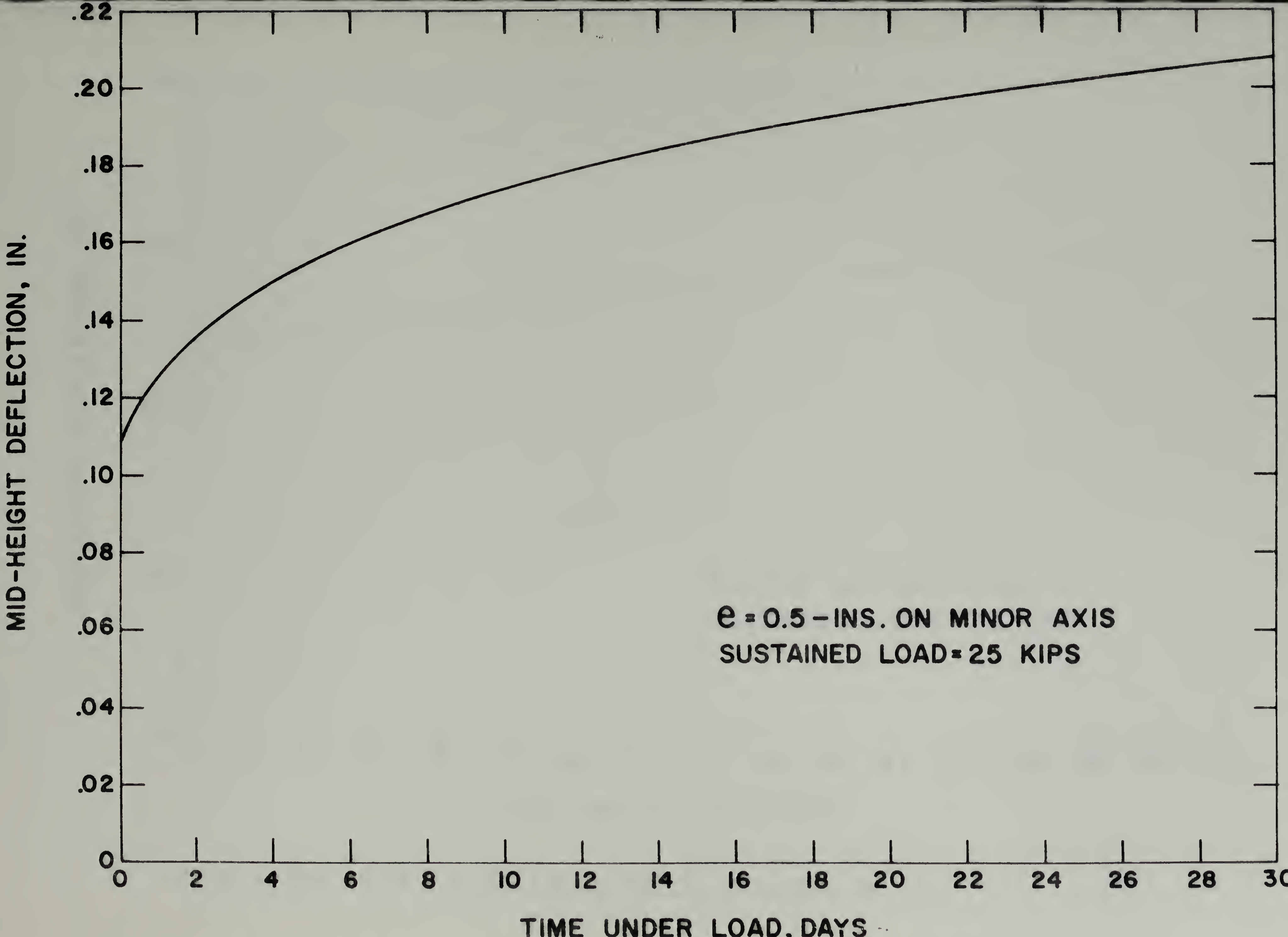


FIG. 10.12 - SUSTAINED LOAD TEST, COLUMN NO. 3

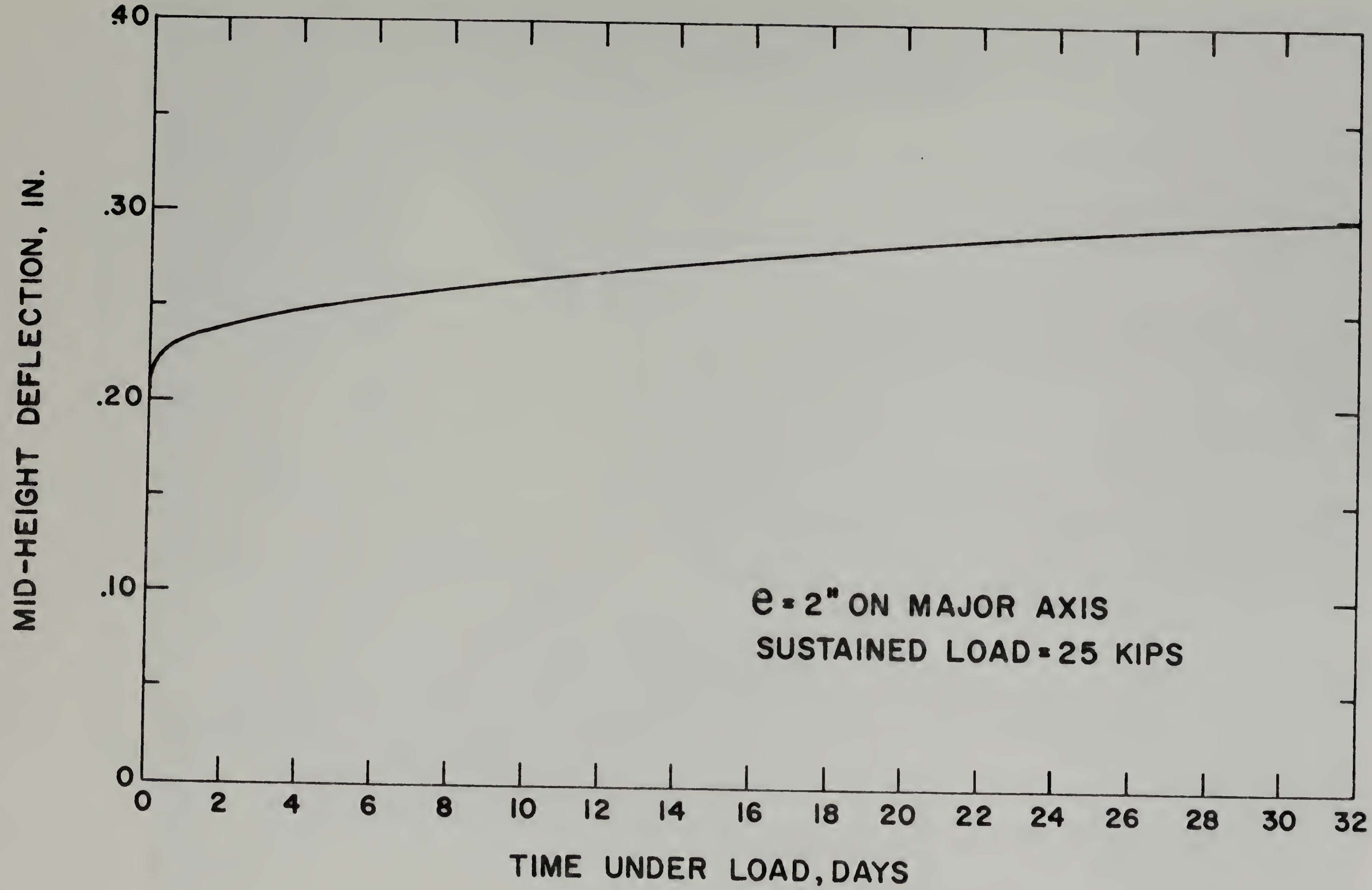


FIG. 10.13 - SUSTAINED LOAD TEST, COLUMN NO. 4

Fig. 10.14 Fatigue test on main beam

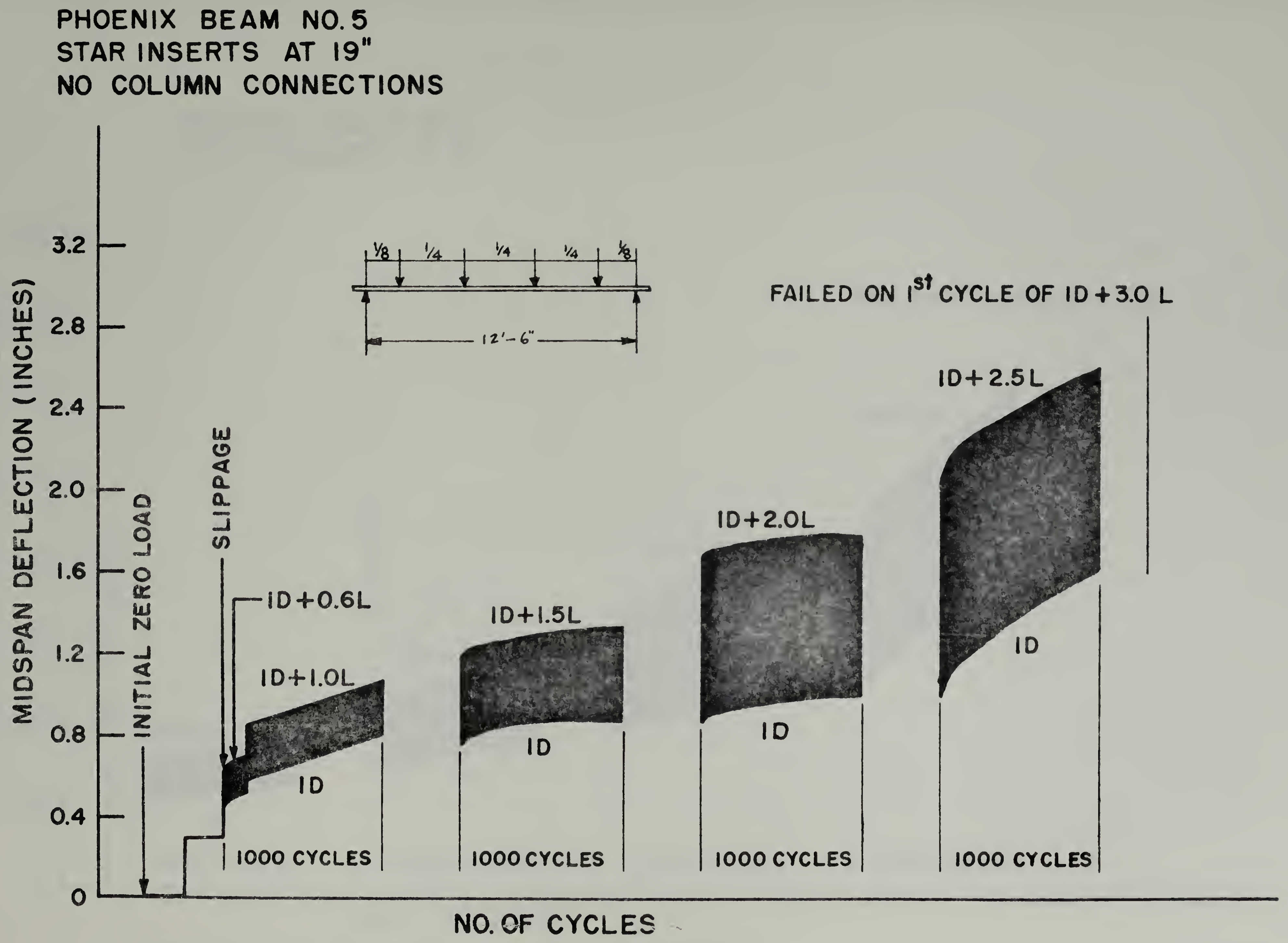


FIGURE 10.15 FATIGUE LOADING, BEAM NO. 5

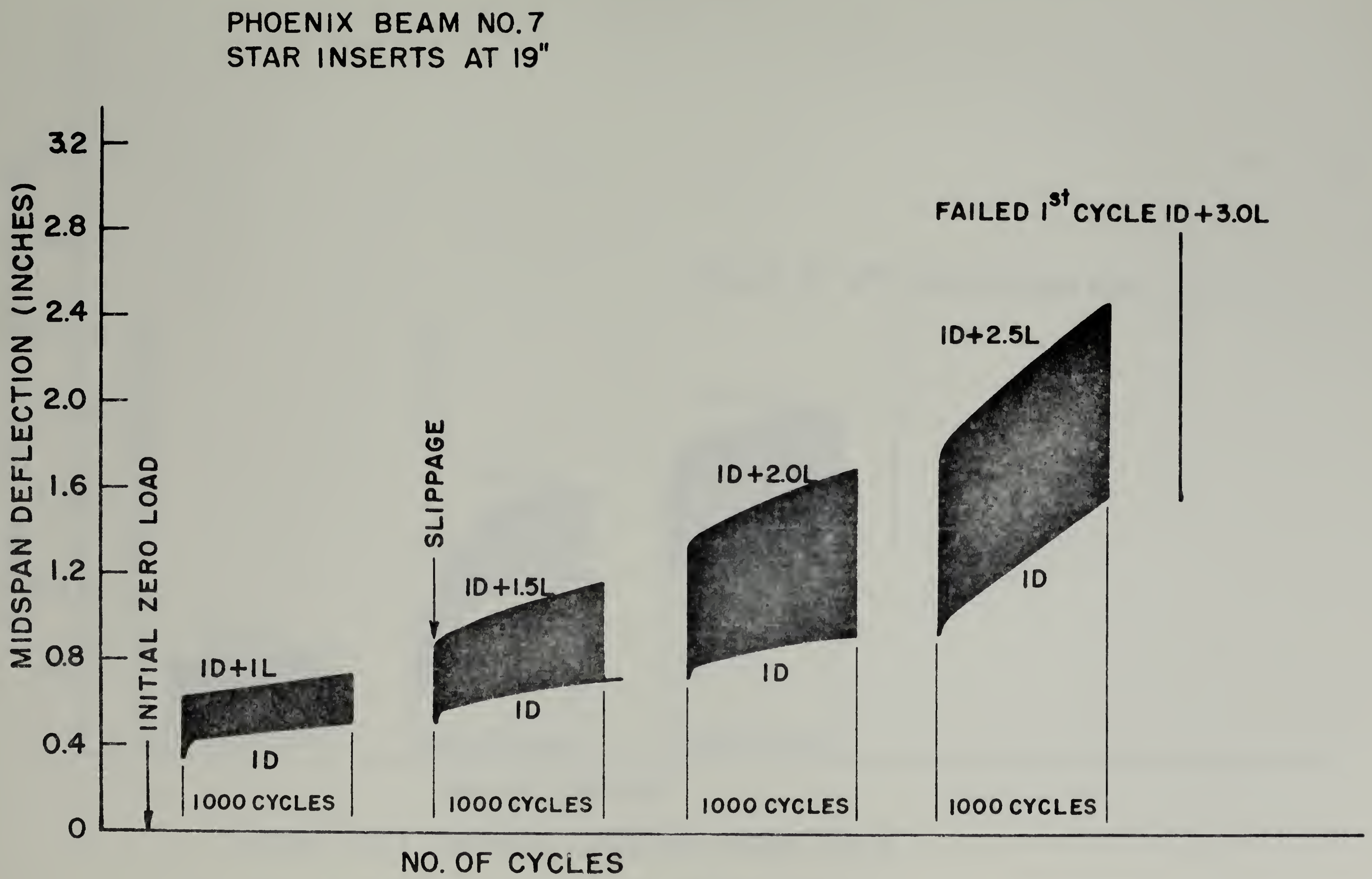


FIGURE 10.16 FATIGUE LOADING, BEAM NO. 7

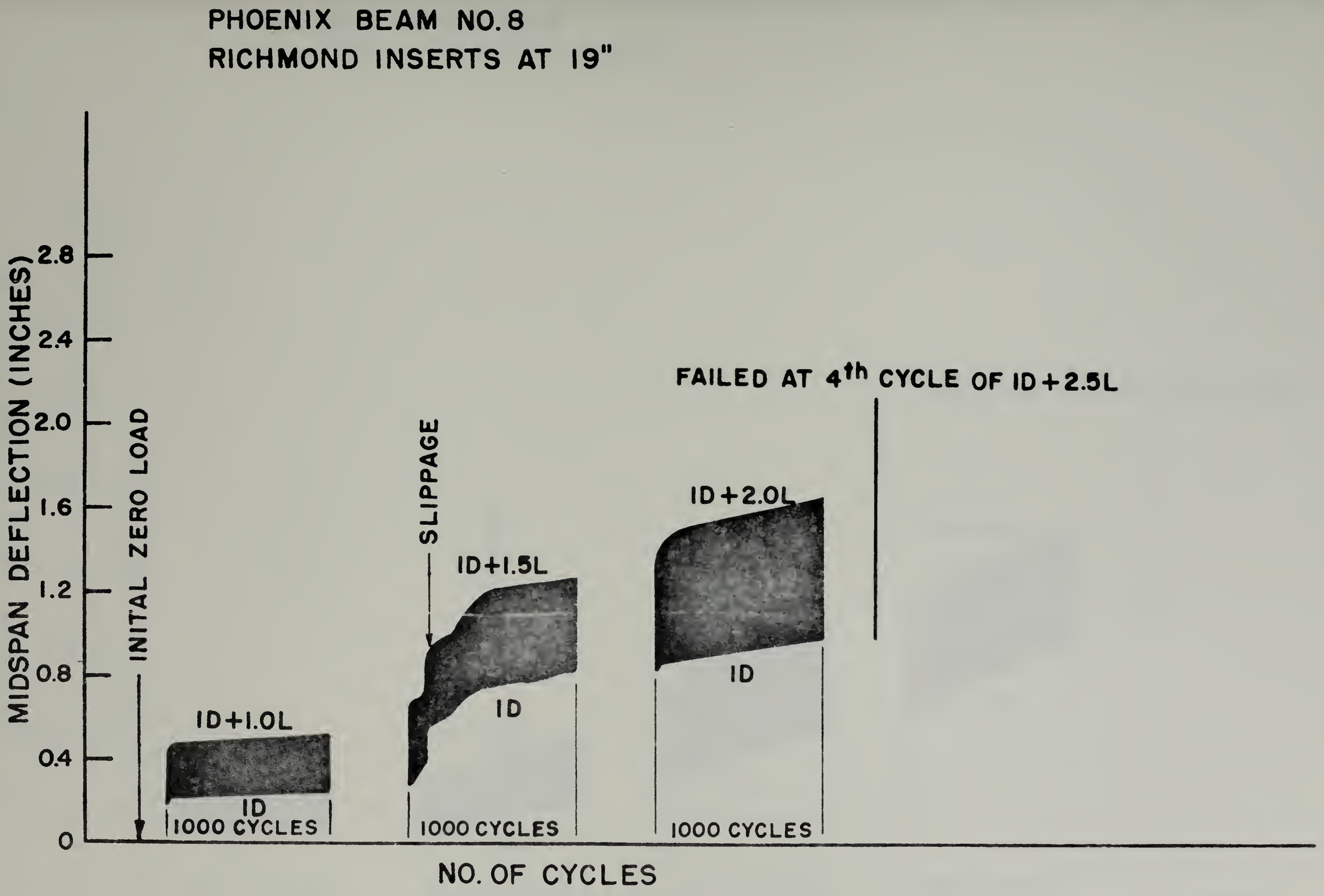


FIGURE 10.17 FATIGUE LOADING, BEAM NO. 8

PHOENIX BEAM NO.6
RICHMOND INSERTS AT 9½"

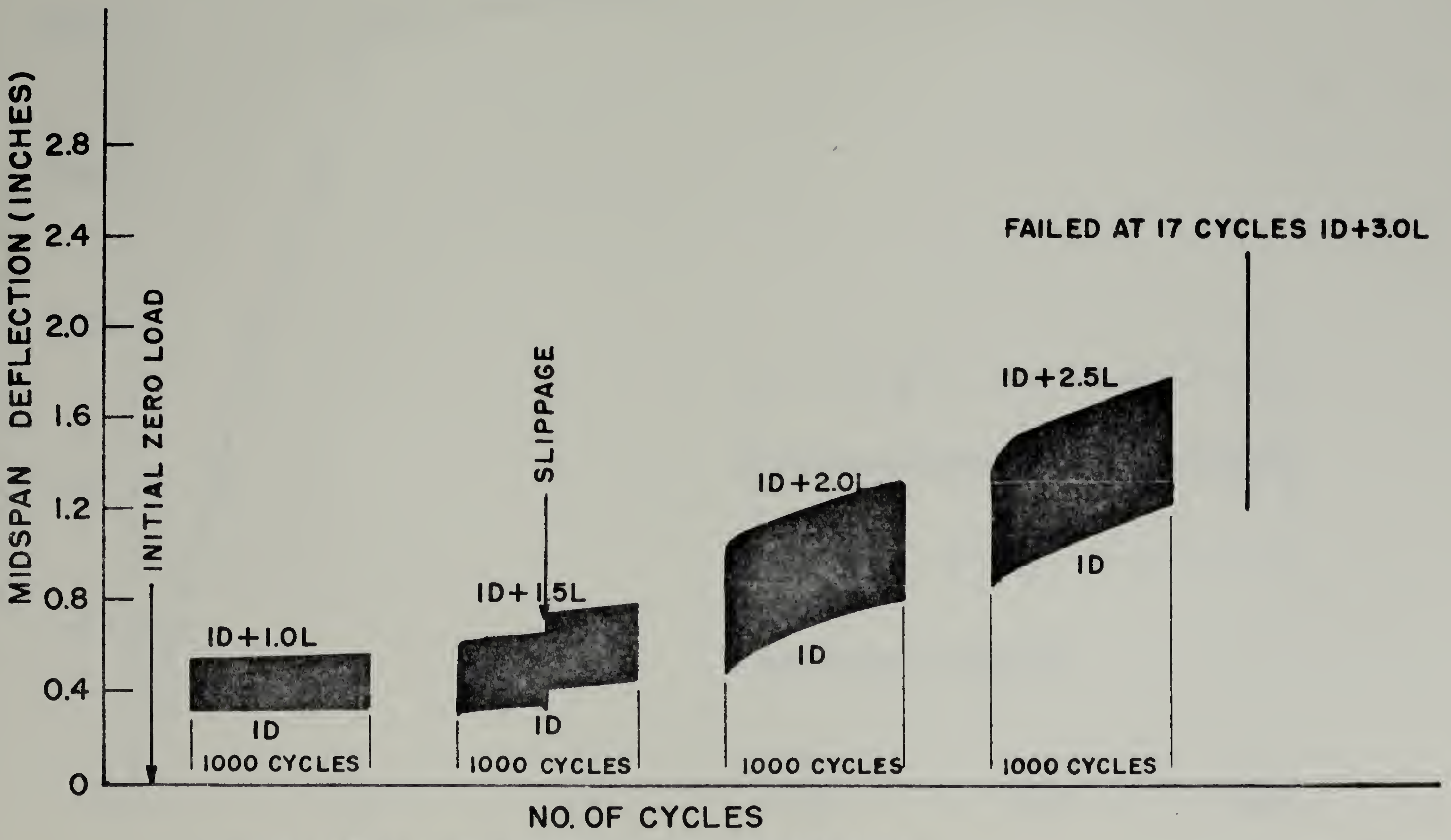


FIGURE 10.18 FATIGUE LOADING, BEAM NO. 6

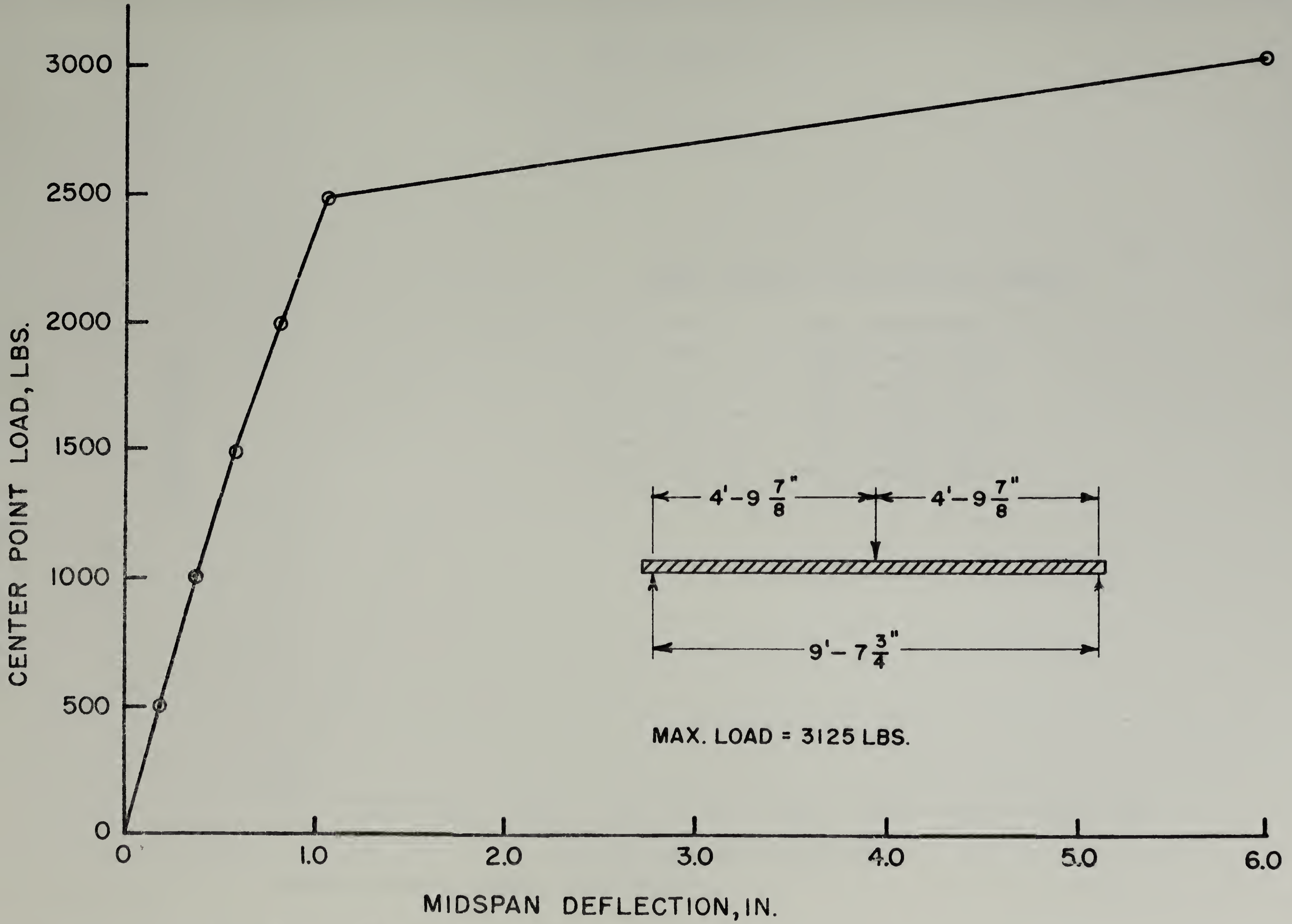


FIGURE 10.19 CHANNEL ROOF SLAB TEST

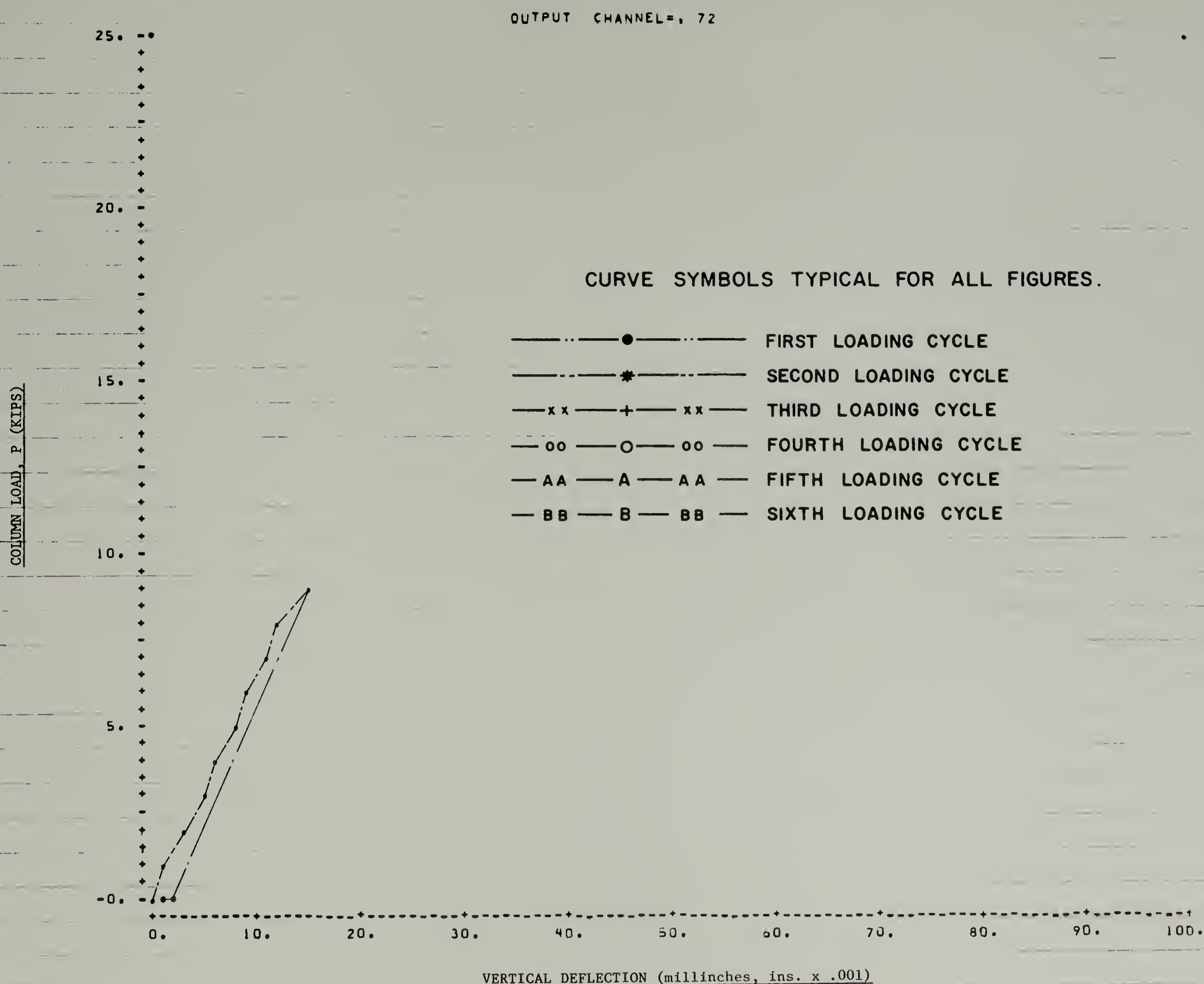


FIGURE NO. A.1 - TEST NO. 1, COLUMN LOAD VS. BEAM DEFLECTION

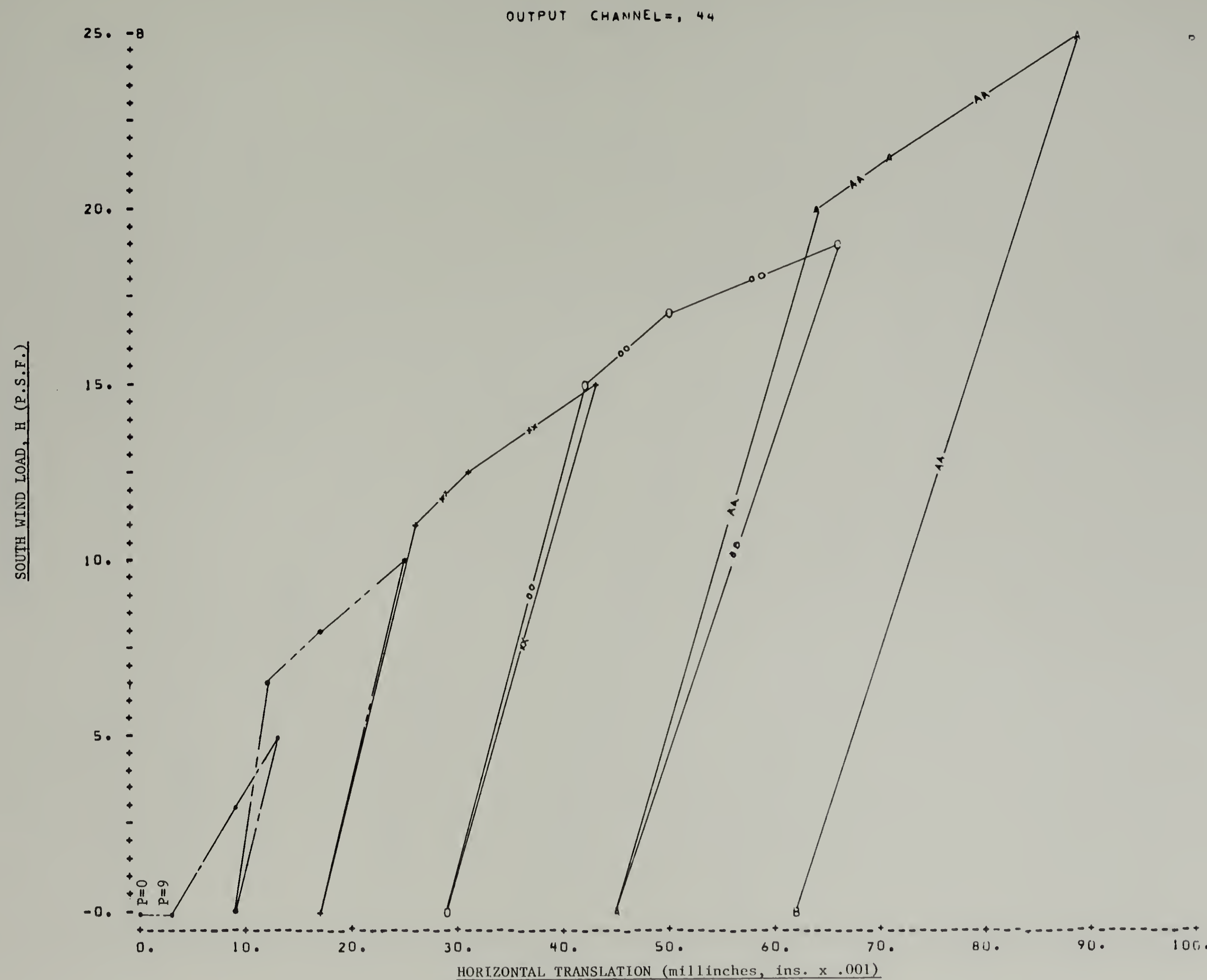


FIGURE NO. A.3 - TEST NO. 2, SOUTH WIND LOAD VS. TRANSLATION

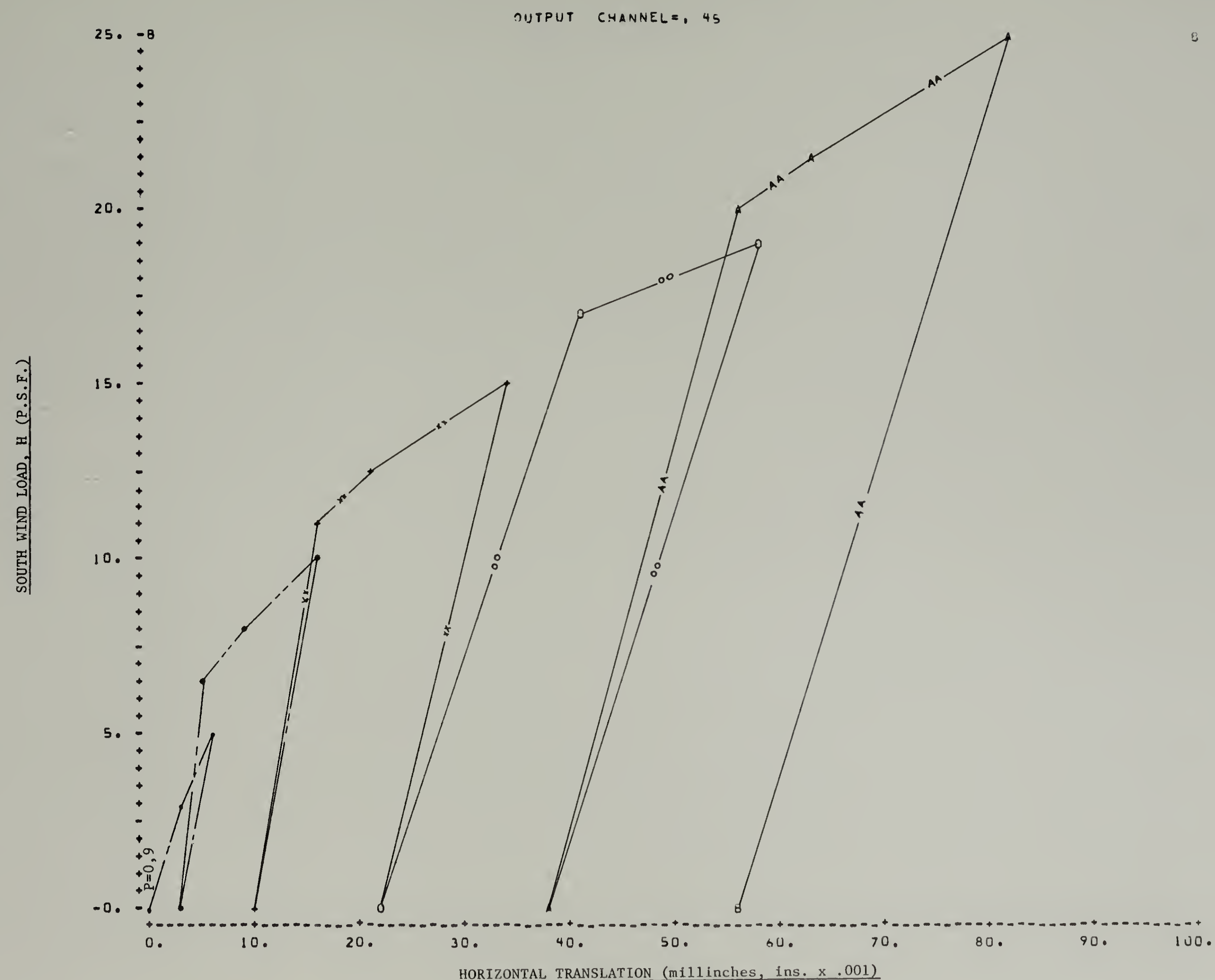


FIGURE No. A.4 - TEST NO. 2, SOUTH WIND LOAD VS. TRANSLATION

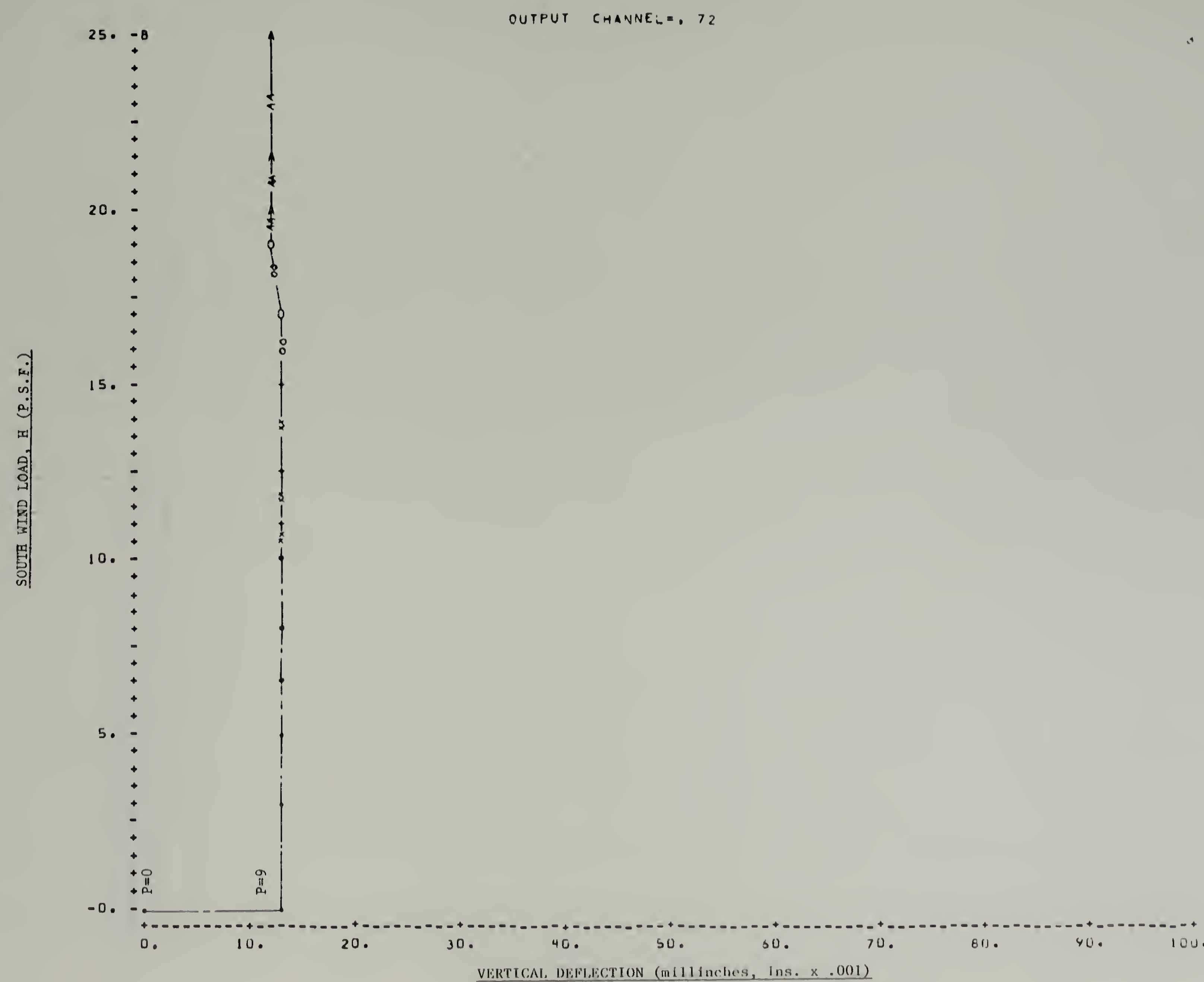


FIGURE NO. A.5 - TEST NO. 2, SOUTH WIND LOAD VS. BEAM DEFLECTION

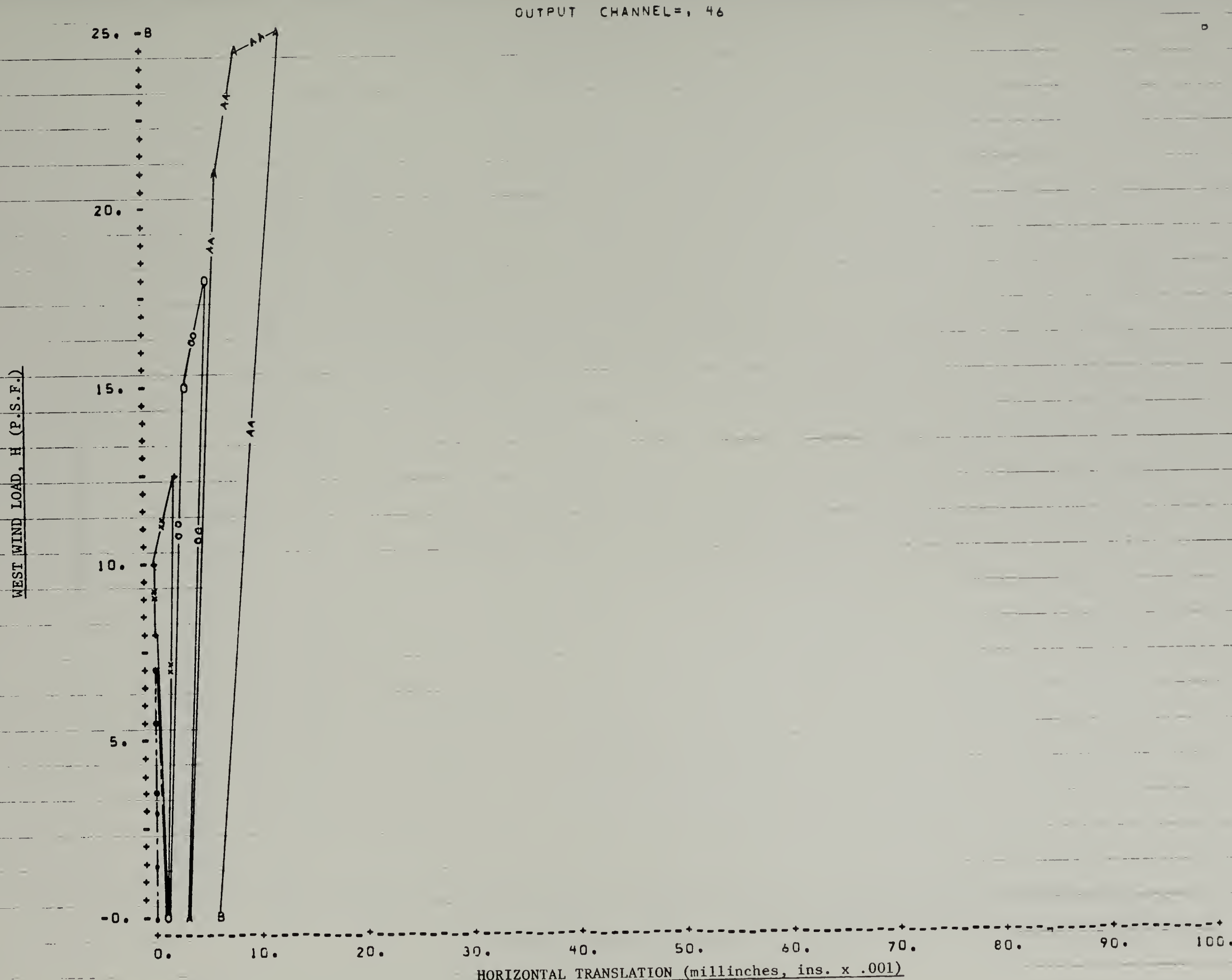


FIGURE NO. A.6 - TEST NO. 3, WEST WIND LOAD VS. TRANSLATION

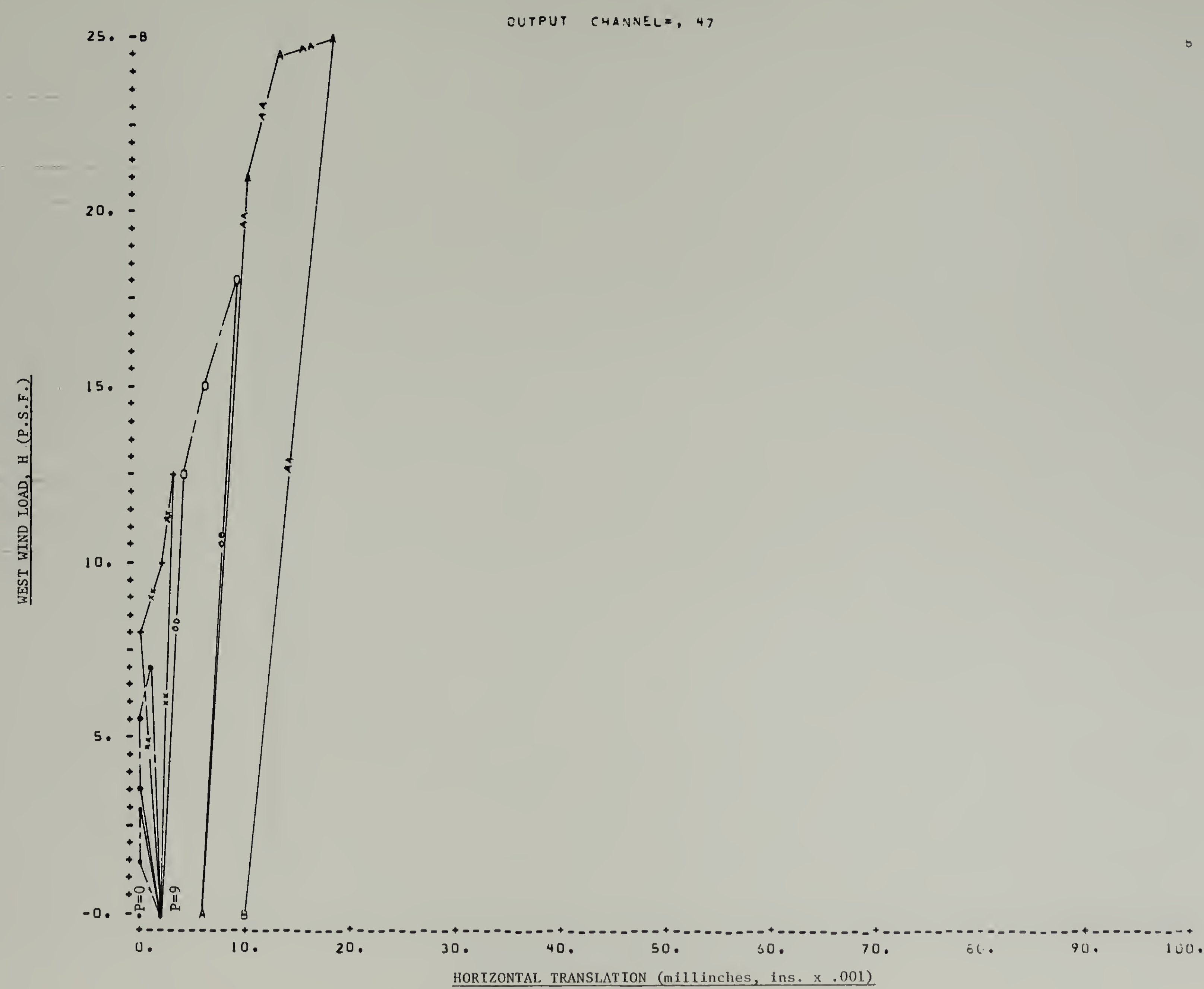


FIGURE NO. A.7 - TEST NO. 3, WEST WIND LOAD VS. TRANSLATION

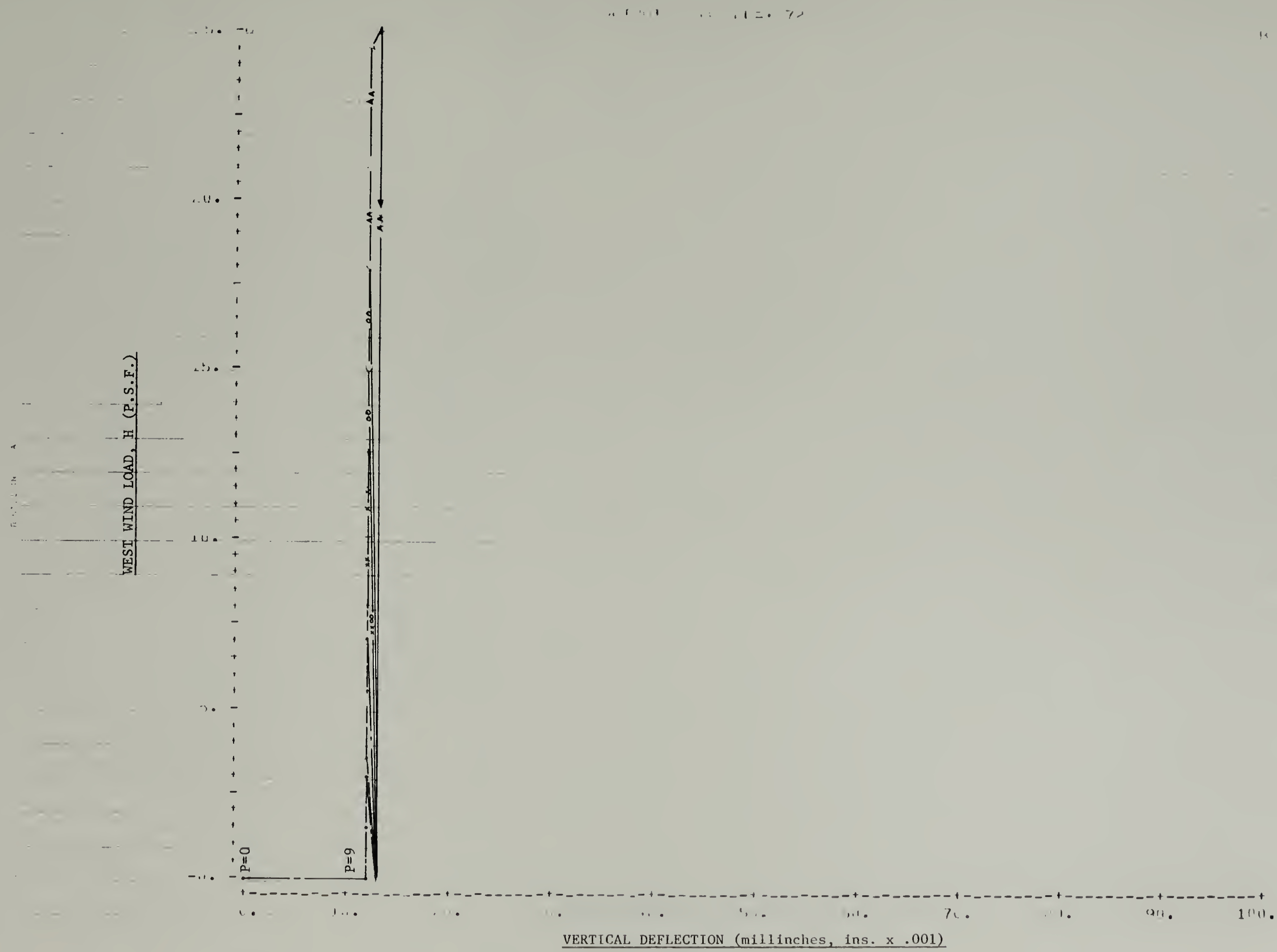


FIGURE NO. A.8 - TEST NO. 3, WEST WIND LOAD VS. BEAM DEFLECTION

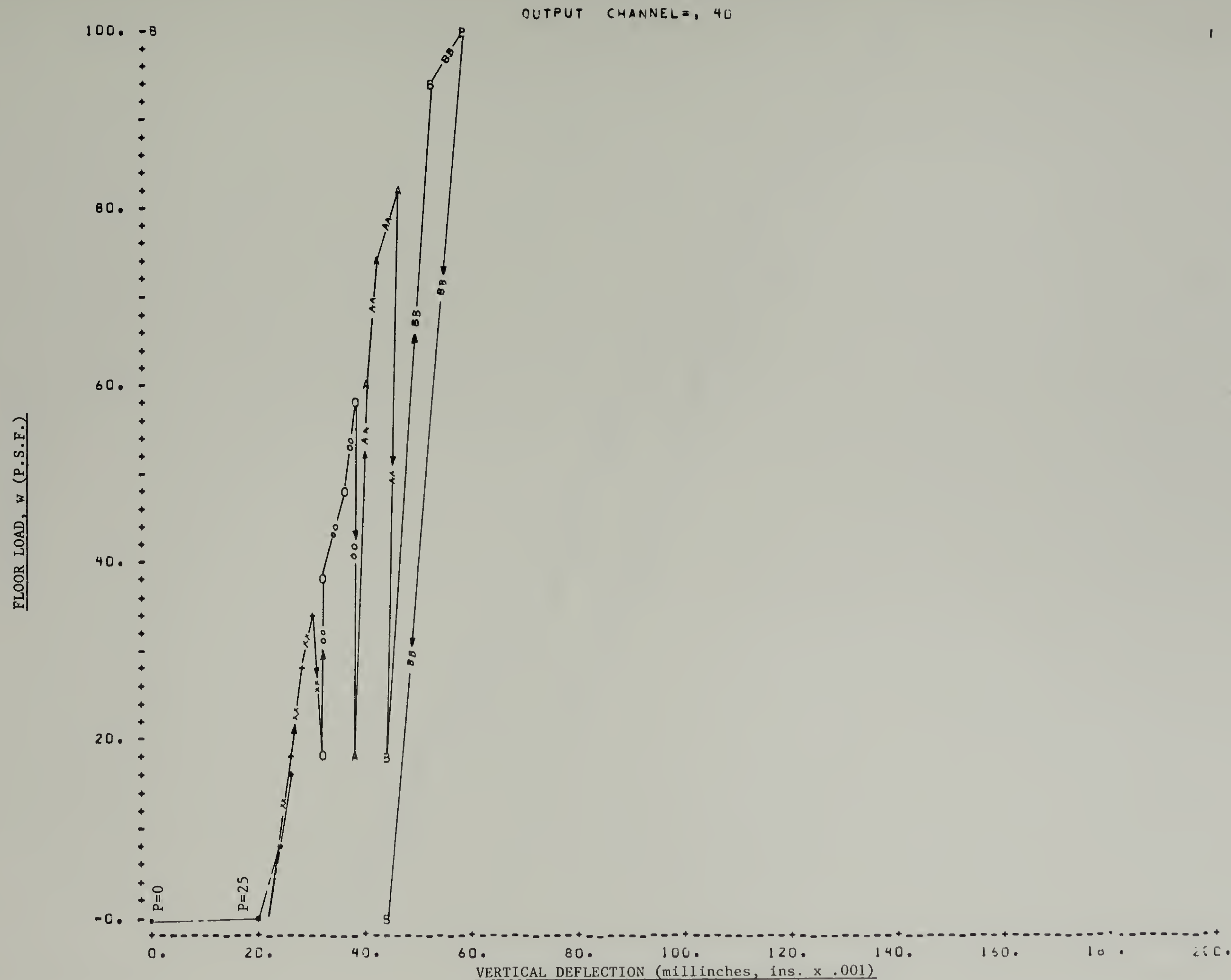


FIGURE NO. A.9 - TEST NO. 4, FLOOR LOAD VS. BEAM DEFLECTION

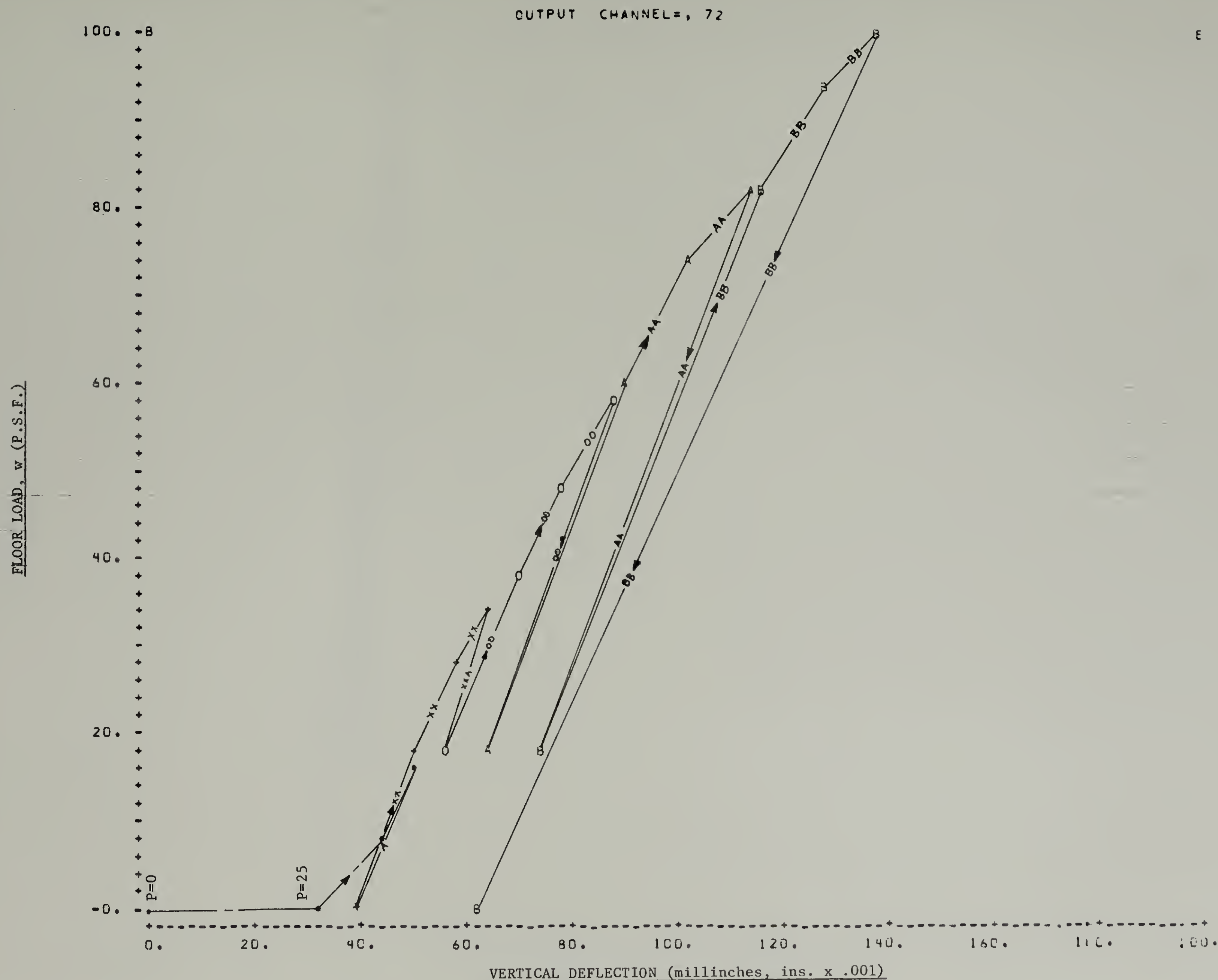


FIGURE NO. A.10 - TEST NO. 4, FLOOR LOAD VS. BEAM DEFLECTION

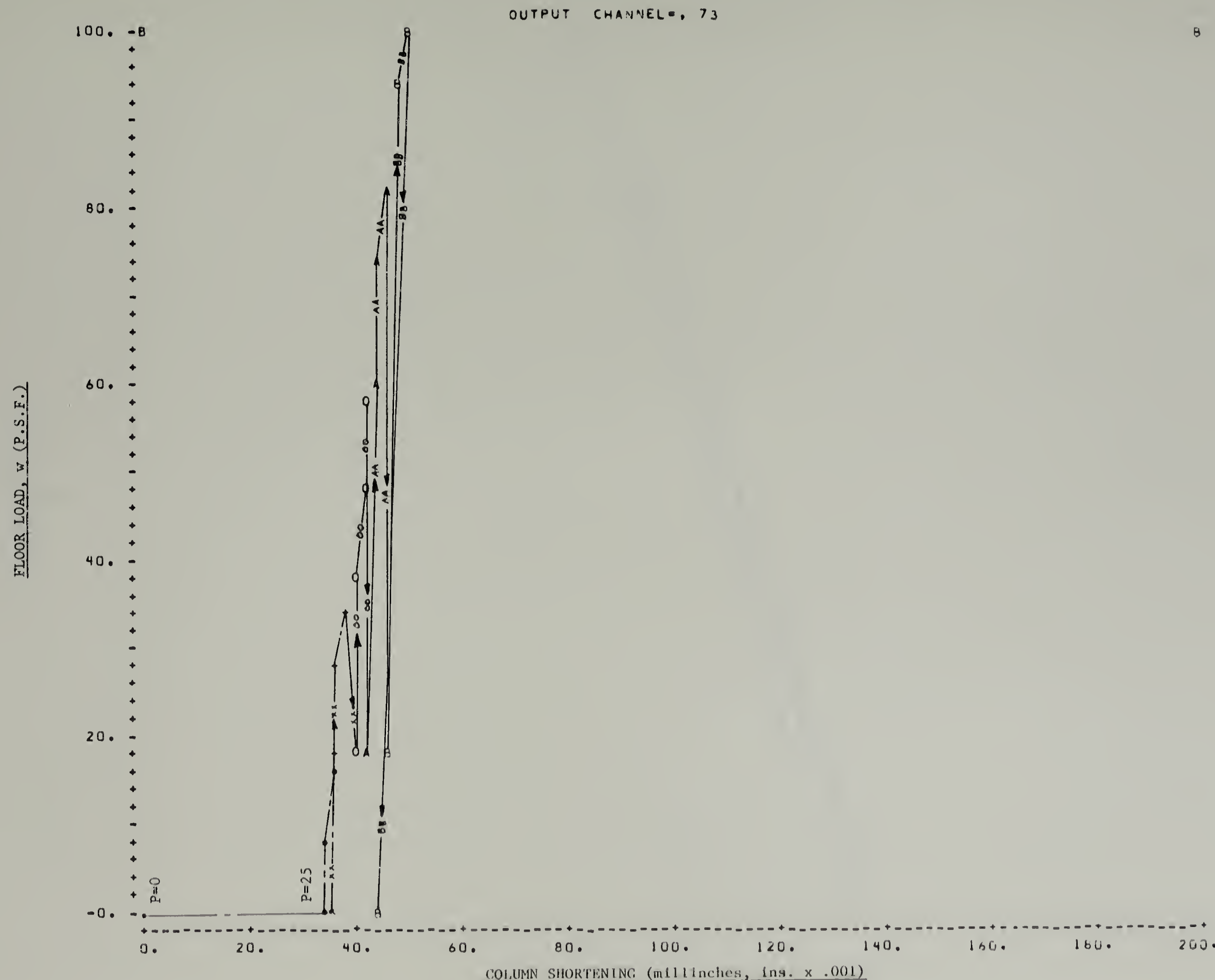


FIGURE NO. A.11 - TEST NO. 4, FLOOR LOAD VS. COLUMN SHORTENING

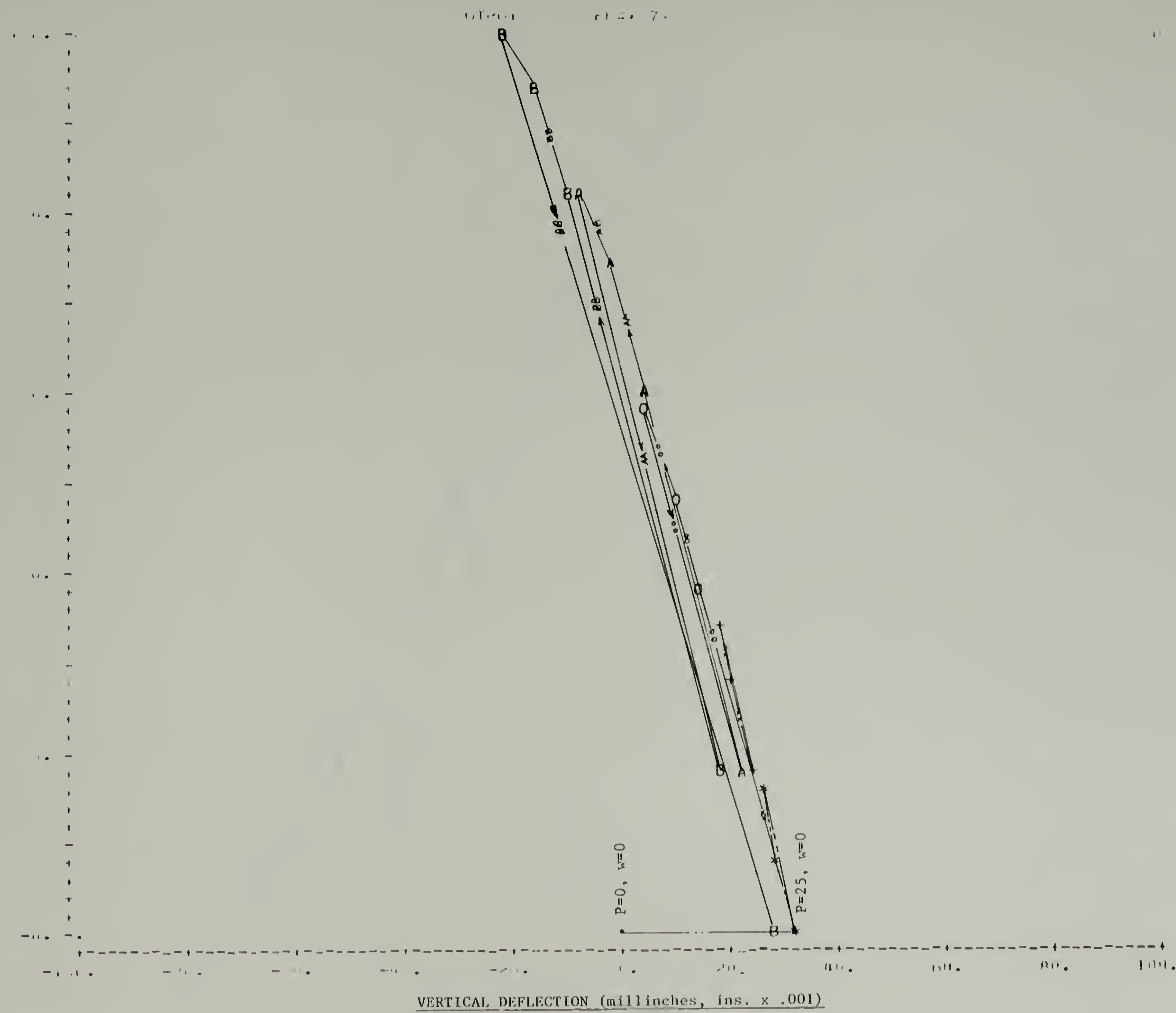


FIGURE NO. A.12 - TEST NO. 4, FLOOR LOAD VS. BEAM DEFLECTION

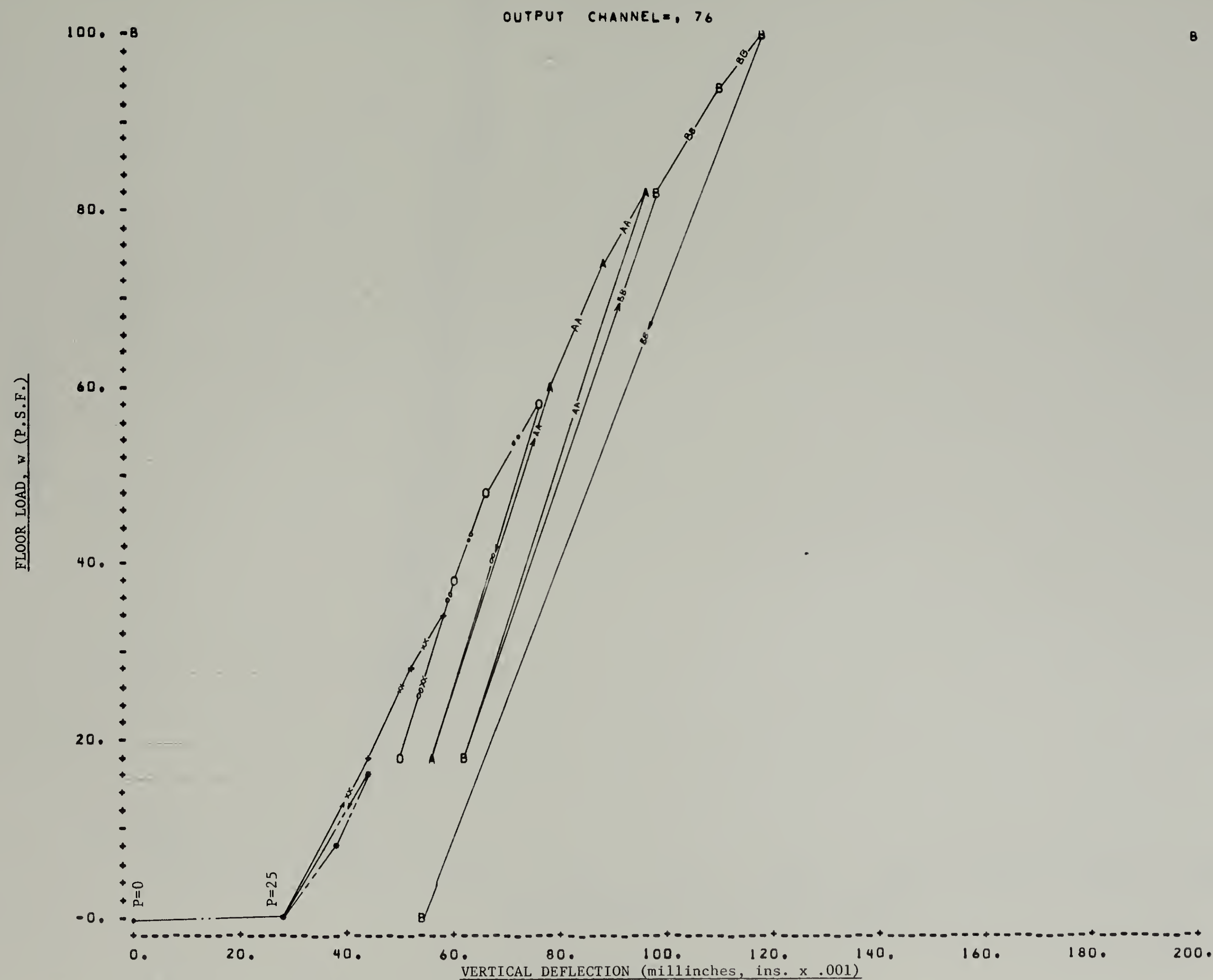


FIGURE NO. A.13 - TEST NO. 4, FLOOR LOAD VS. SLAB DEFLECTION

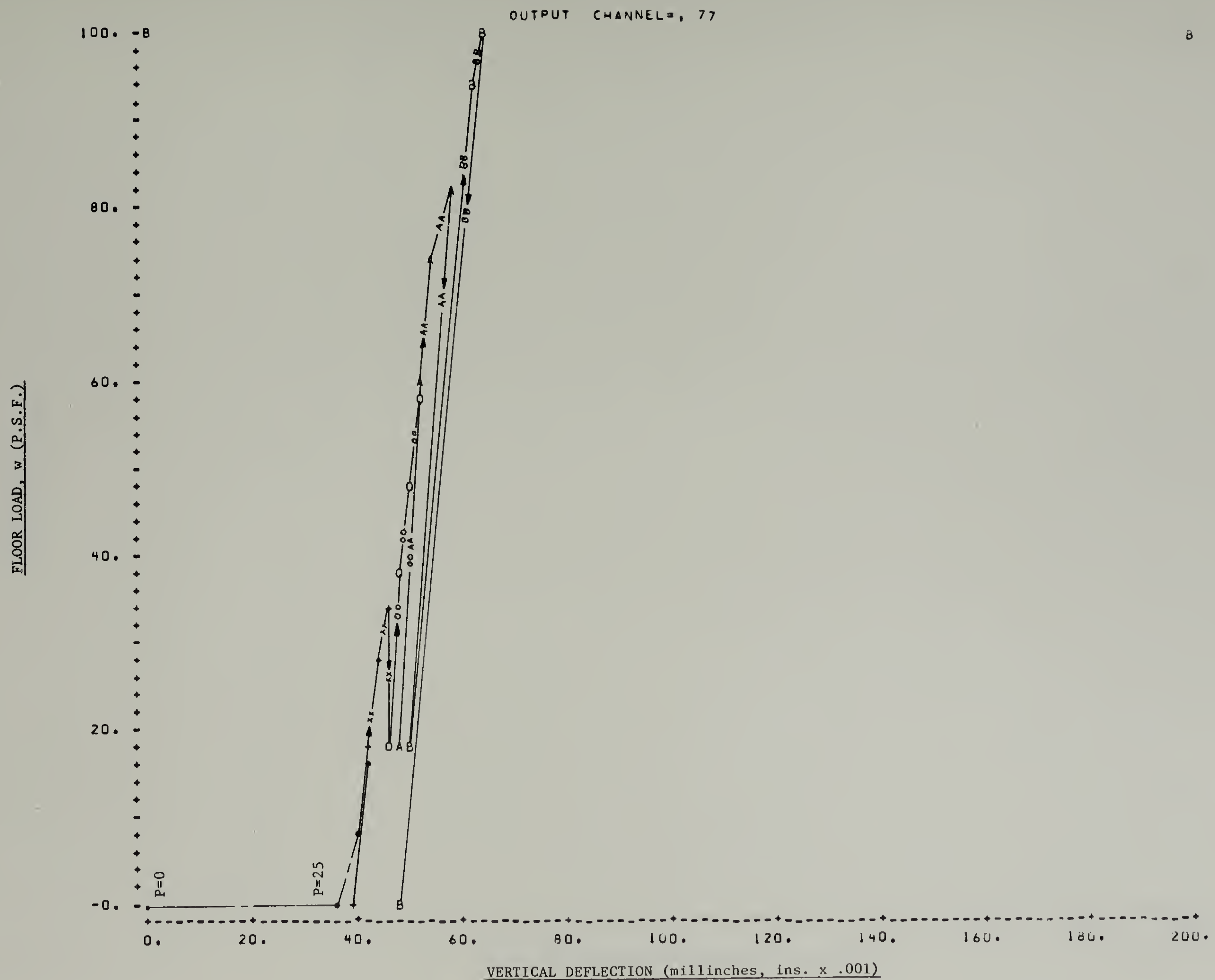


FIGURE NO. A.14 - TEST NO. 4, FLOOR LOAD VS. BEAM DEFLECTION

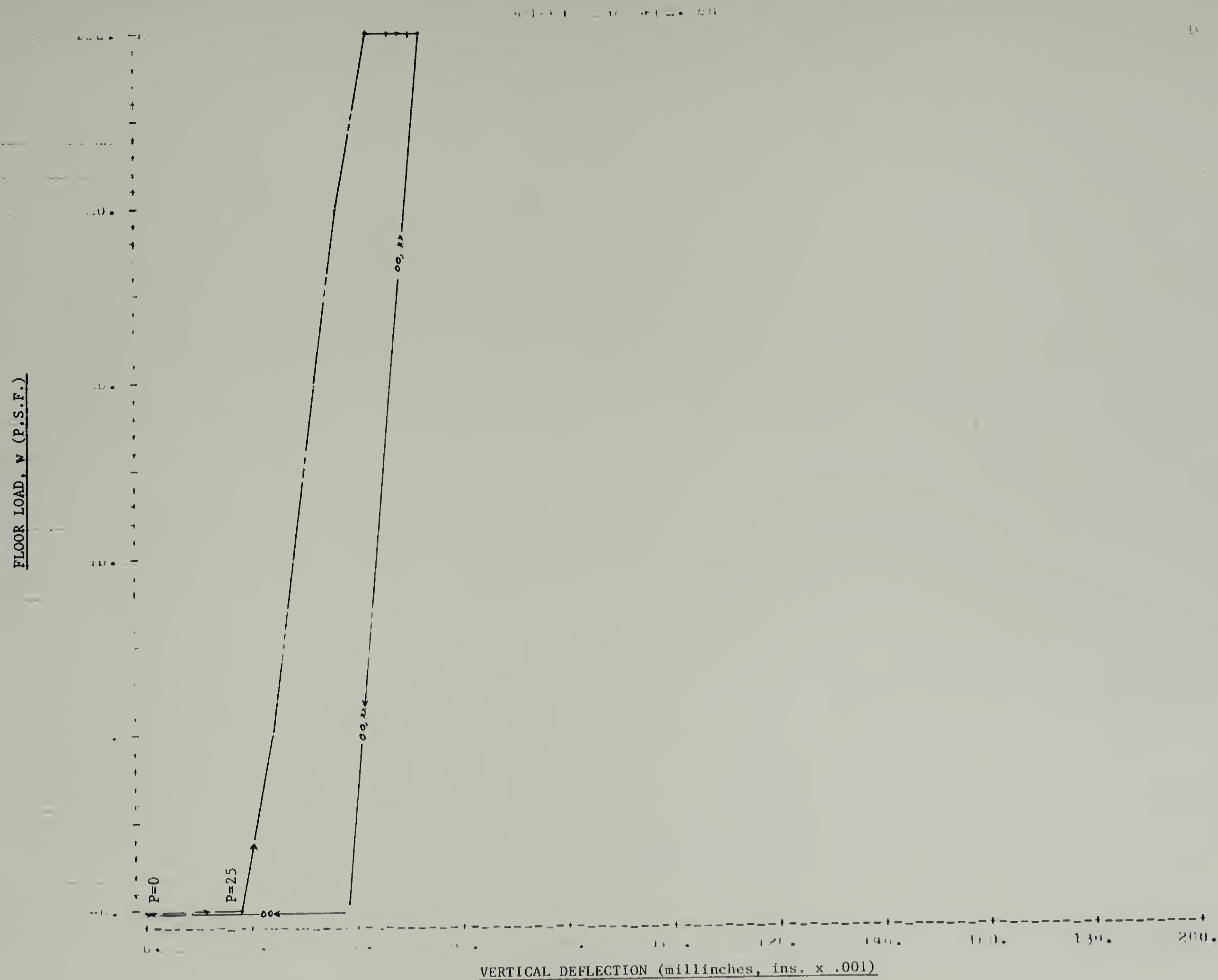


FIGURE NO. A.15 - TEST NO. 5, SUSTAINED FLOOR LOAD VS. BEAM DEFLECTION

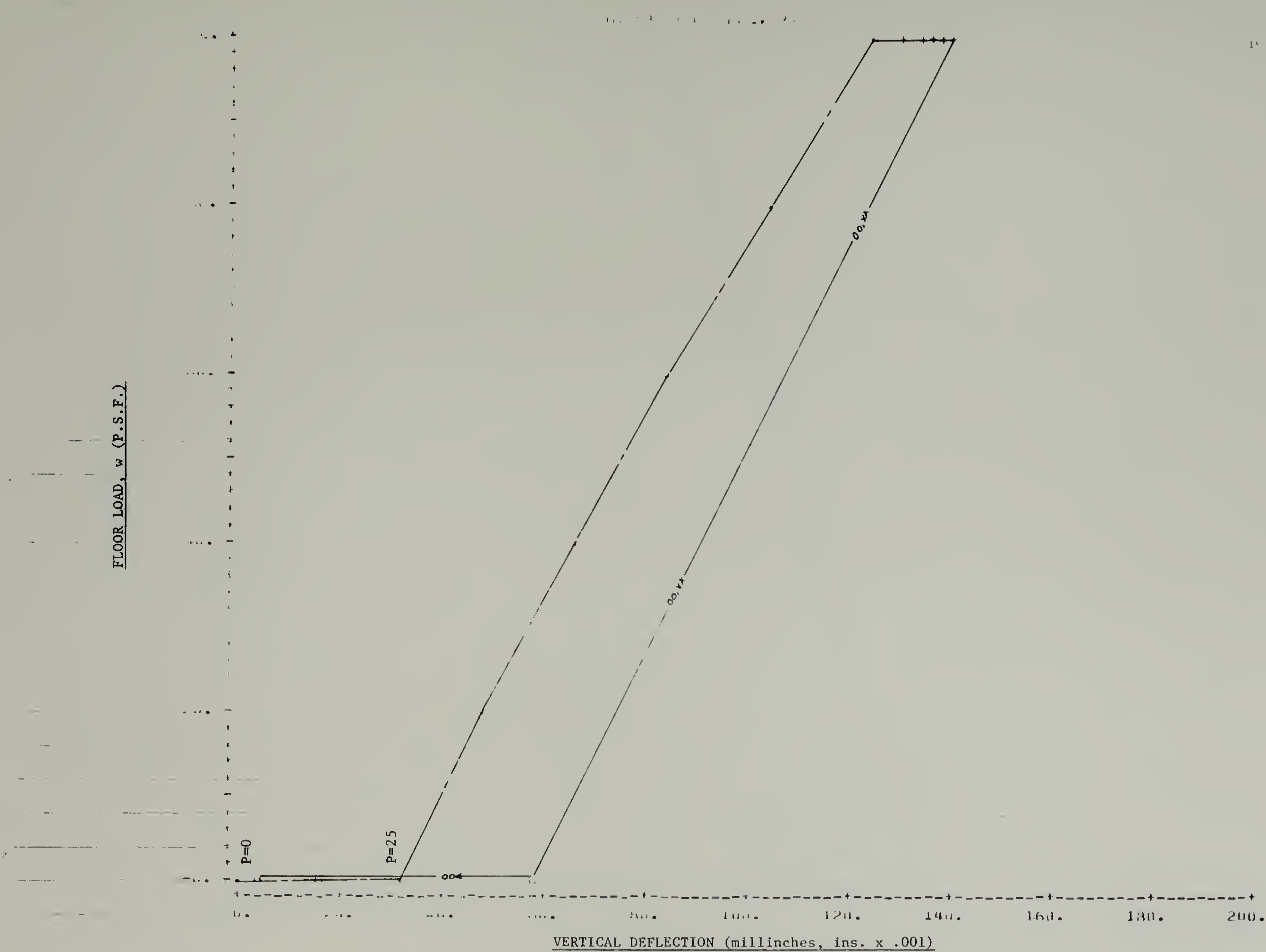


FIGURE NO. A.16 - TEST NO. 5, SUSTAINED FLOOR LOAD VS. BEAM DEFLECTION

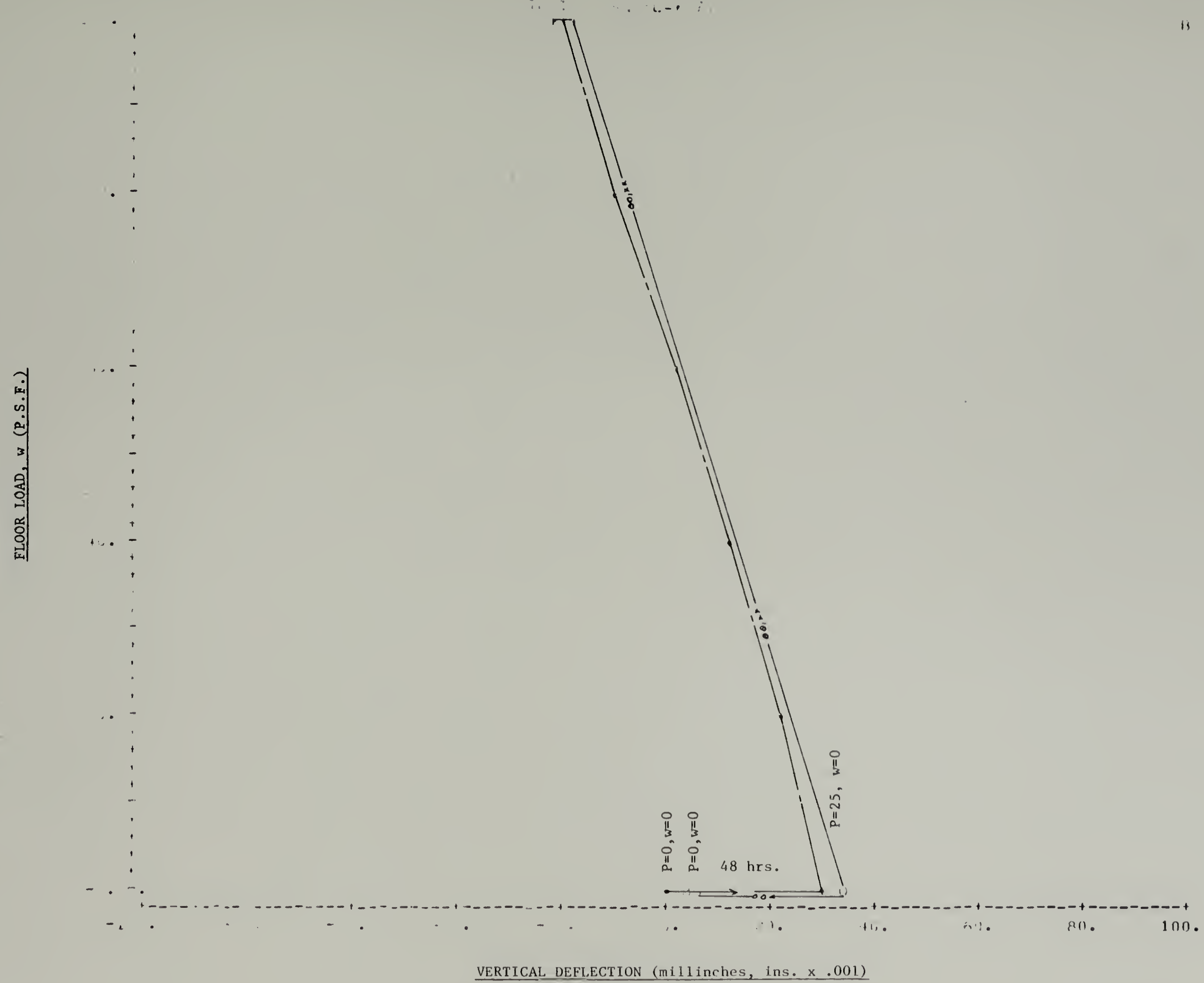


FIGURE NO. A.17 - TEST NO. 5, SUSTAINED FLOOR LOAD VS. BEAM DEFLECTION

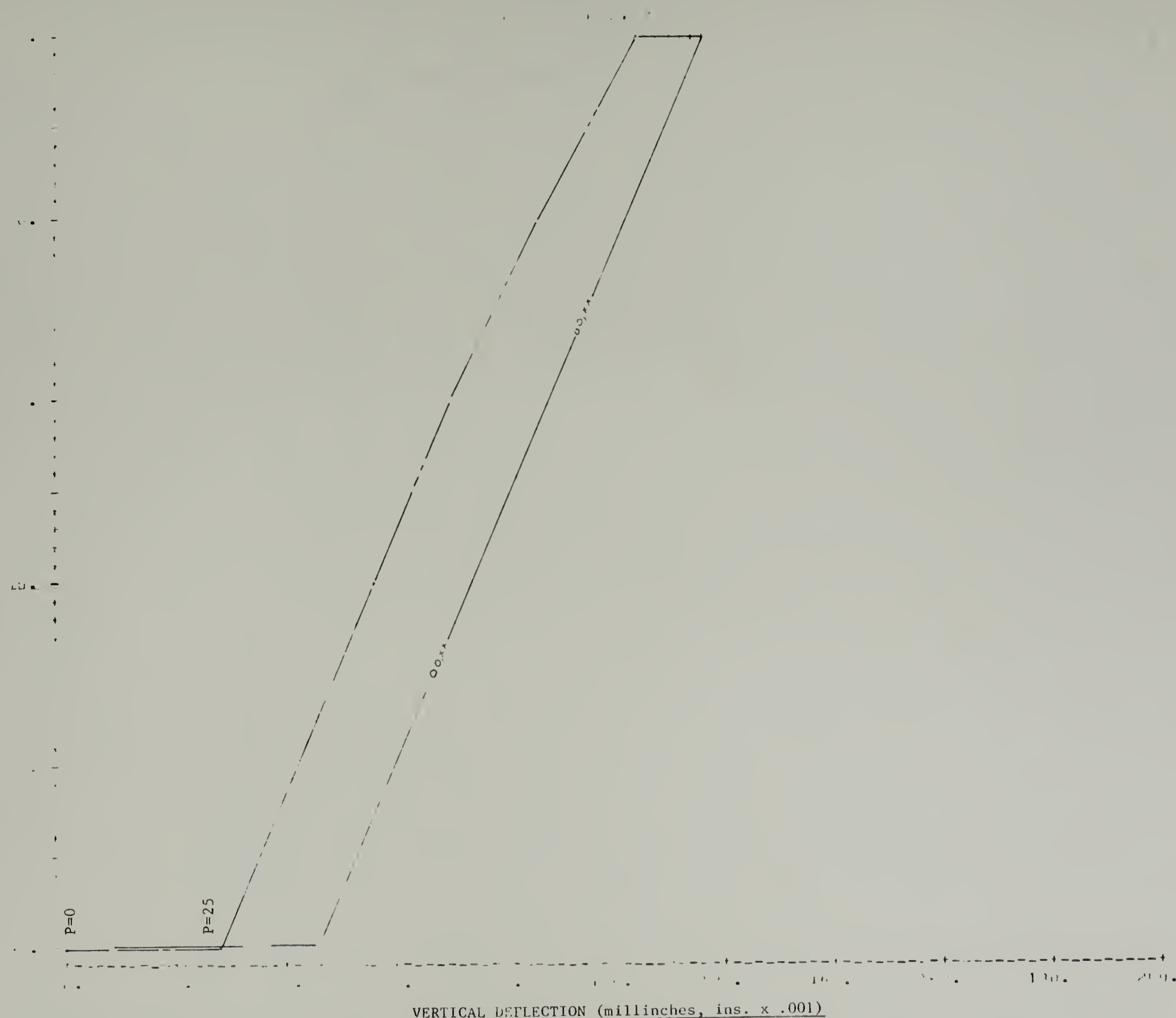


FIGURE NO. A.18 _ TEST NO. 5, SUSTAINED FLOOR ... DEFLECTION

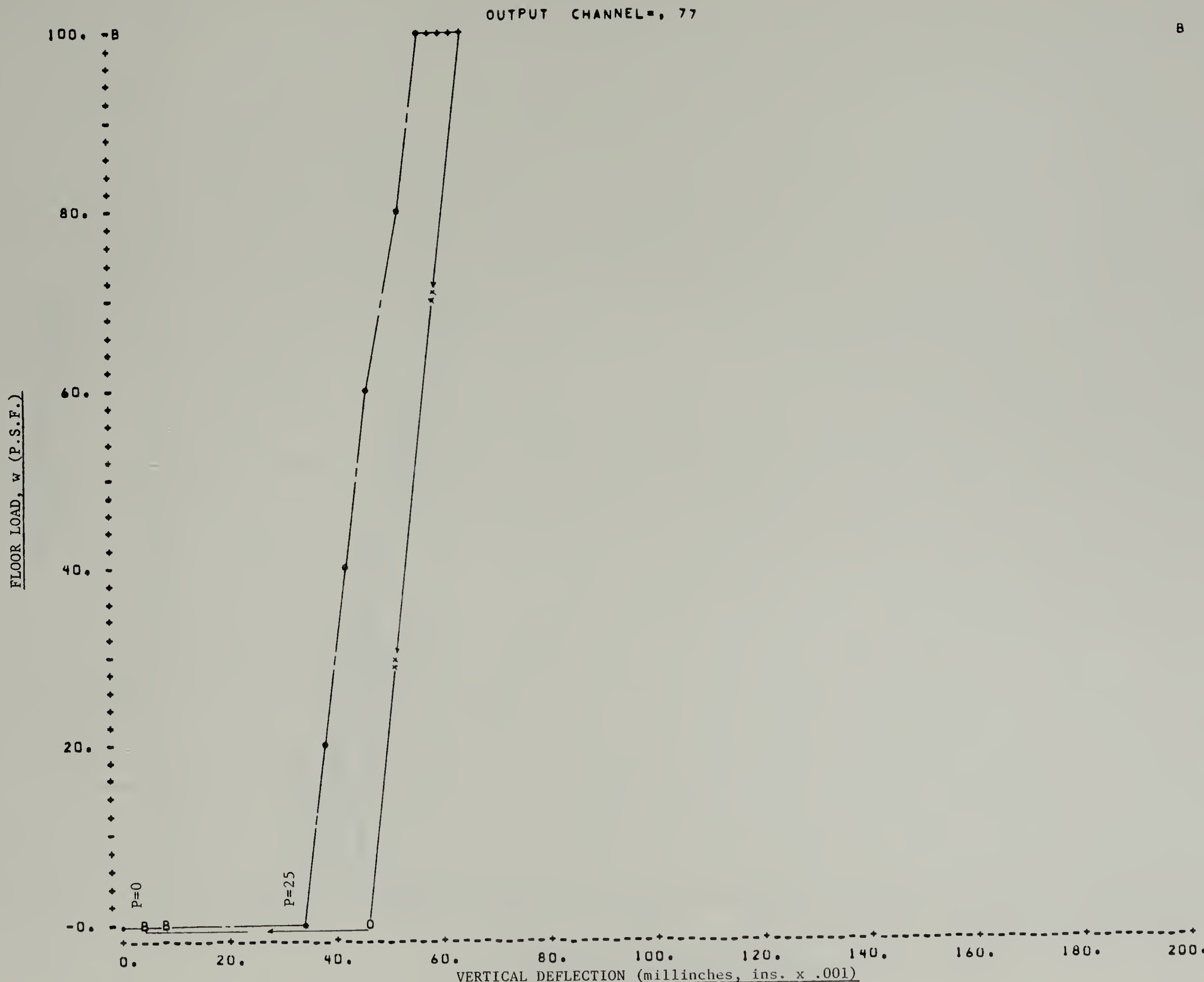


FIGURE NO. A.19 - TEST NO. 5, SUSTAINED FLOOR LOAD VS. BEAM DEFLECTION

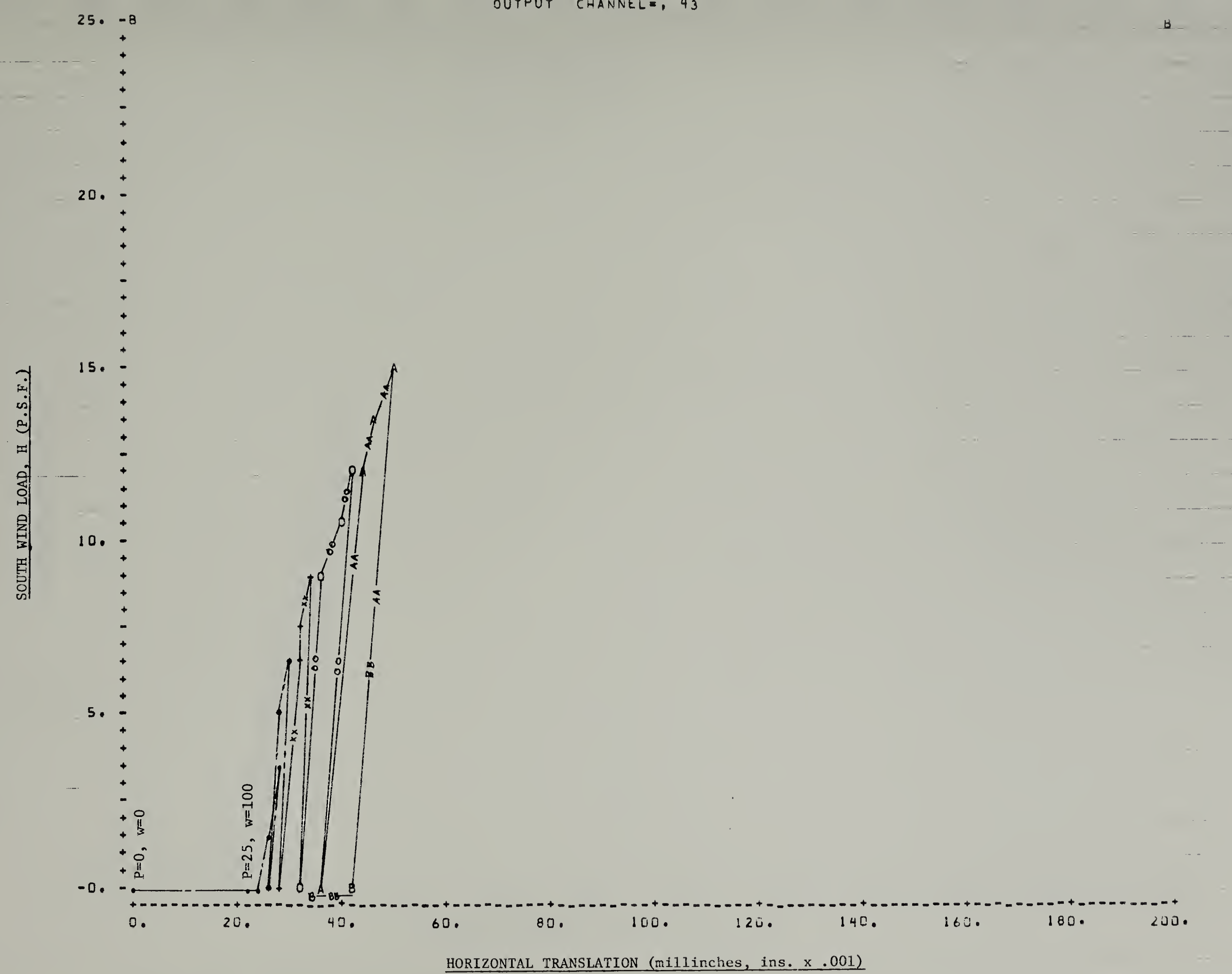


FIGURE NO. A.20 - TEST NO. 7, SOUTH WIND LOAD VS. TRANSLATION

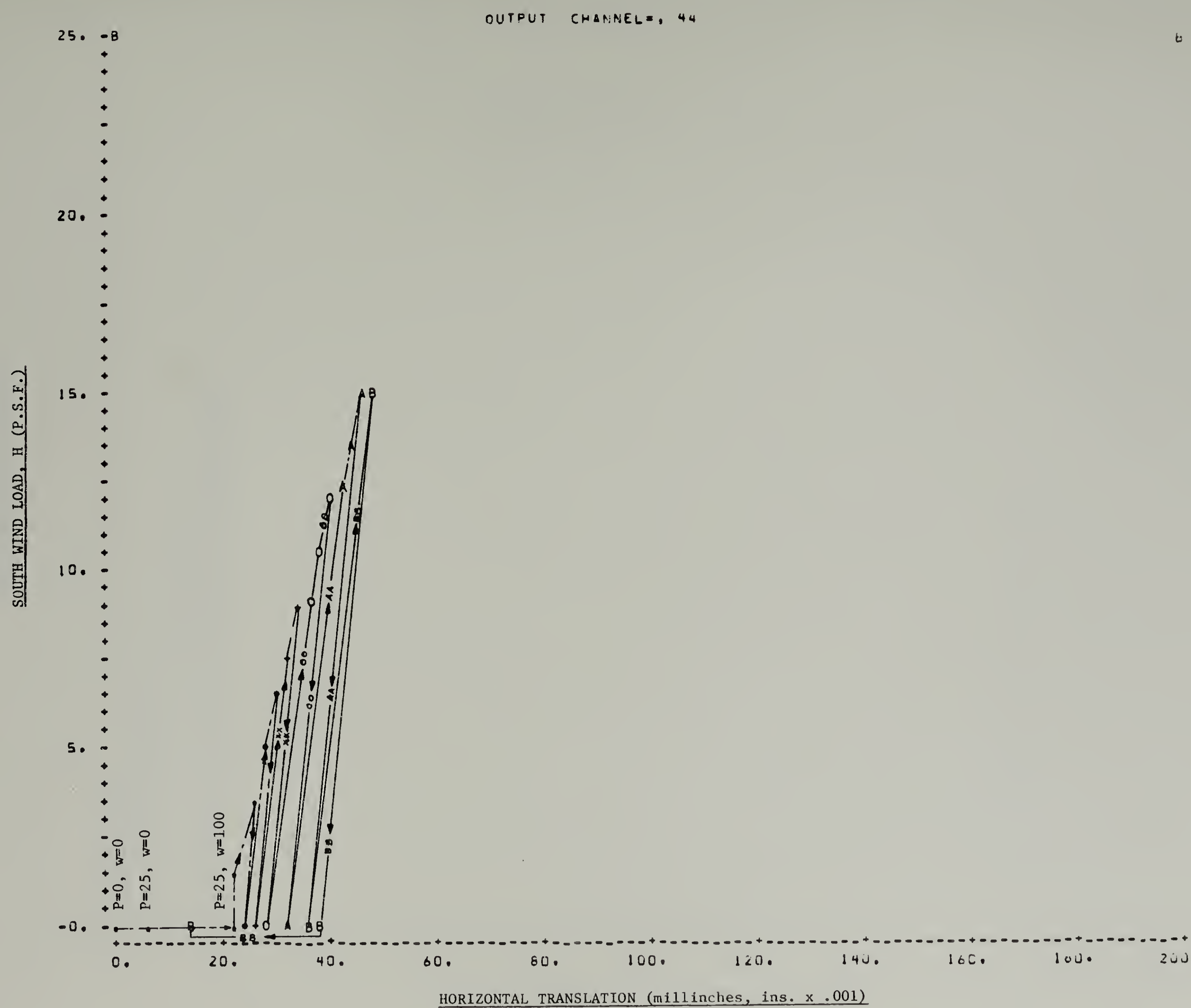


FIGURE NO. A.21 - TEST NO. 7, SOUTH WIND LOAD VS. TRANSLATION

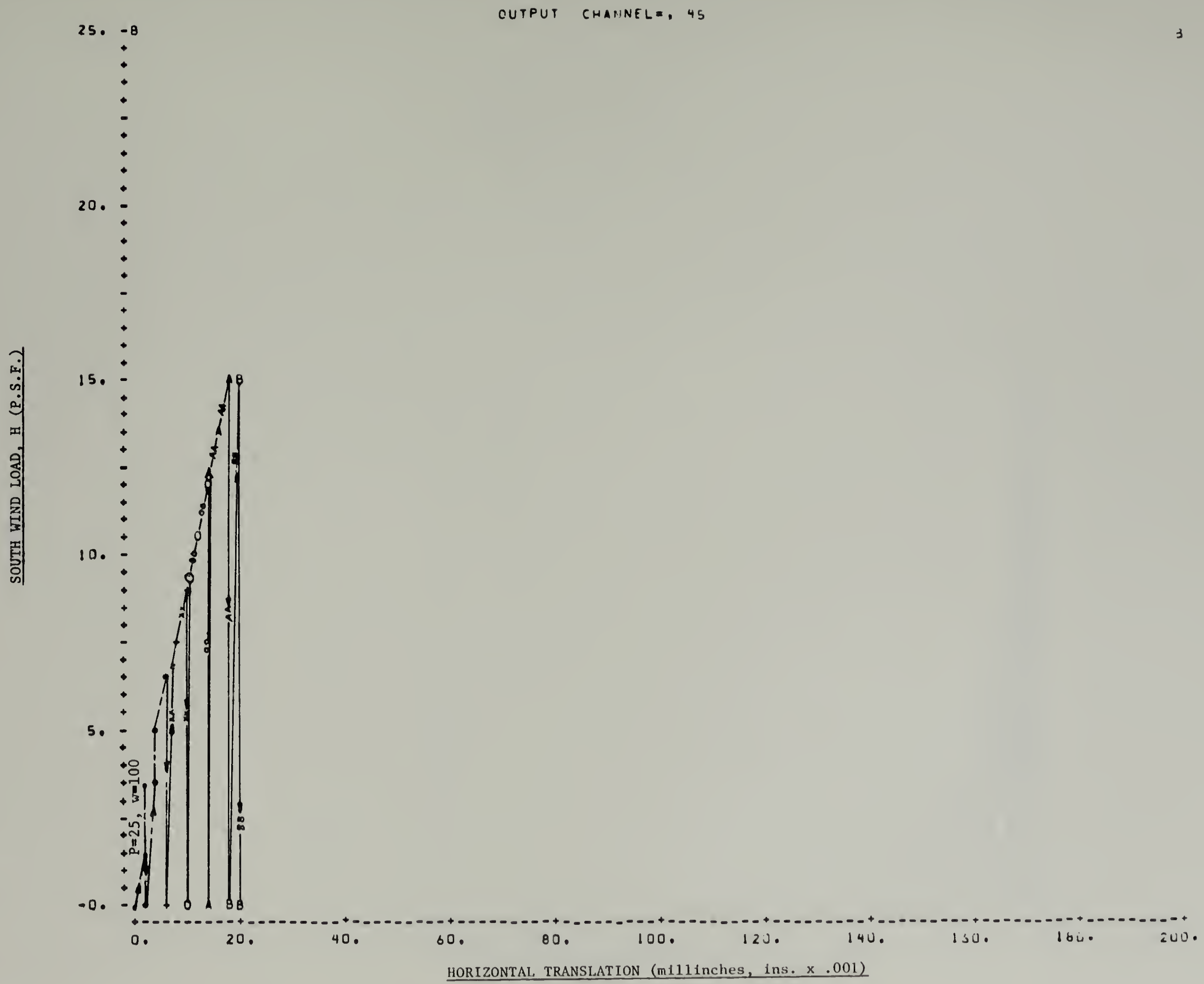


FIGURE NO. A.22 - TEST NO. 7, SOUTH WIND LOAD VS. TRANSLATION

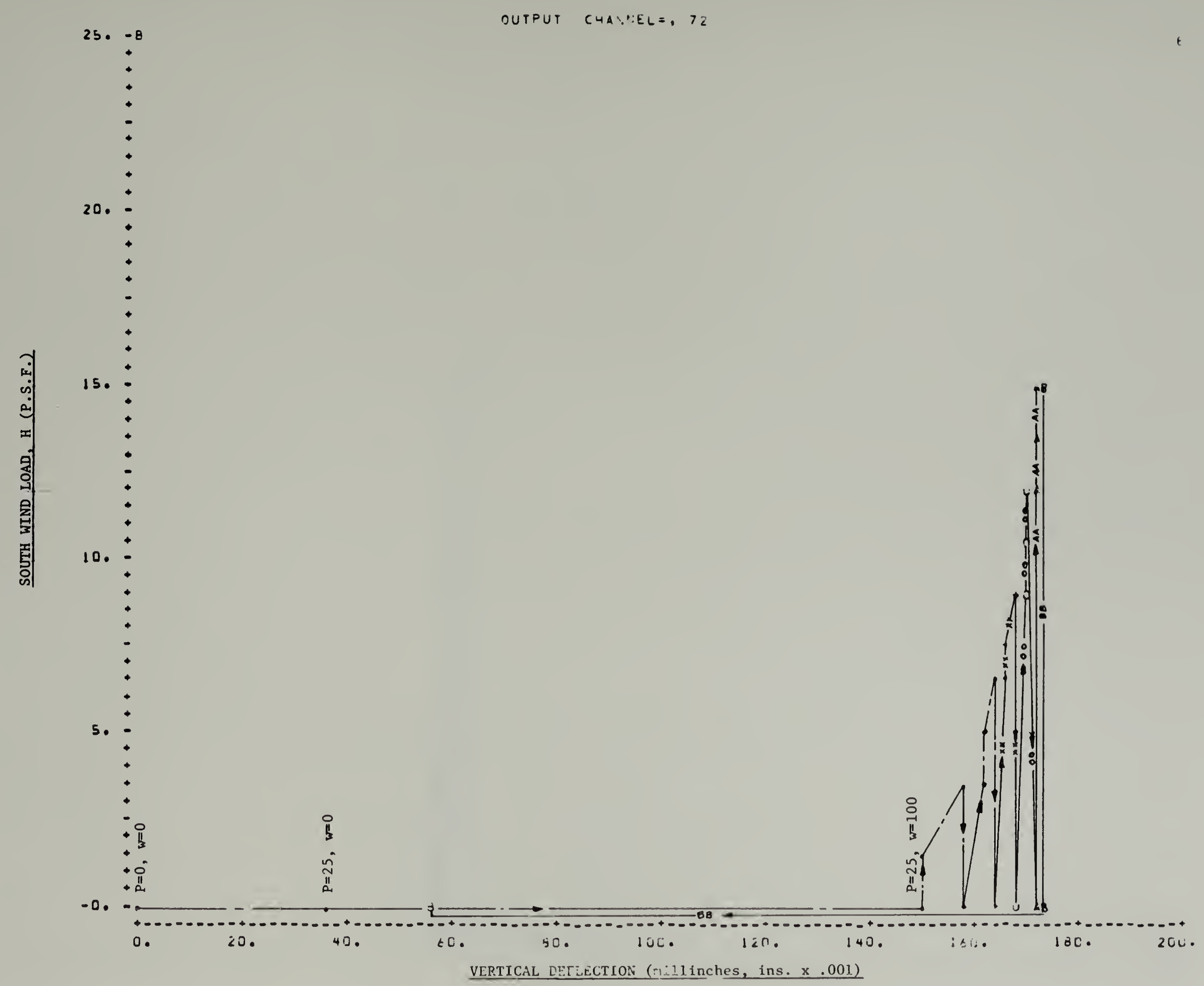


FIGURE NO. A.23 - TEST NO. 7, SOUTH WIND LOAD VS. BEAM DEFLECTION

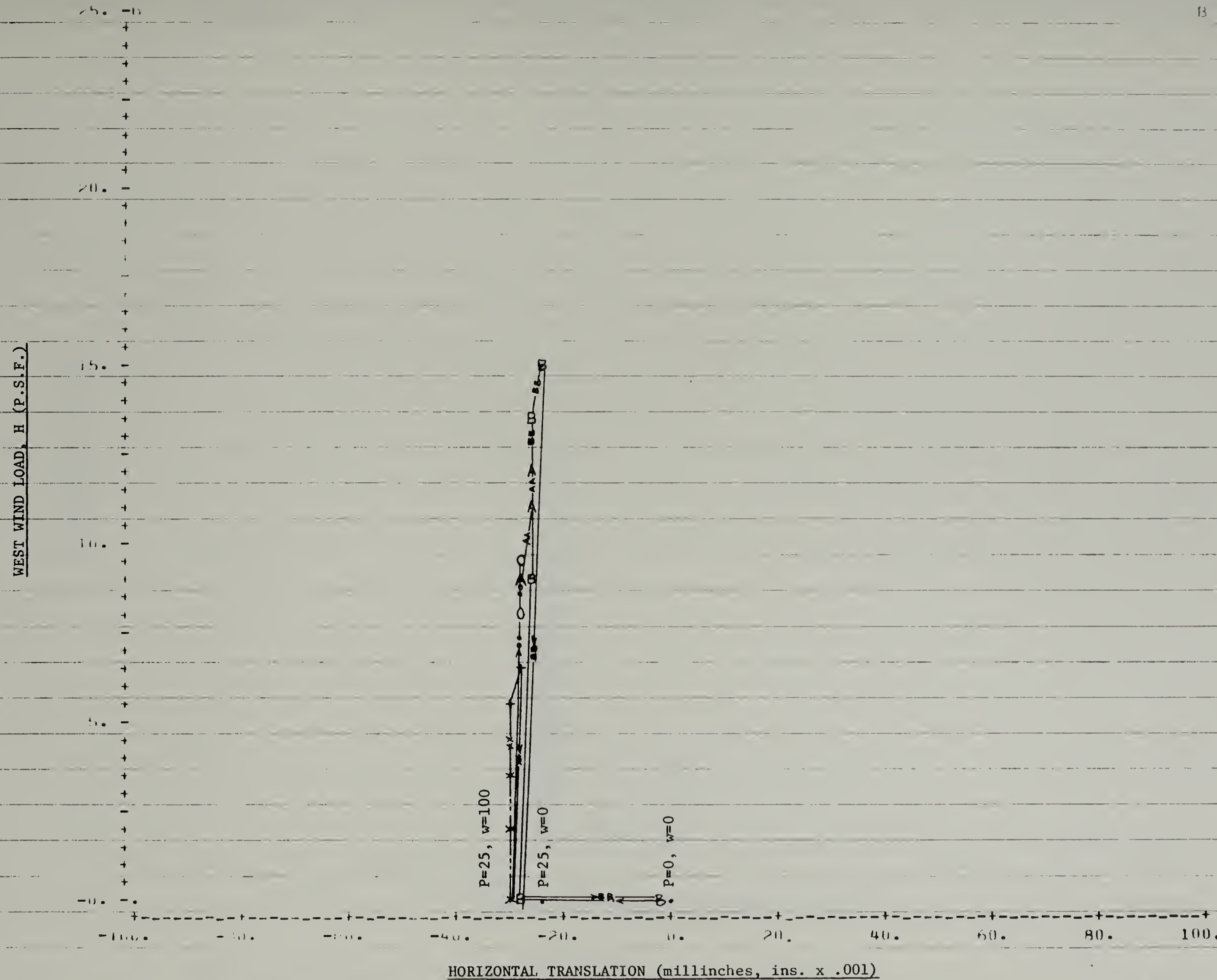


FIGURE NO. A.24 - TEST NO. 8, WEST WIND LOAD VS. TRANSLATION

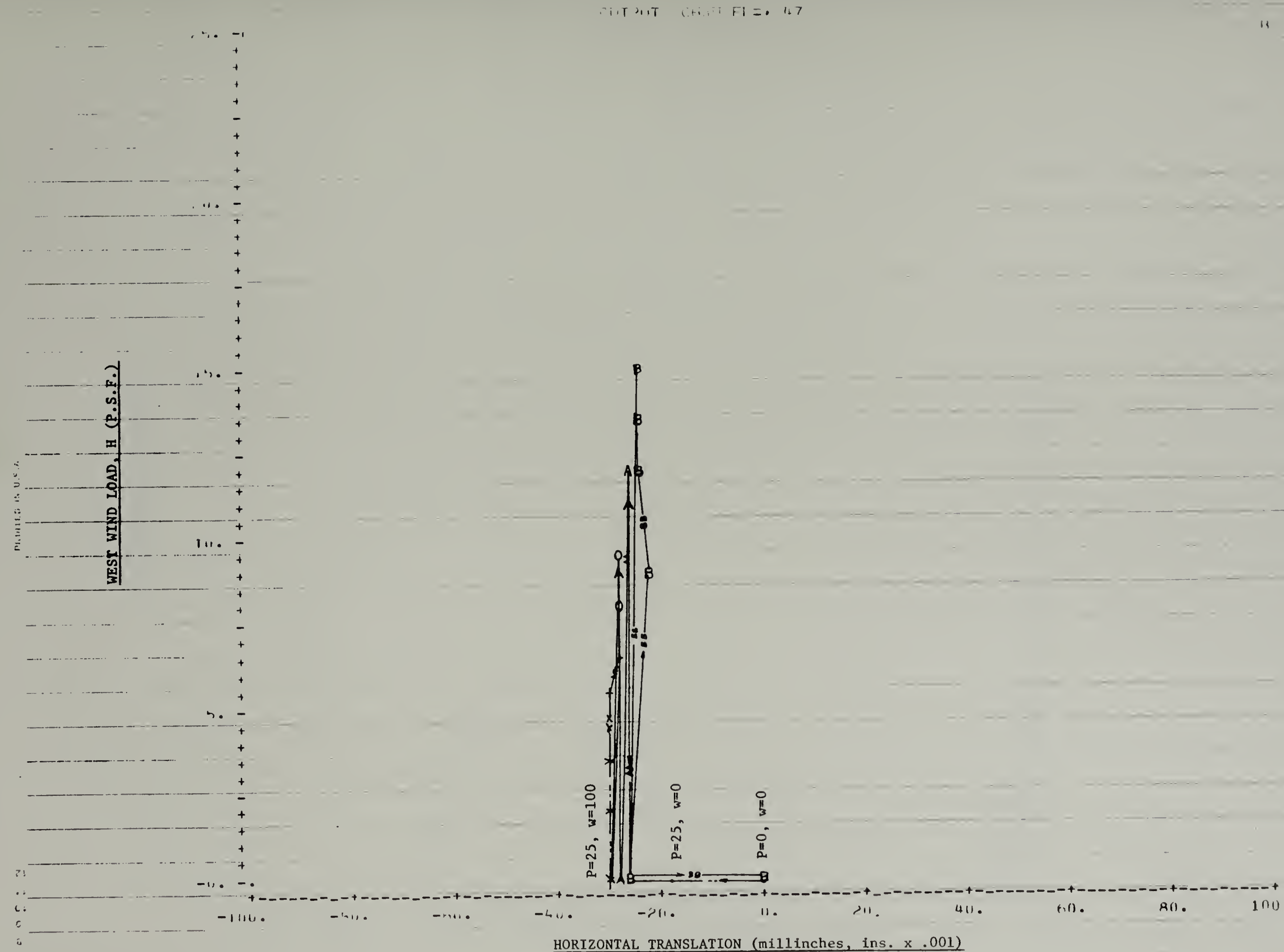


FIGURE NO. A.25 - TEST NO. 8, WEST WIND LOAD VS. TRANSLATION

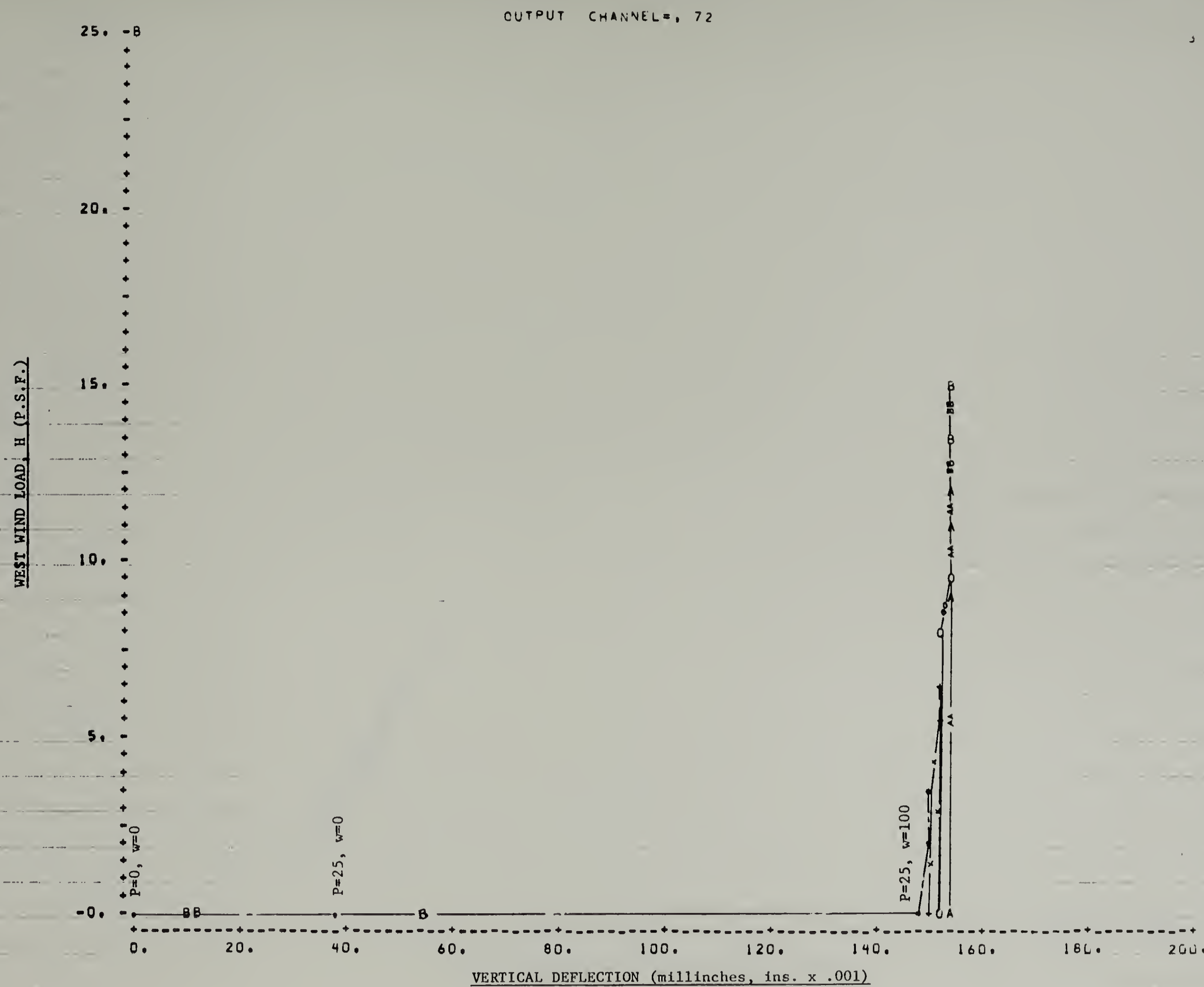


FIGURE NO. A.26 - TEST NO. 8 - WEST WIND LOAD VS. BEAM DEFLECTION

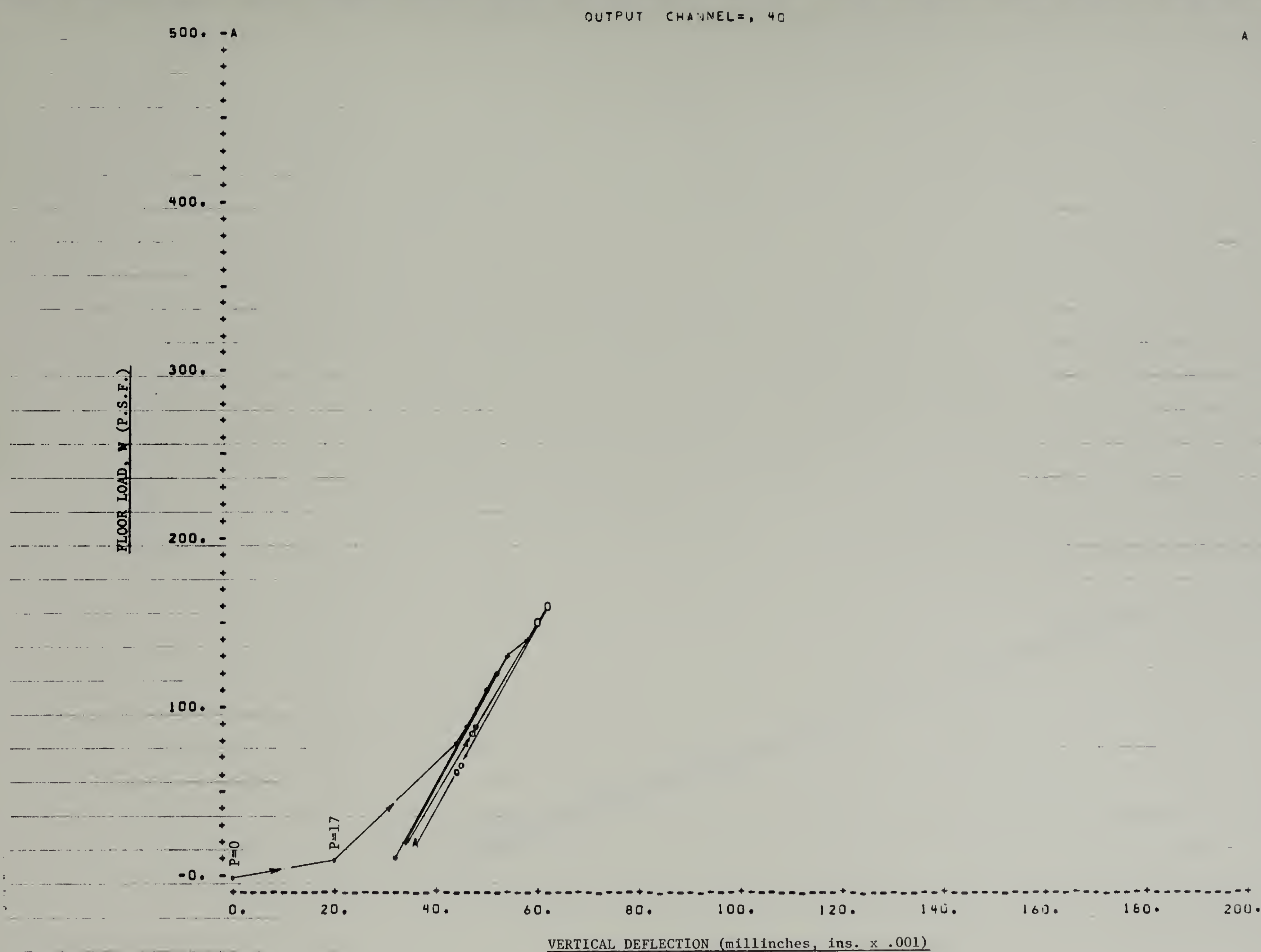


FIGURE NO. A.27 - TEST NO. 9, MAJOR FLOOR LOAD VS. BEAM DEFLECTION

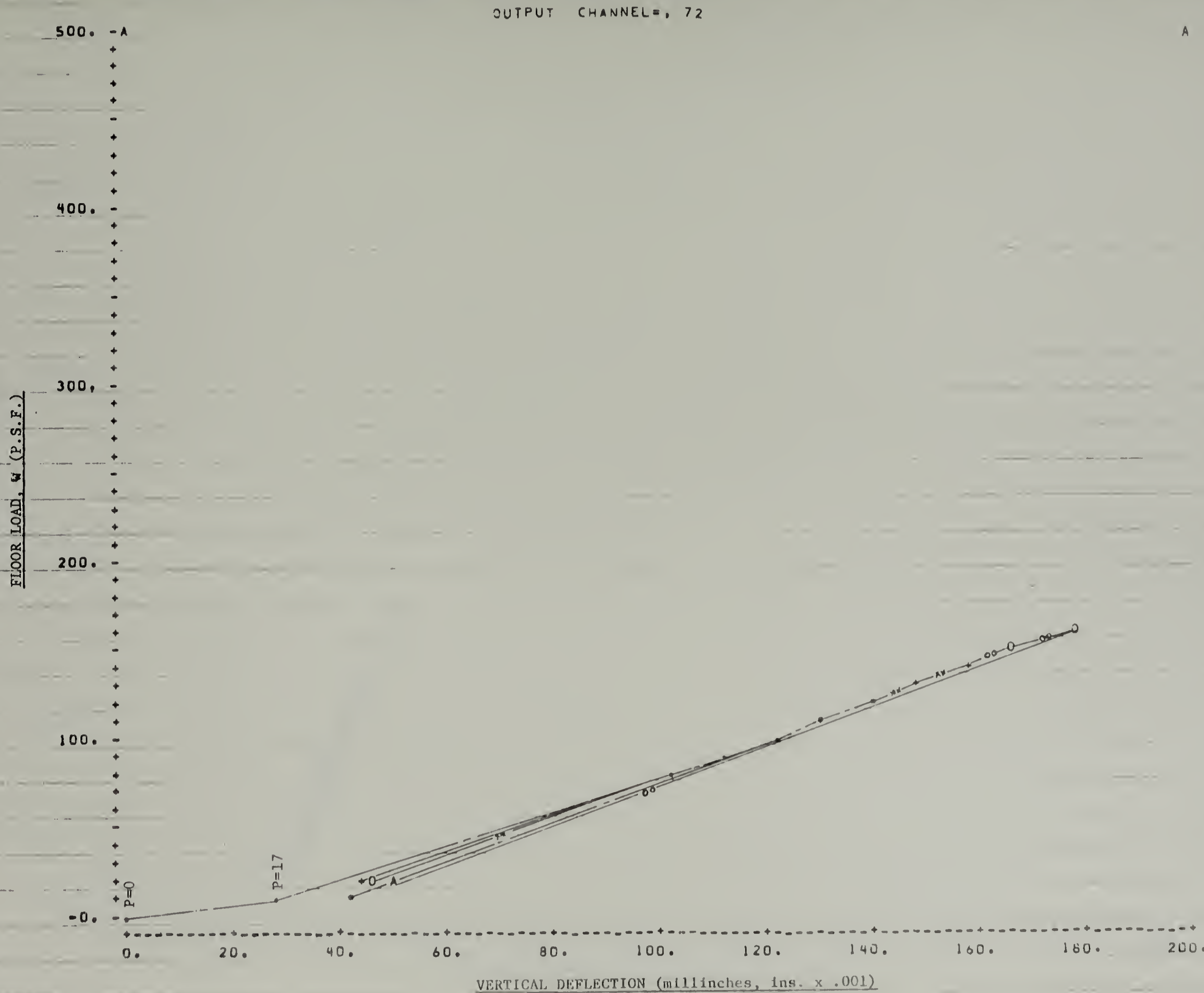


FIGURE NO. A.28 - TEST NO. 9, MAJOR FLOOR LOAD VS. BEAM DEFLECTION

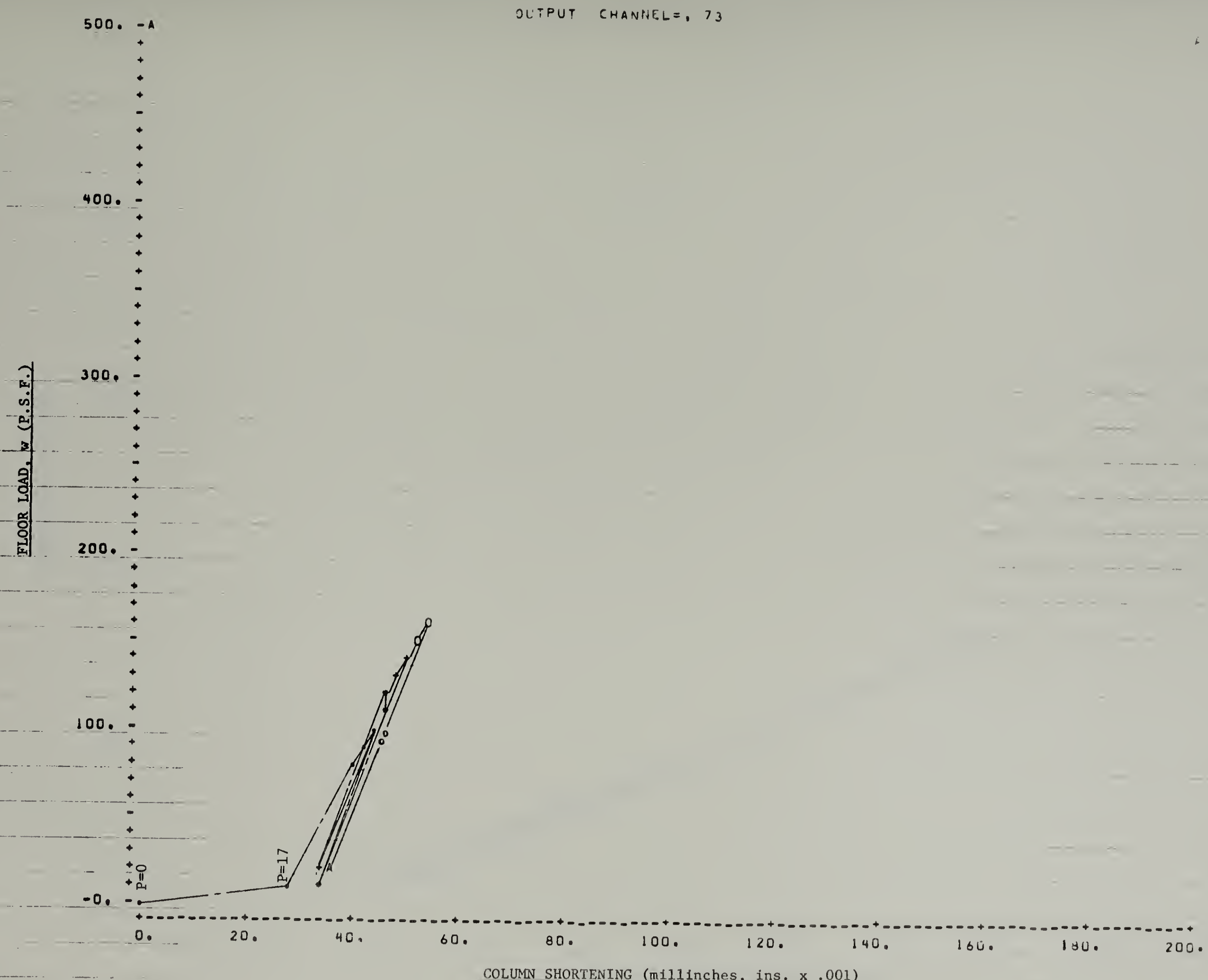


FIGURE NO. A.29 - TEST NO. 9, MAJOR FLOOR LOAD VS. COLUMN SHORTENING

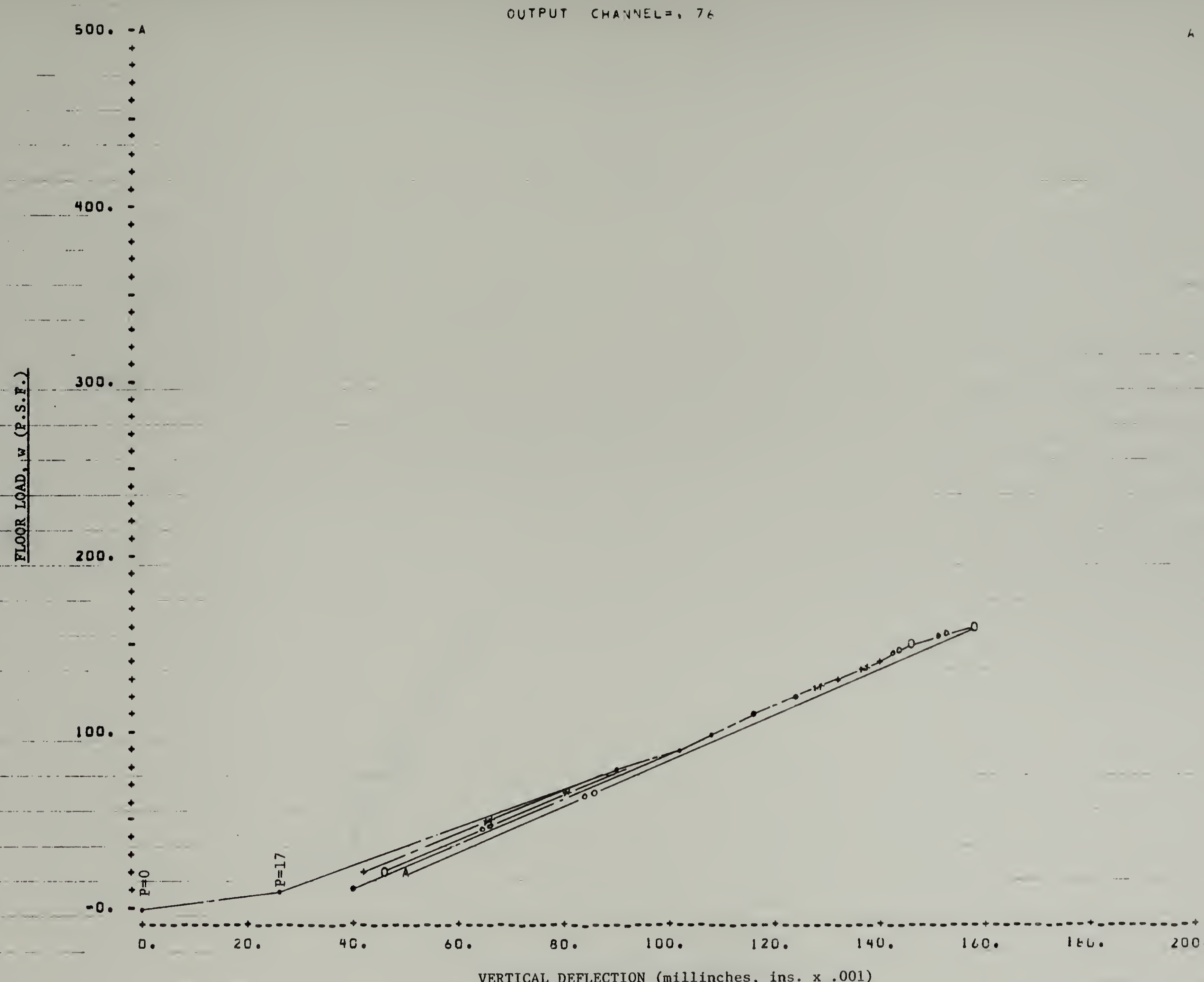


FIGURE NO. A.30 - TEST NO. 9, MAJOR FLOOR LOAD VS. SLAB DEFLECTION

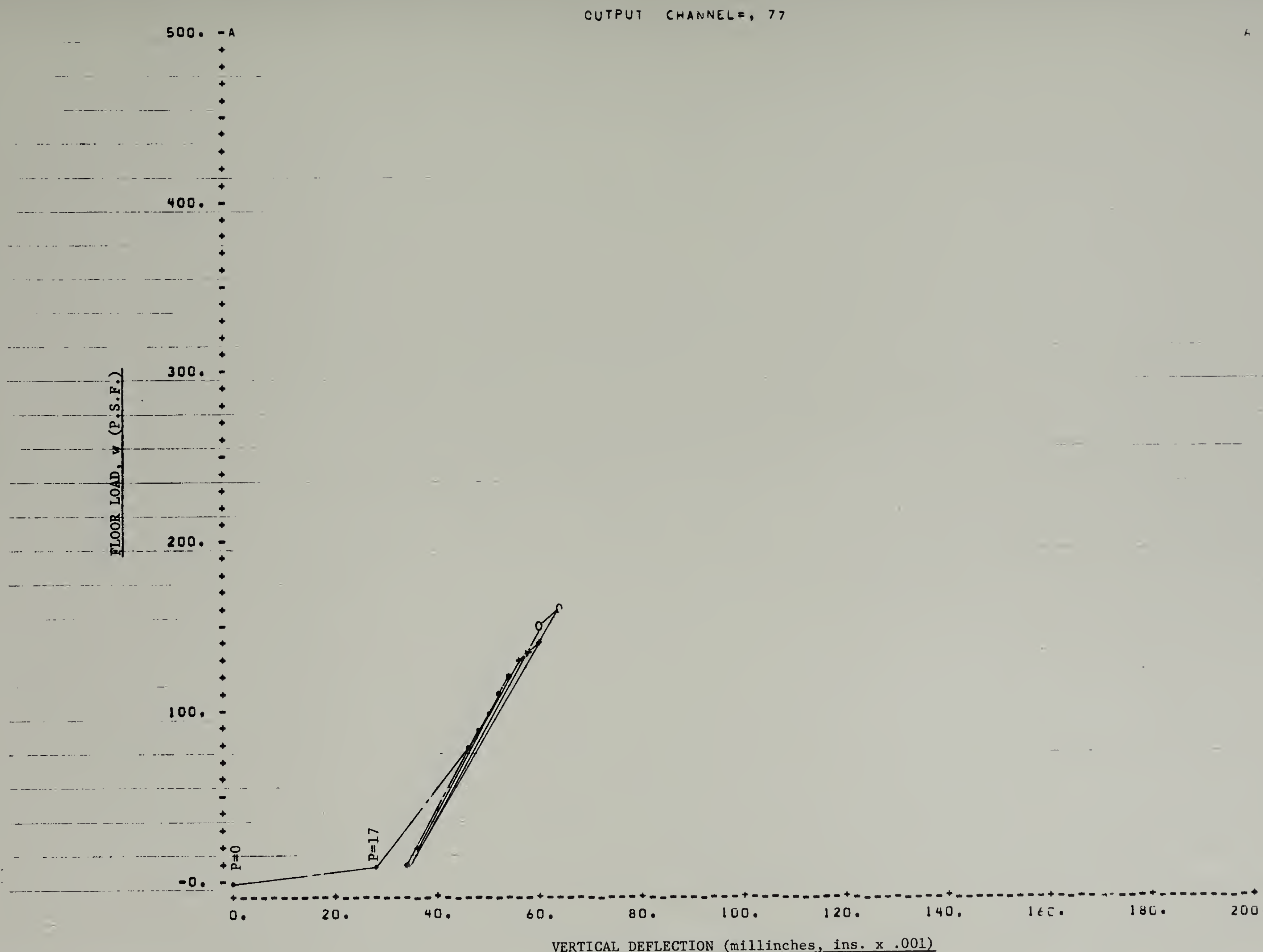


FIGURE NO. A.31 - TEST NO. 9, MAJOR FLOOR LOAD VS. BEAM DEFLECTION

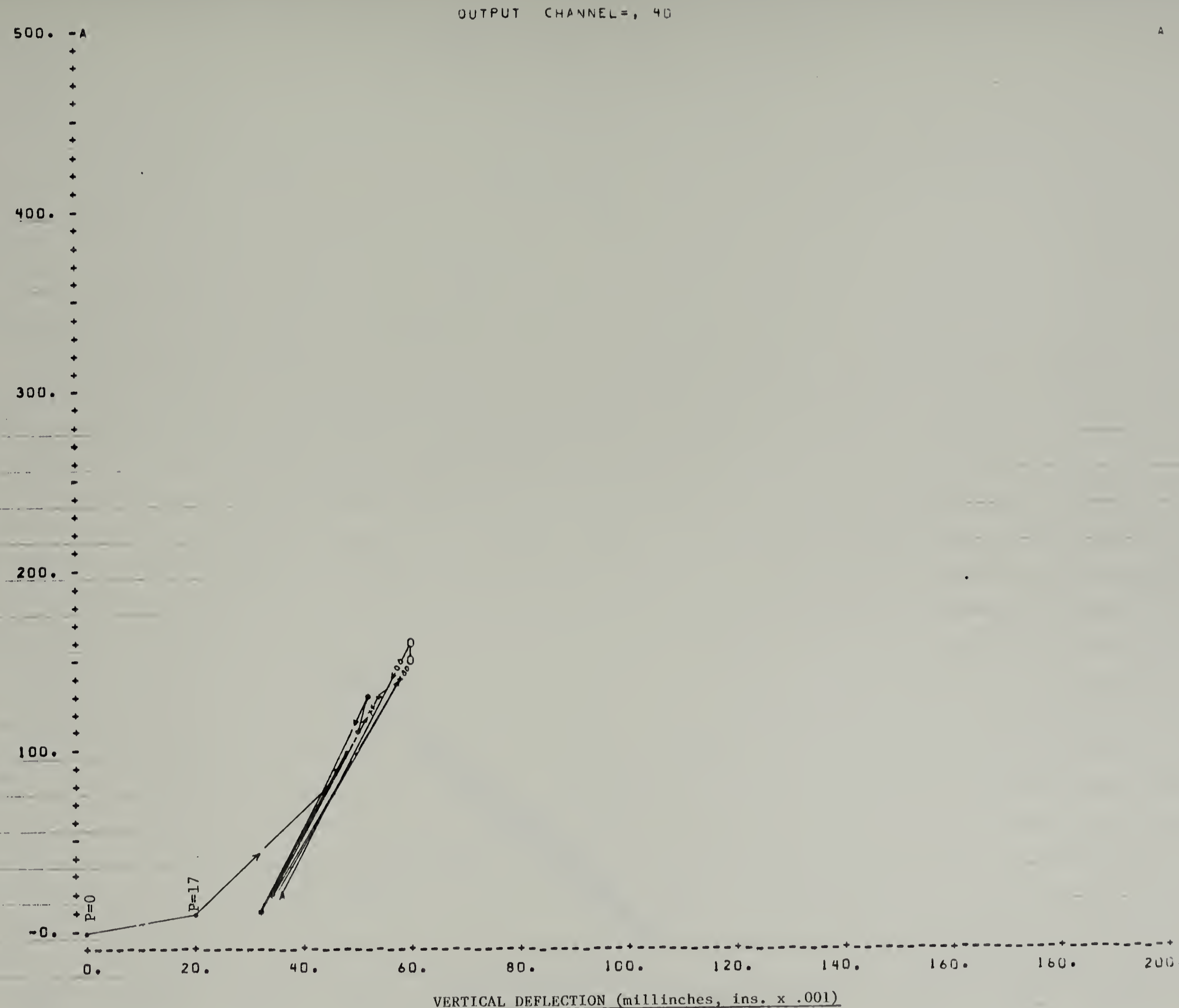


FIGURE NO. A.32 - TEST NO. 9A, FLOOR LOADS VS. BEAM DEFLECTION

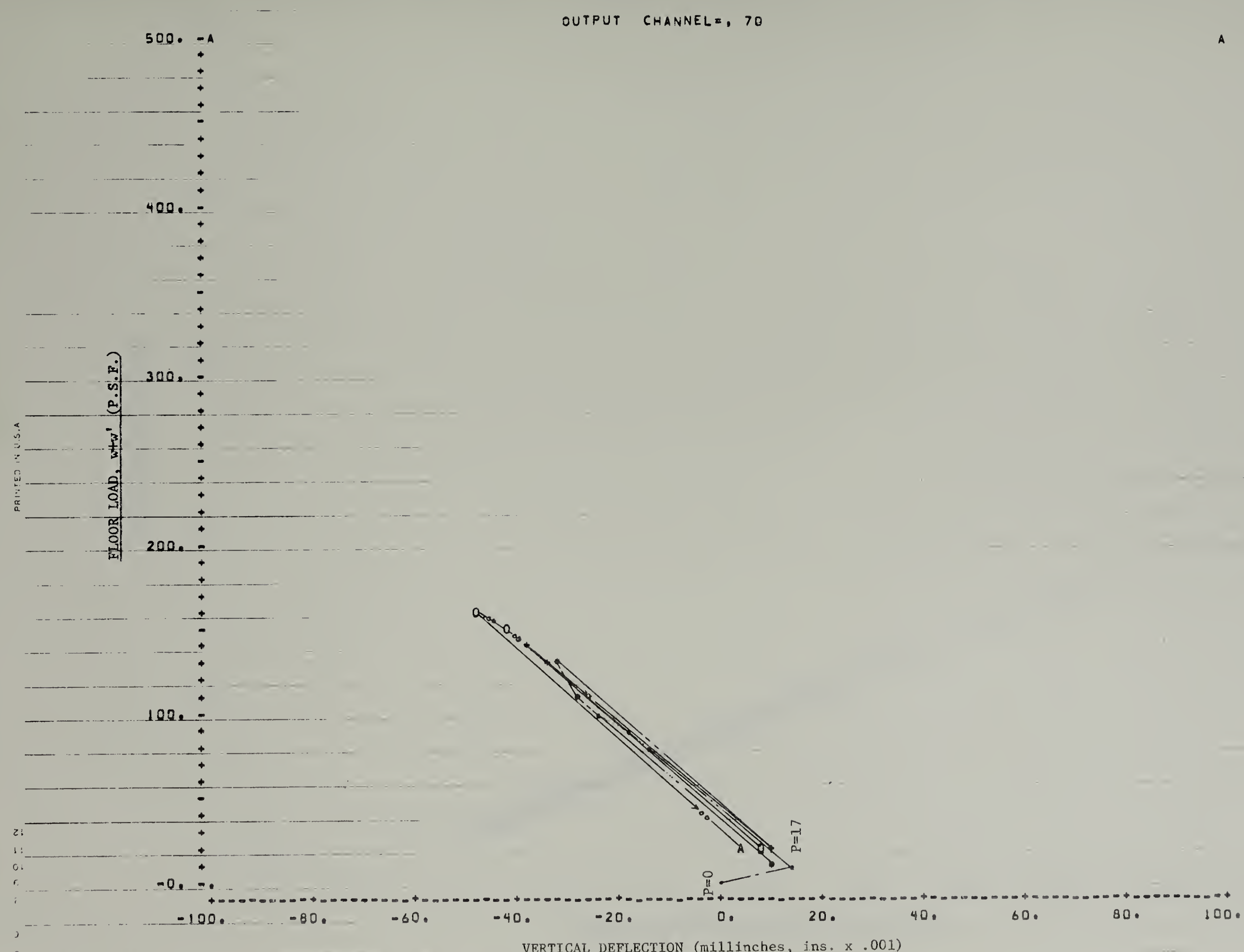


FIGURE NO. A.33 - TEST NO. 9A, FLOOR LOADS VS. BEAM DEFLECTION

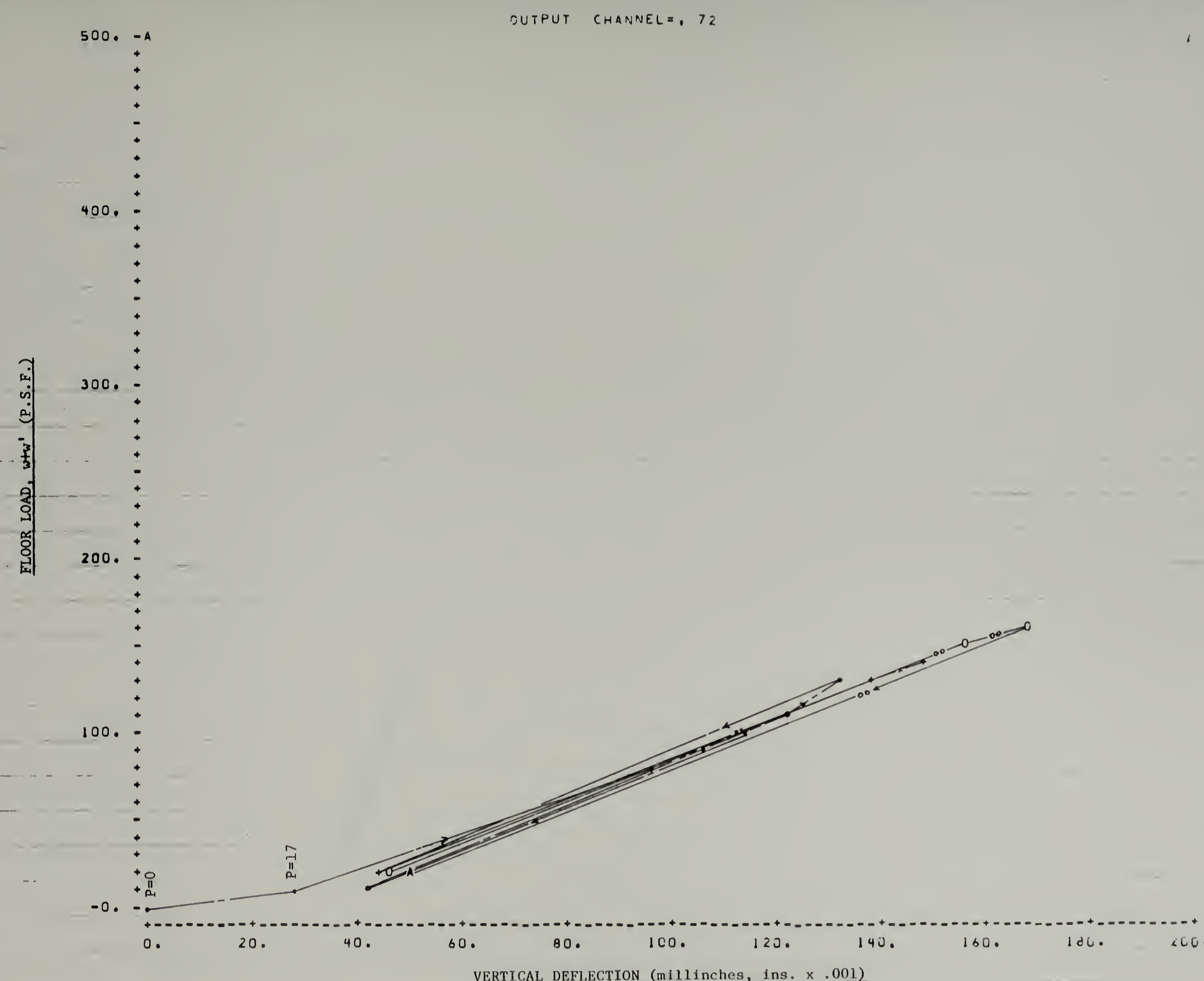


FIGURE NO. A.34 - TEST NO. 9A, FLOOR LOADS VS. BEAM DEFLECTION

OUTPUT CHANNEL=, 73

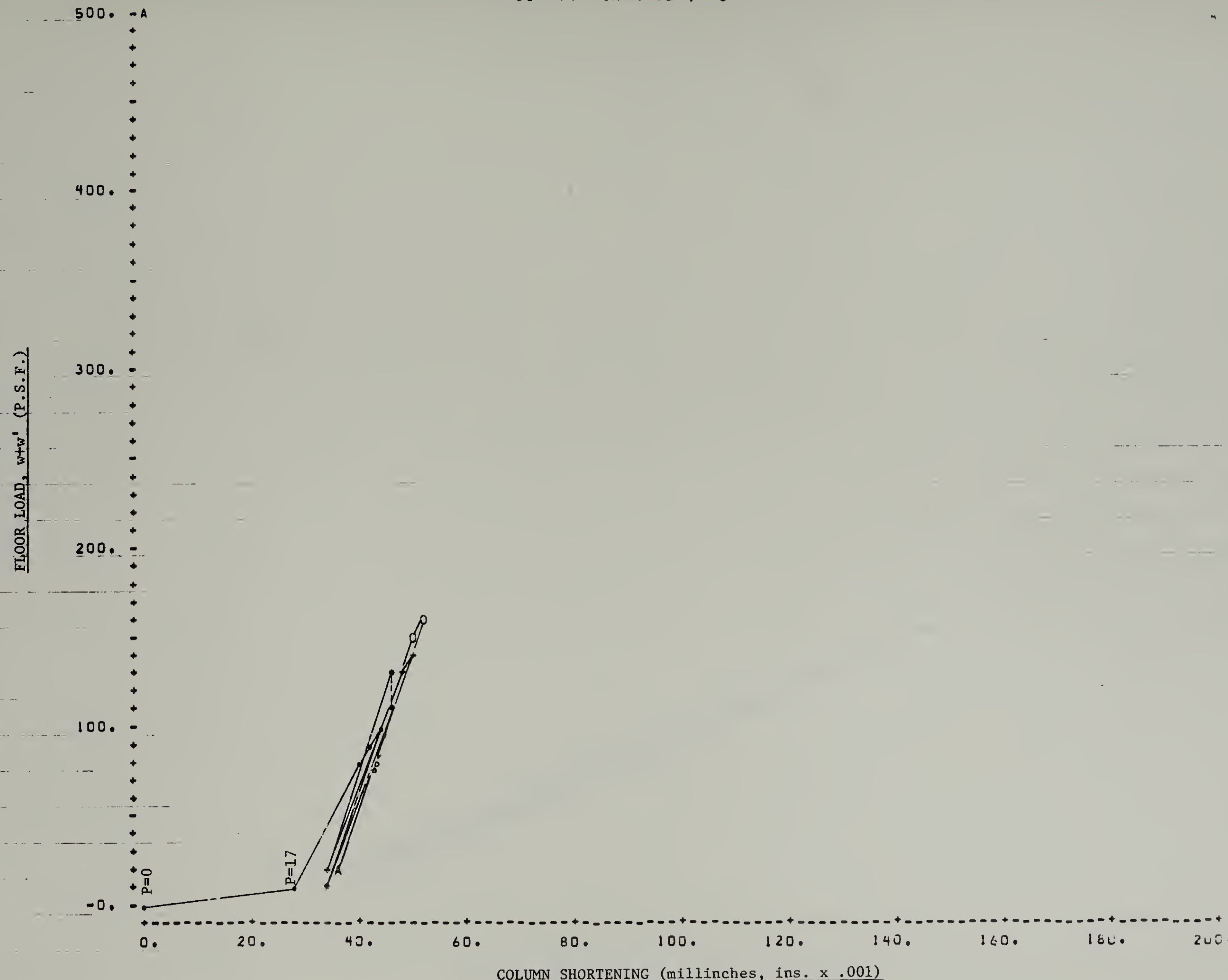


FIGURE NO. A.35 - TEST NO. 9A, FLOOR LOAD VS. COLUMN SHORTENING

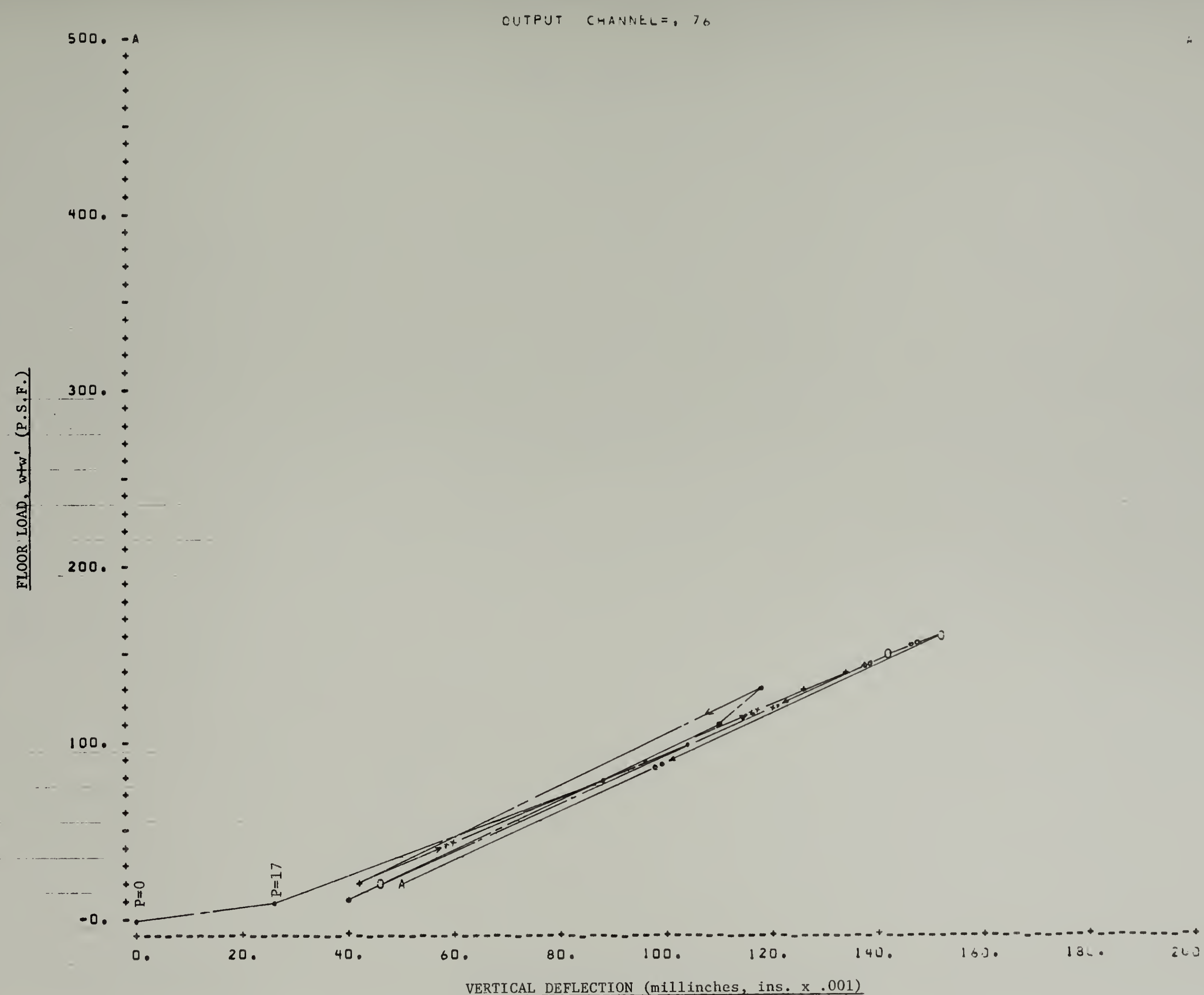


FIGURE NO. A.36 - TEST NO. 9A, FLOOR LOAD VS. SLAB DEFLECTION

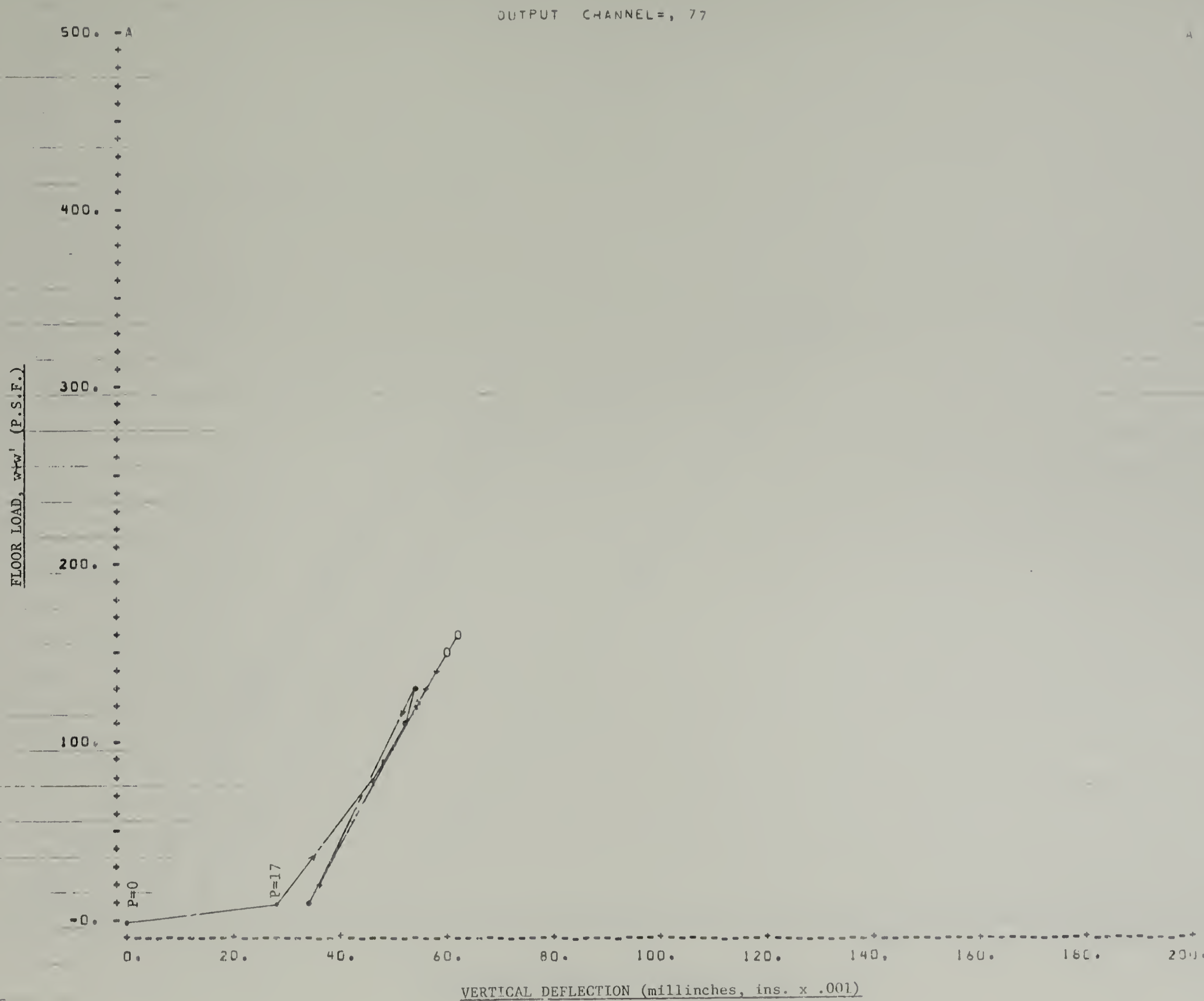


FIGURE NO. A.37 - TEST NO. 9A, FLOOR LOAD VS. BEAM DEFLECTION

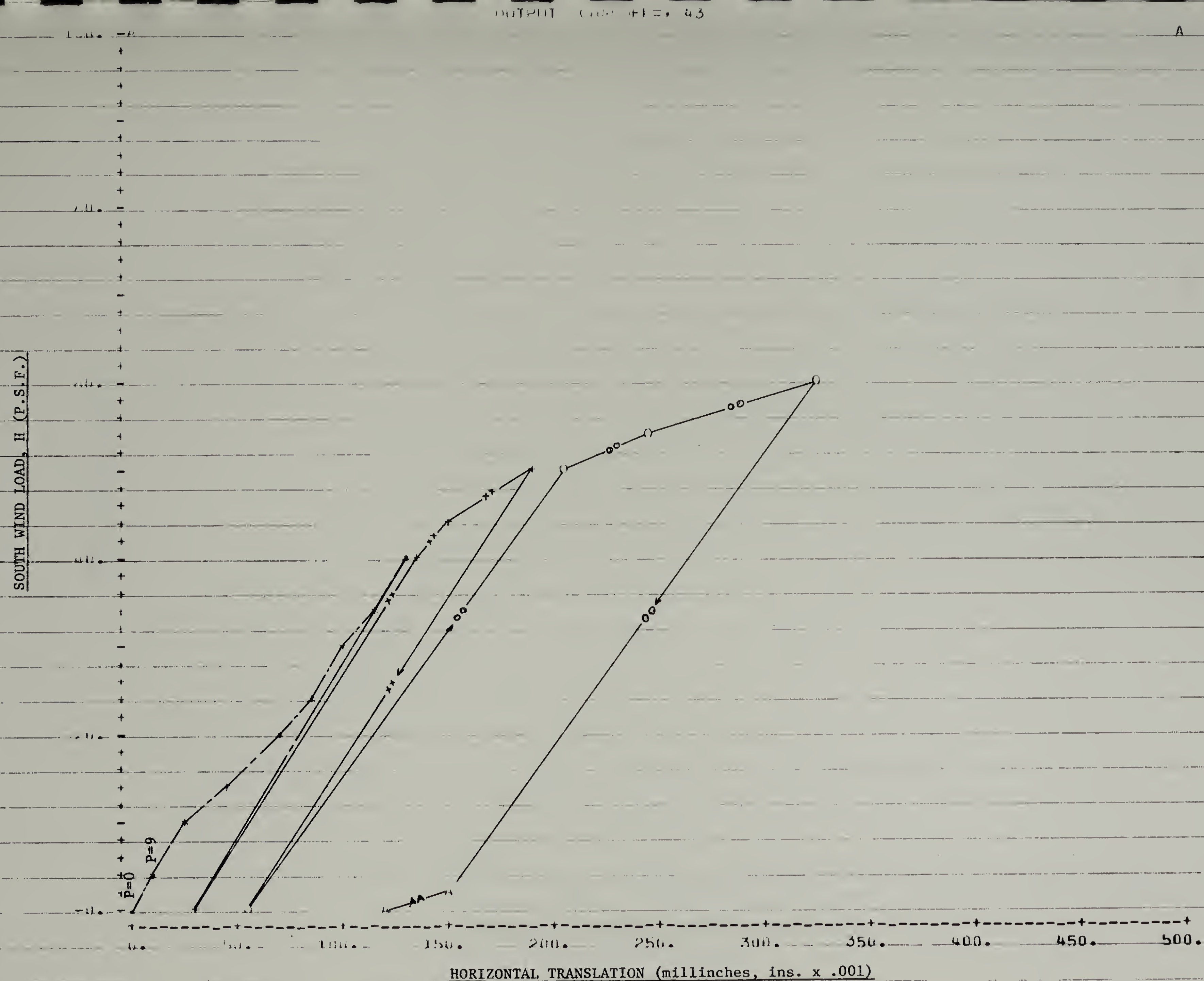


FIGURE NO. A.38 - TEST NO. 10, SOUTH WIND LOAD VS. TRANSLATION

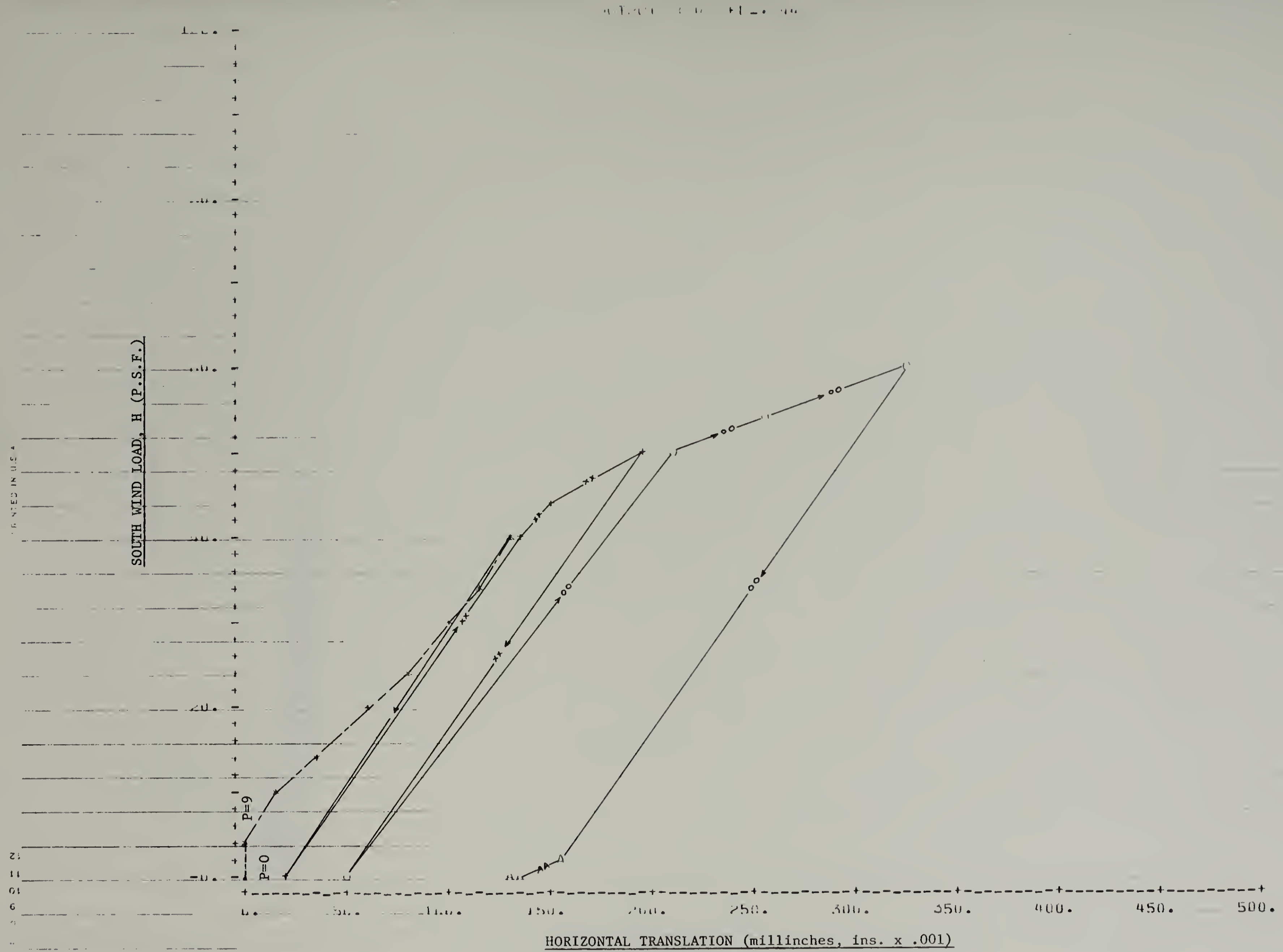


FIGURE NO. A.39 - TEST NO. 10, SOUTH WIND LOAD VS. TRANSLATION

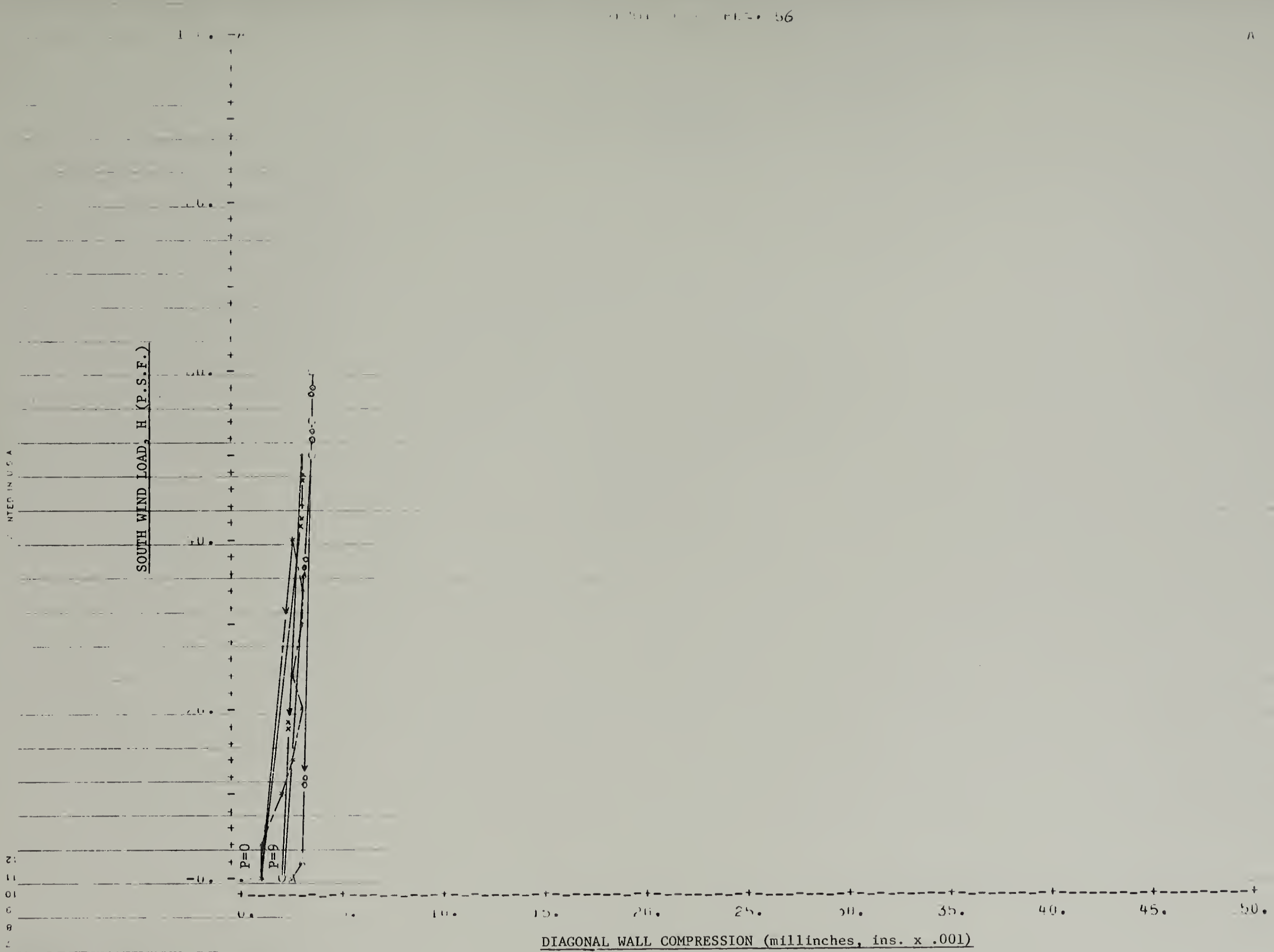


FIGURE NO. A.40 - TEST NO. 10, SOUTH WIND LOAD VS. WALL COMPRESSION

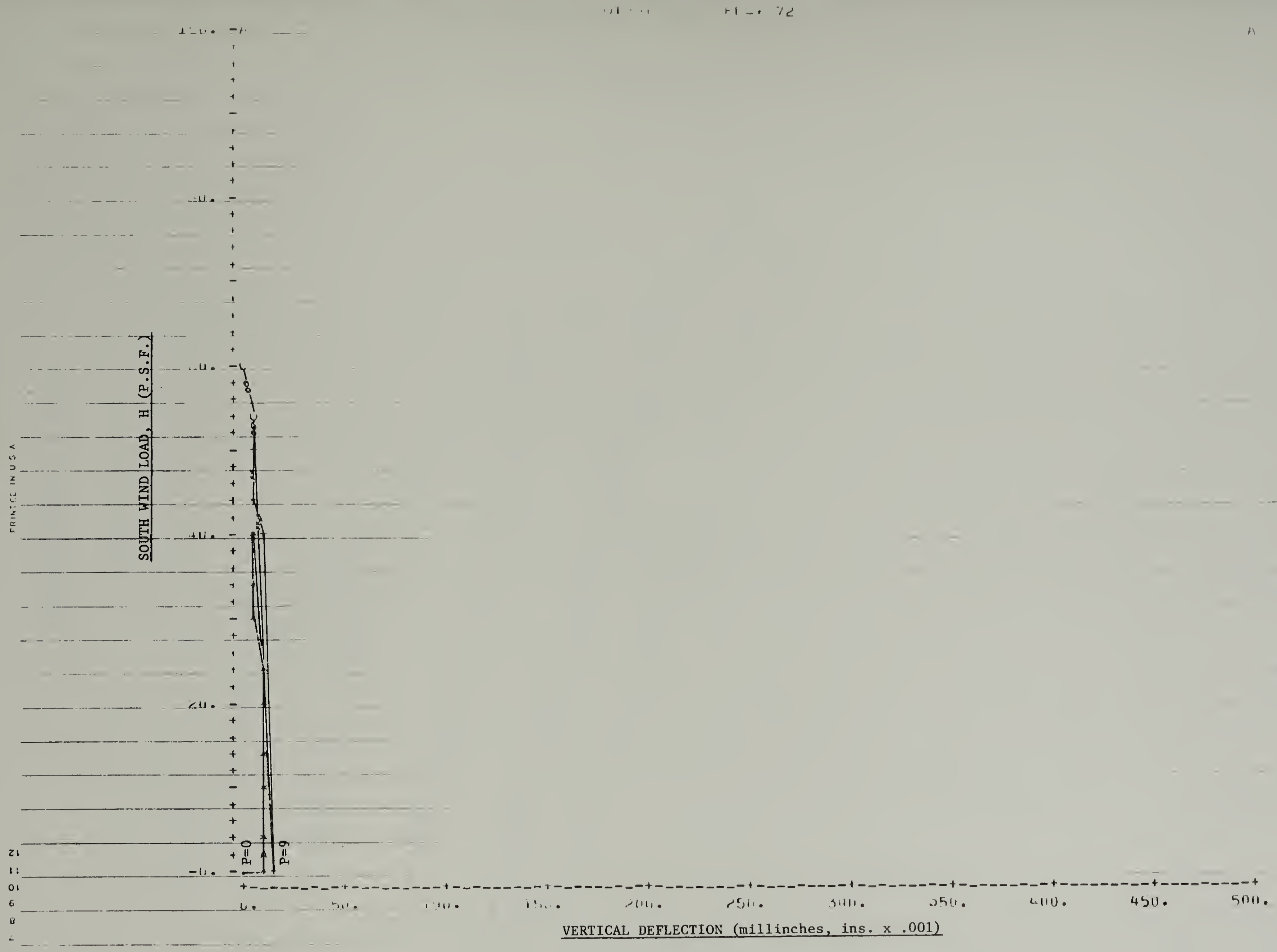


FIGURE NO. A.42 - TEST NO. 10, SOUTH WIND LOAD VS. BEAM DEFLECTION

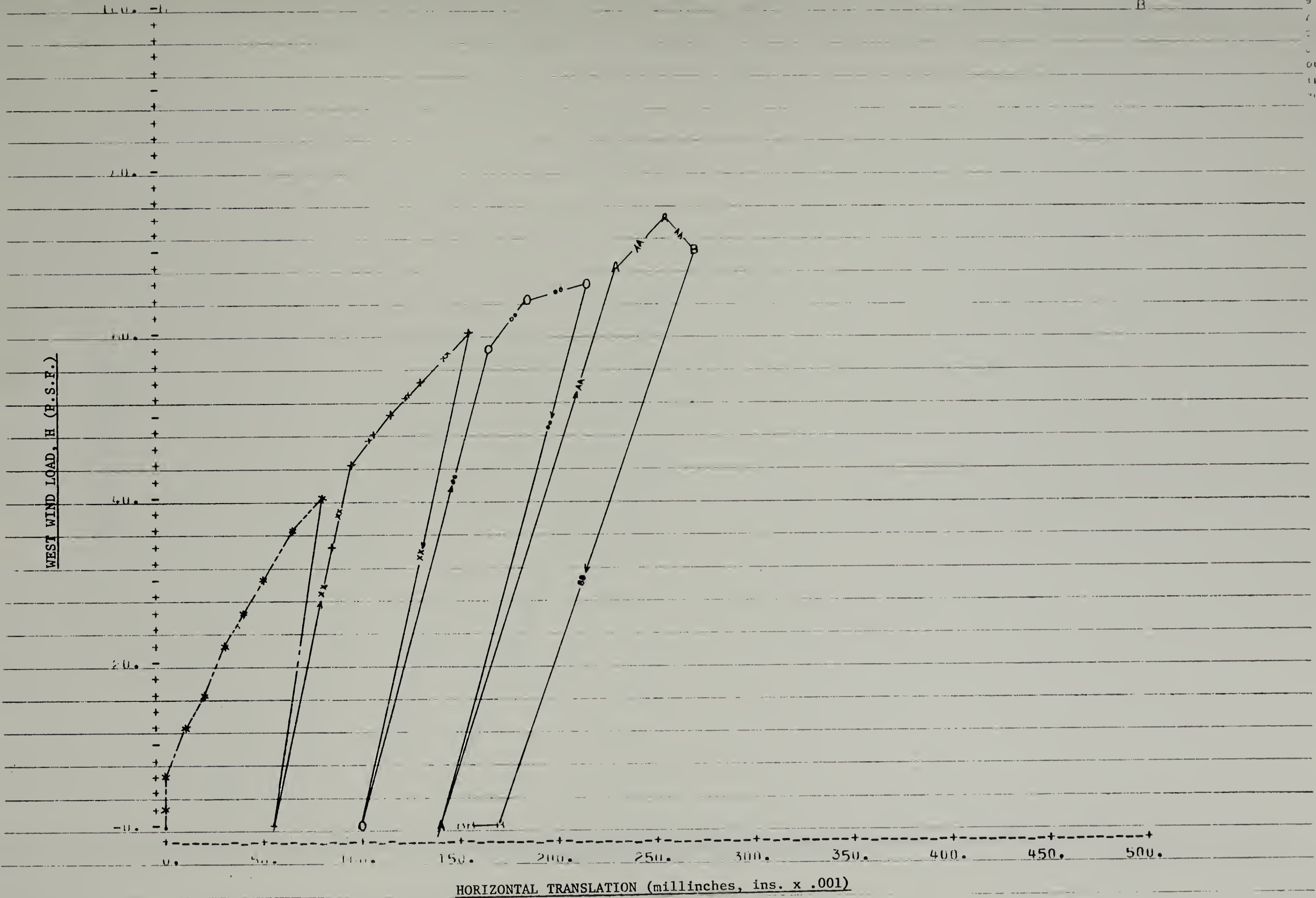


FIGURE NO. A.43 - TEST NO. 11, WEST WIND LOAD VS. TRANSLATION

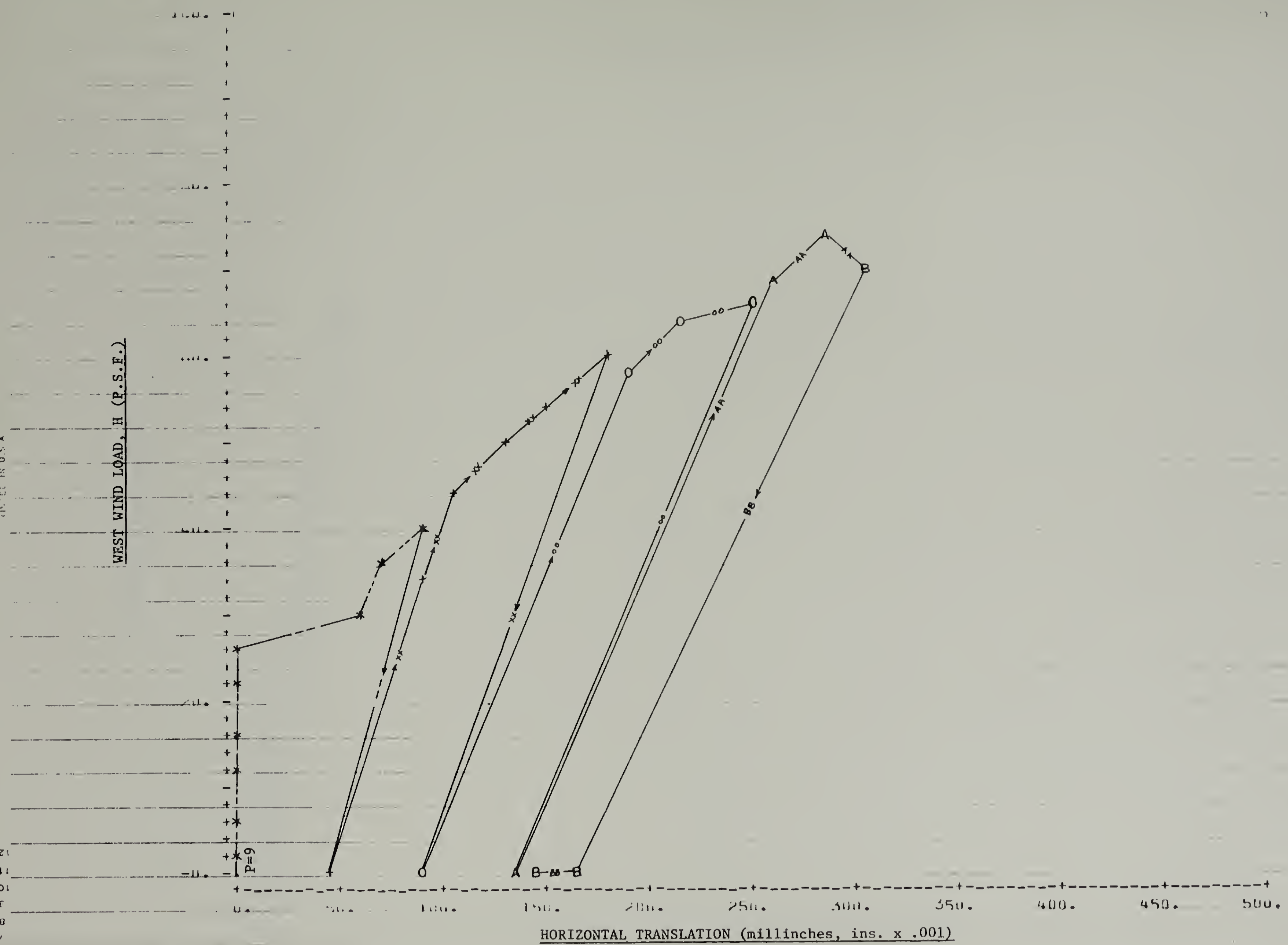


FIGURE NO. A.44 - TEST NO. 11, WEST WIND LOAD VS. TRANSLATION

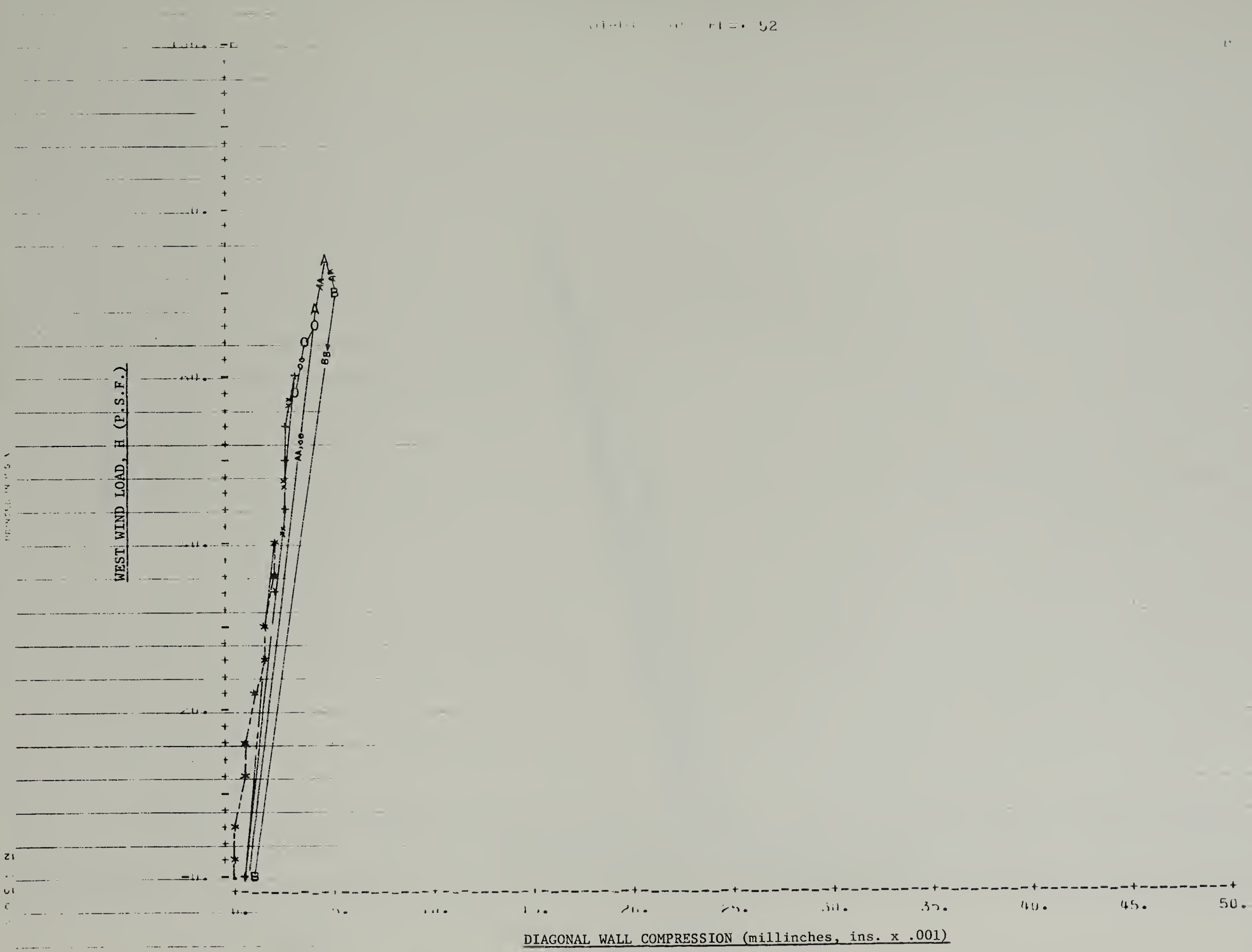


FIGURE NO. A.45 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

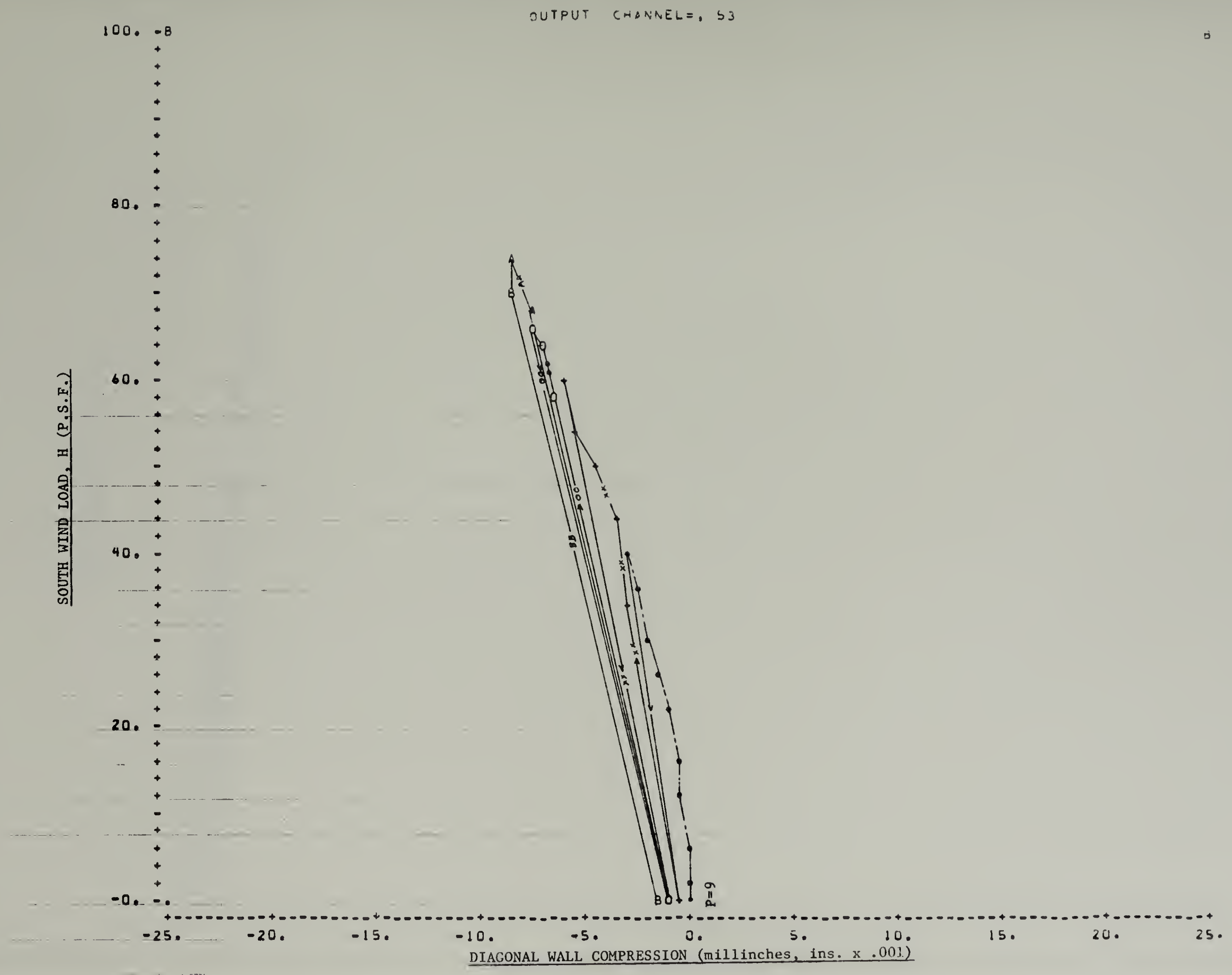


FIGURE NO. A.46 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

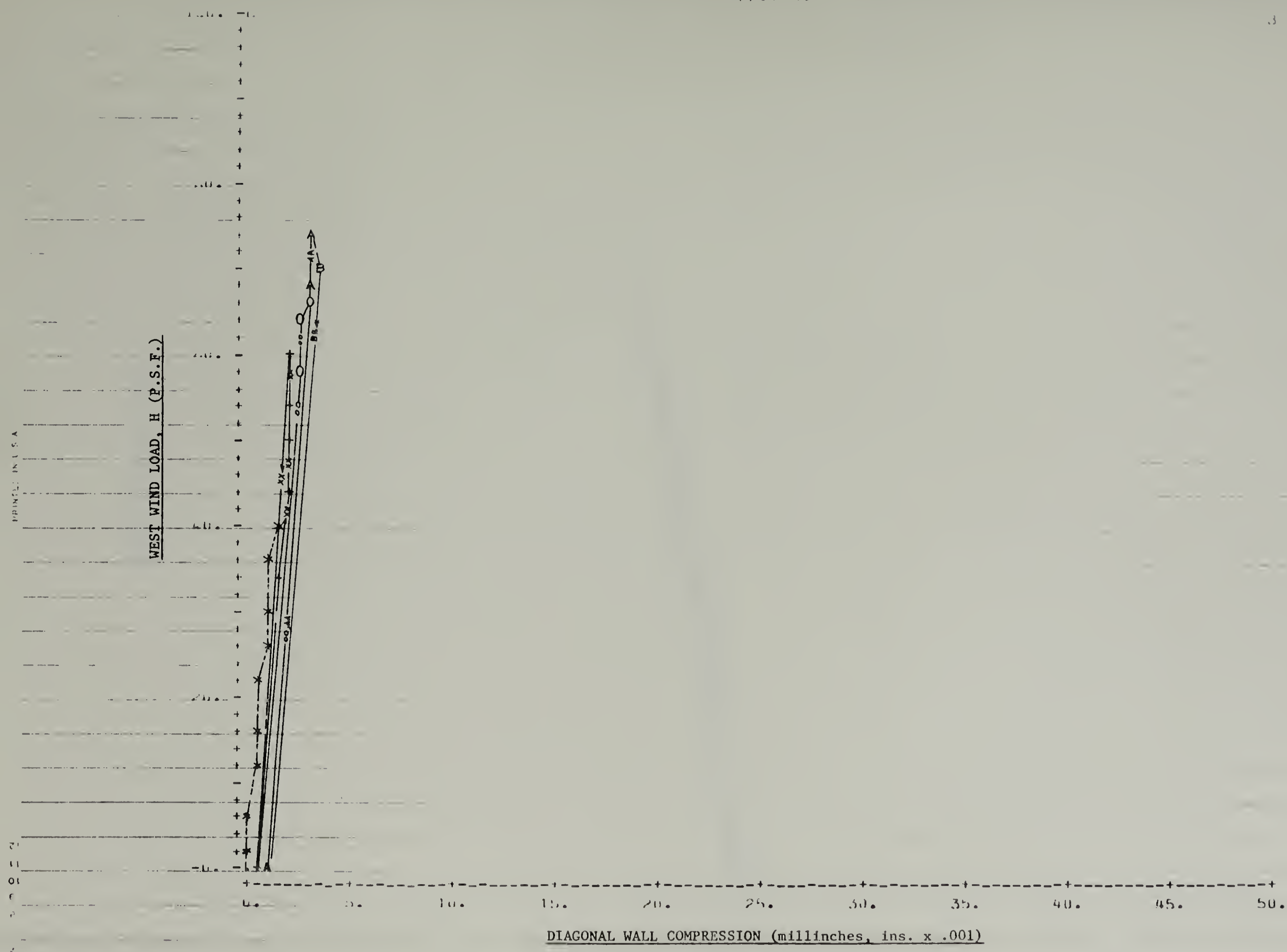


FIGURE NO. A.47, TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

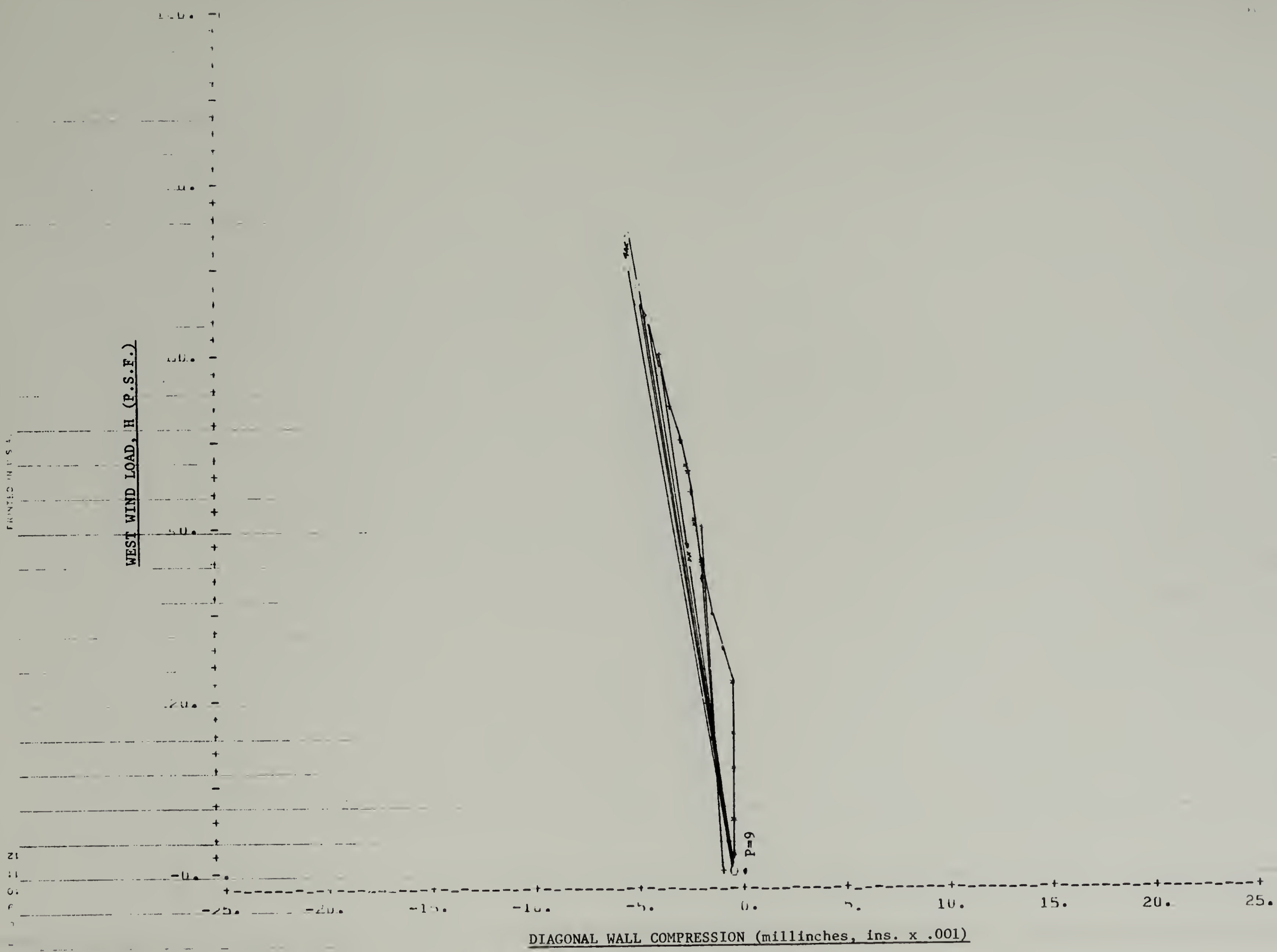


FIGURE NO. A-48 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

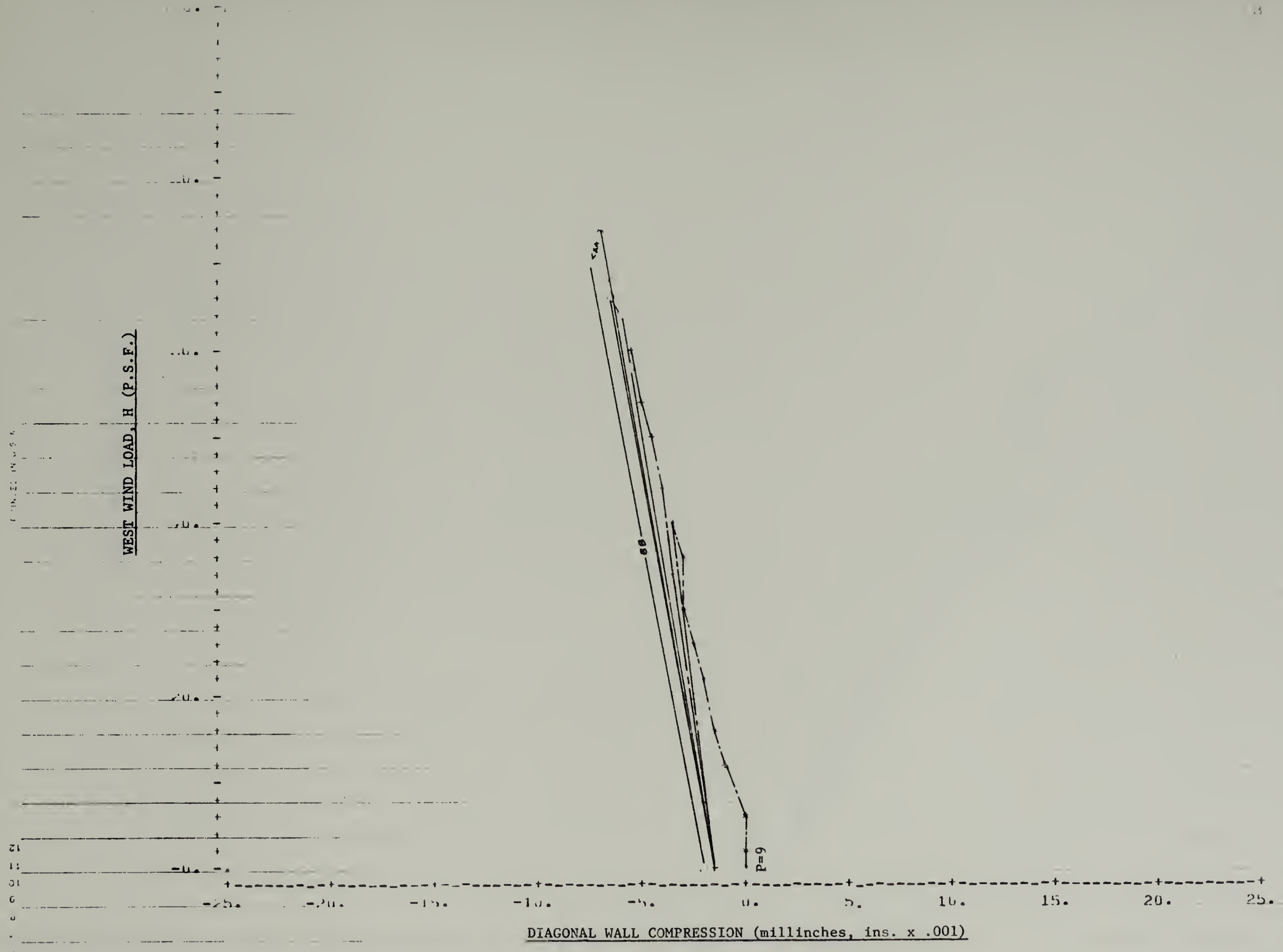


FIGURE NO. A.49 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

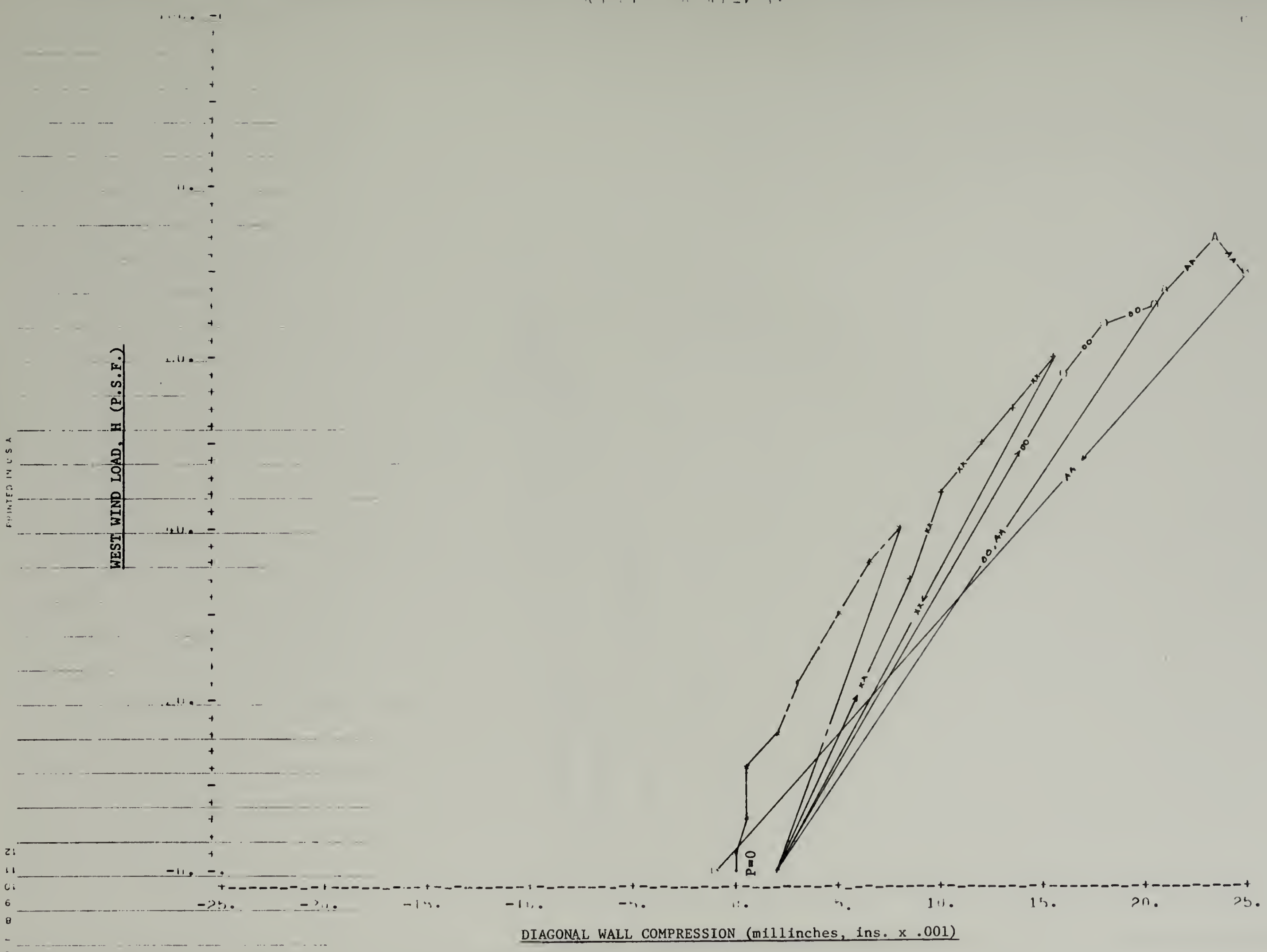


FIGURE NO. A.50 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

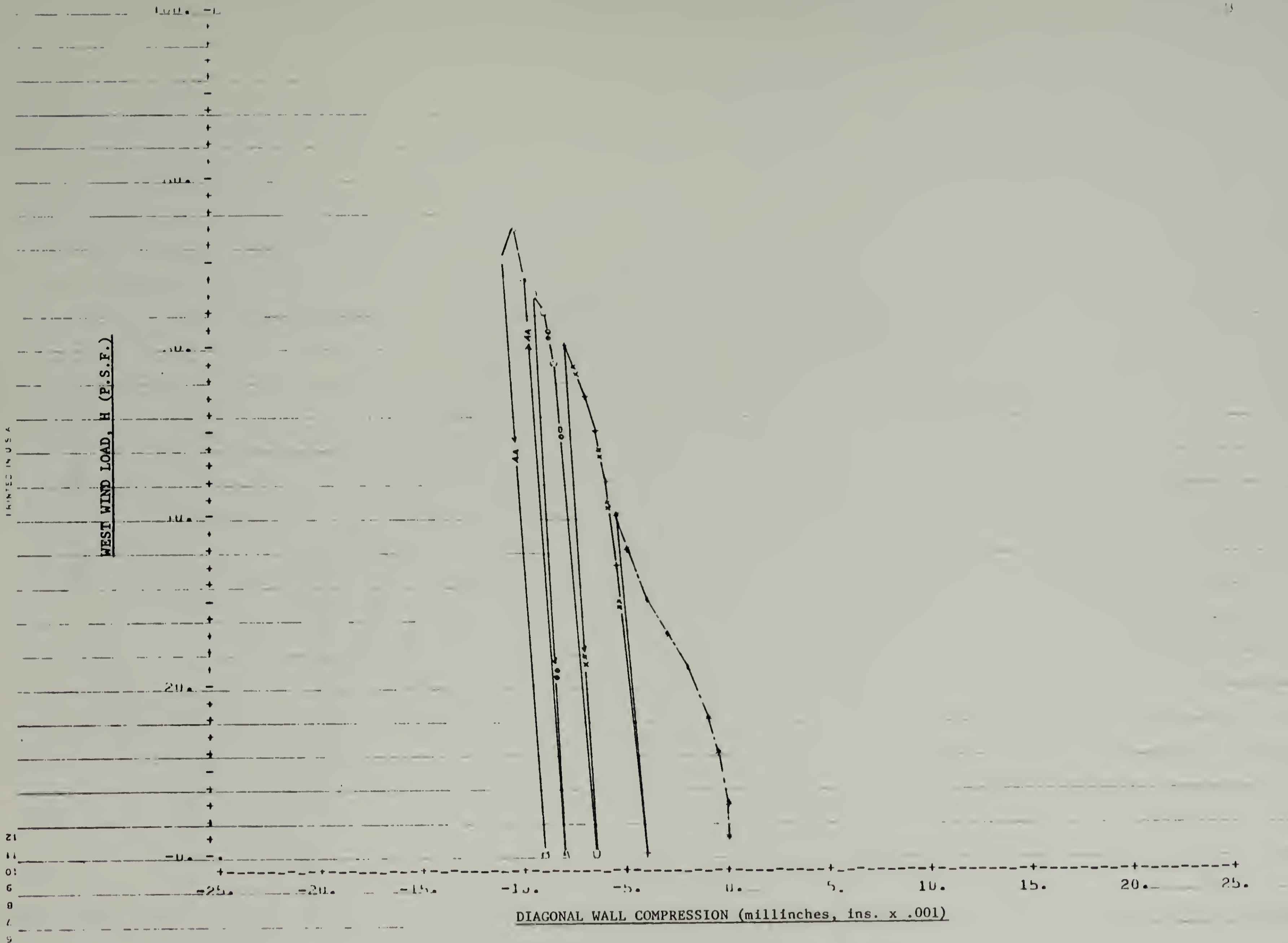


FIGURE NO. A.51 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

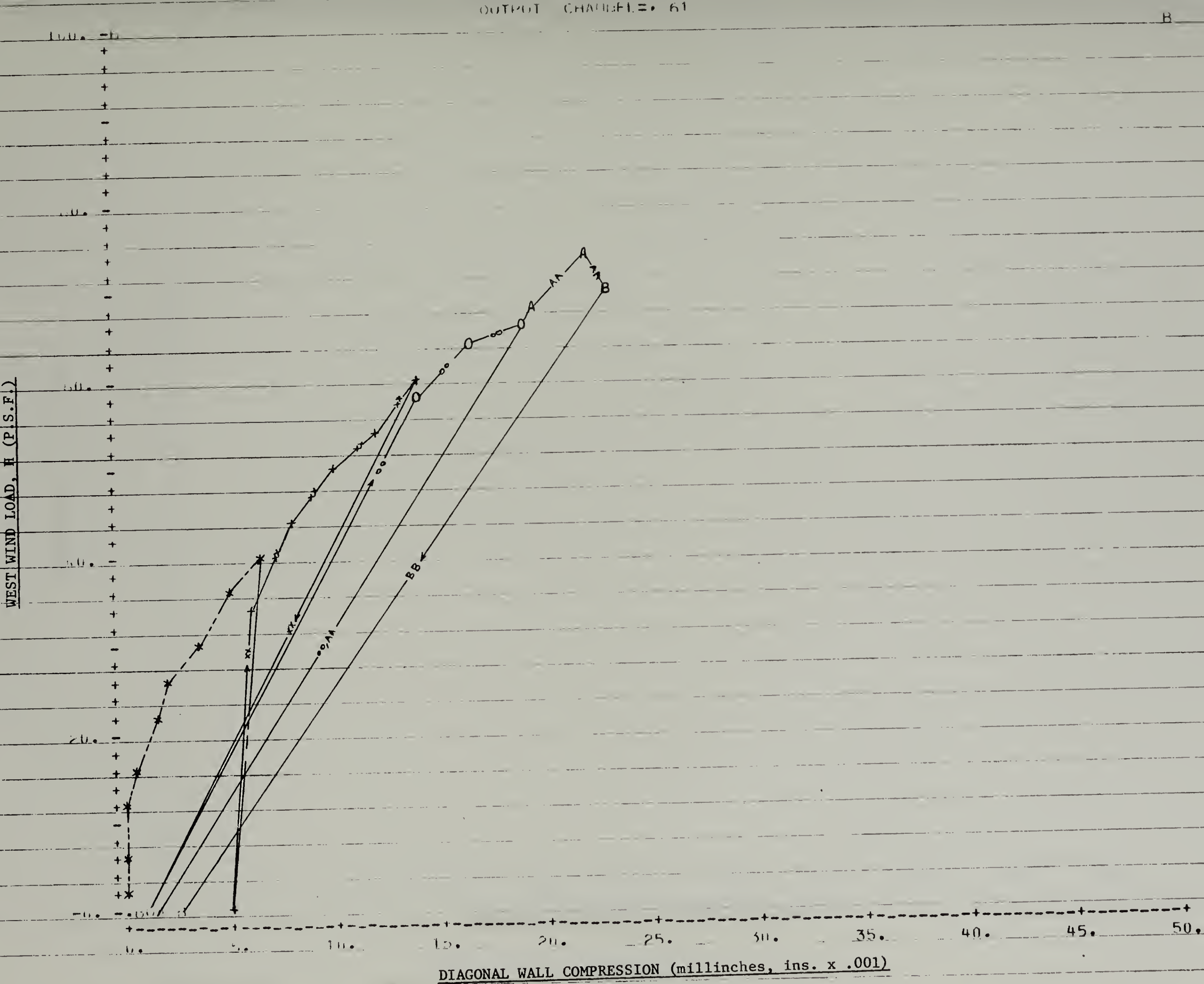


FIGURE NO. A.52 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

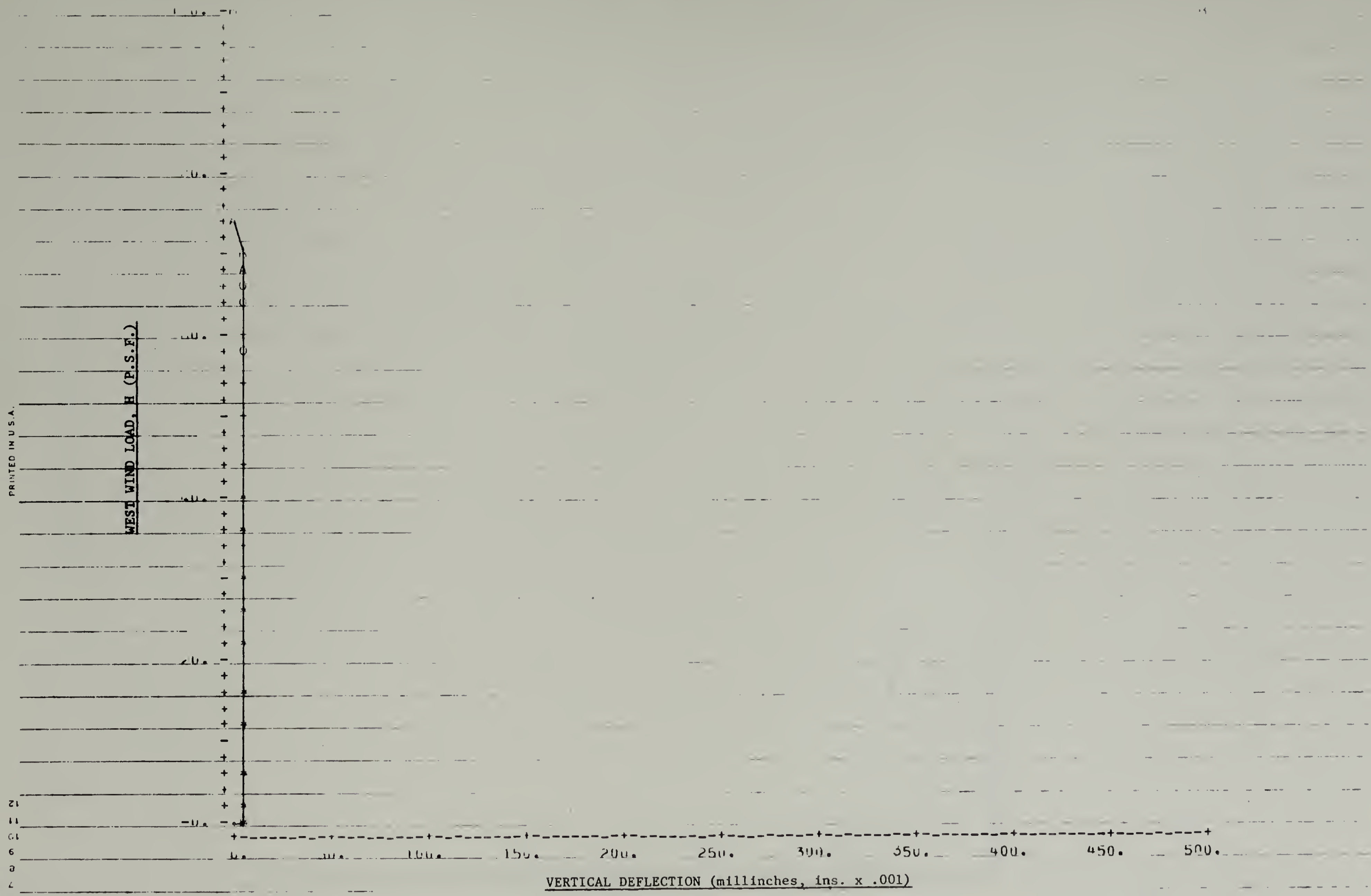


FIGURE NO. A.53 - TEST NO. 11, WEST WIND LOAD VS. BEAM DEFLECTION

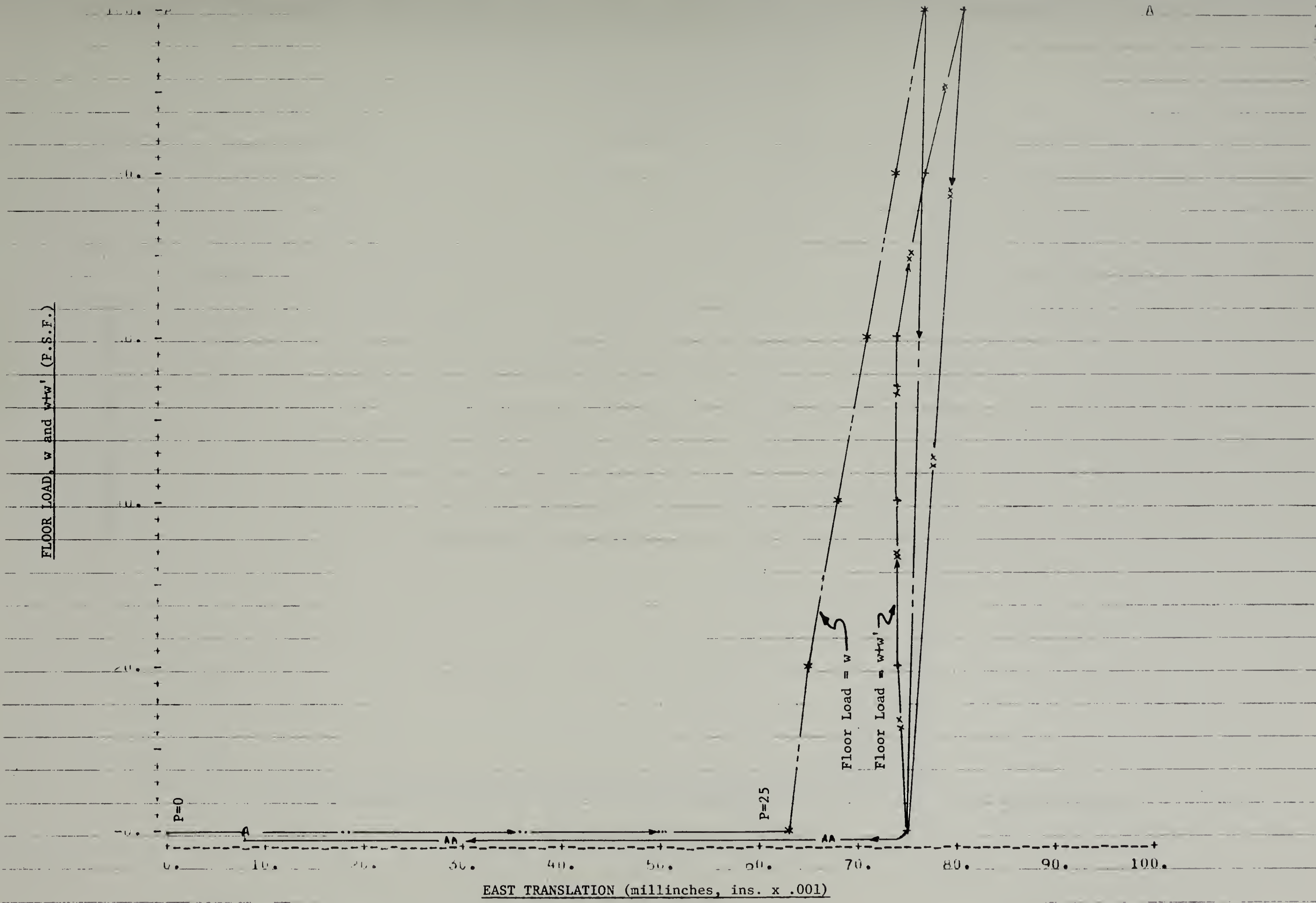


FIGURE NO. A.54 - TEST NO. 12 & 13, VERTICAL LOAD VS. EAST TRANSLATION

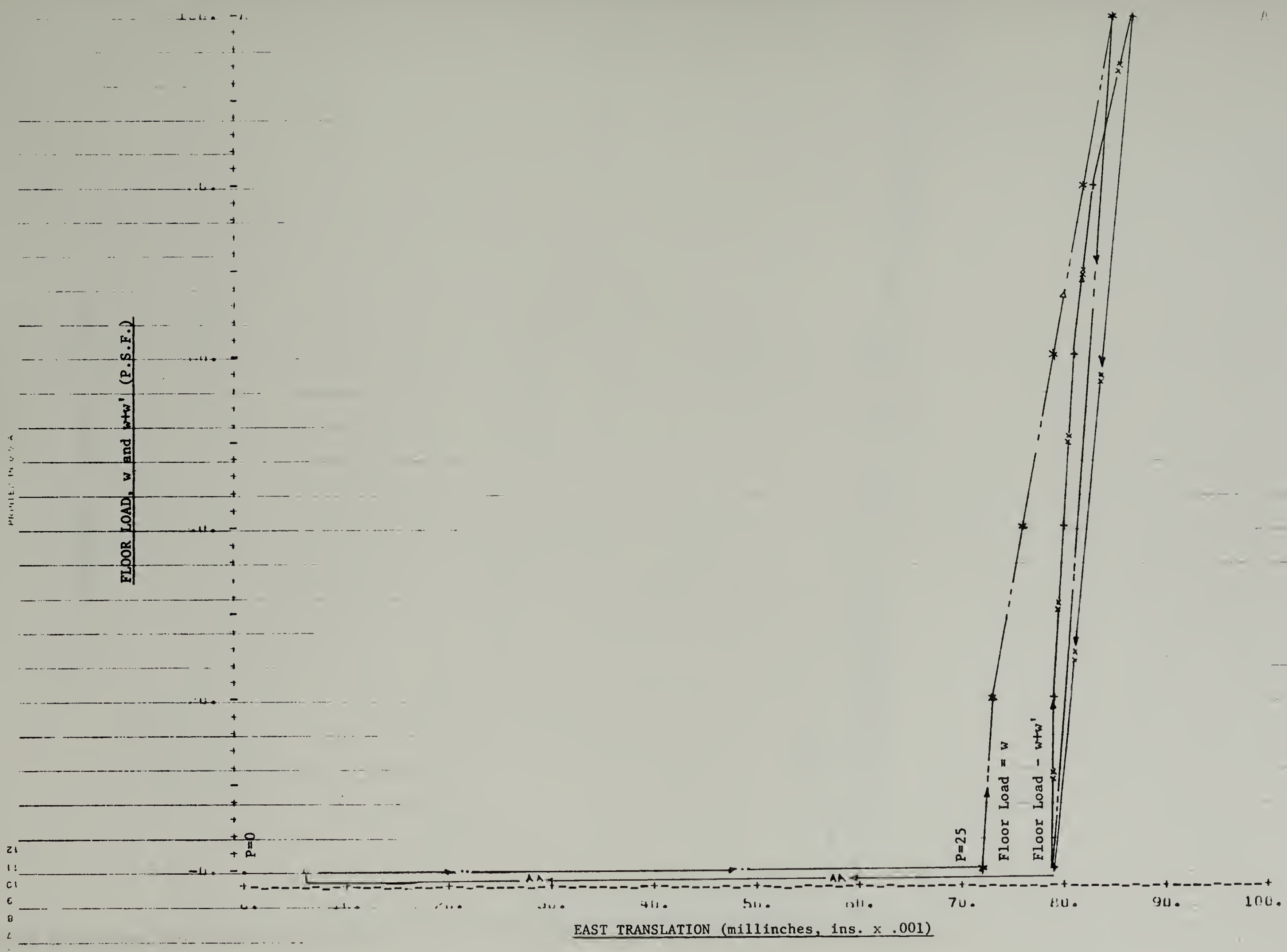


FIGURE NO. A.55 - TEST NO. 12 & 13, VERTICAL LOAD VS. EAST TRANSLATION

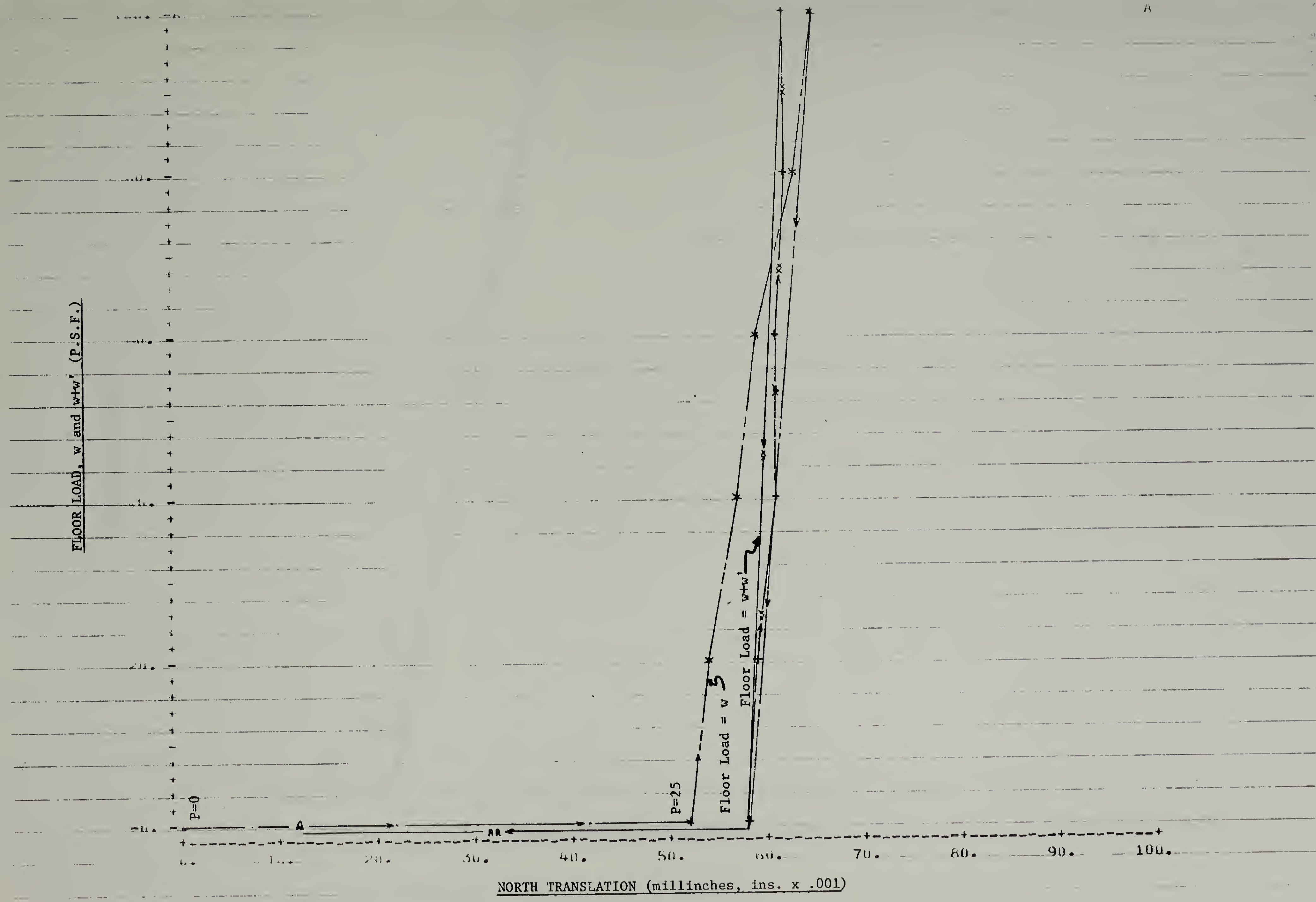


FIGURE NO. A.56 - TEST NO. 12A and 13A, VERTICAL LOAD VS. NORTH TRANSLATION

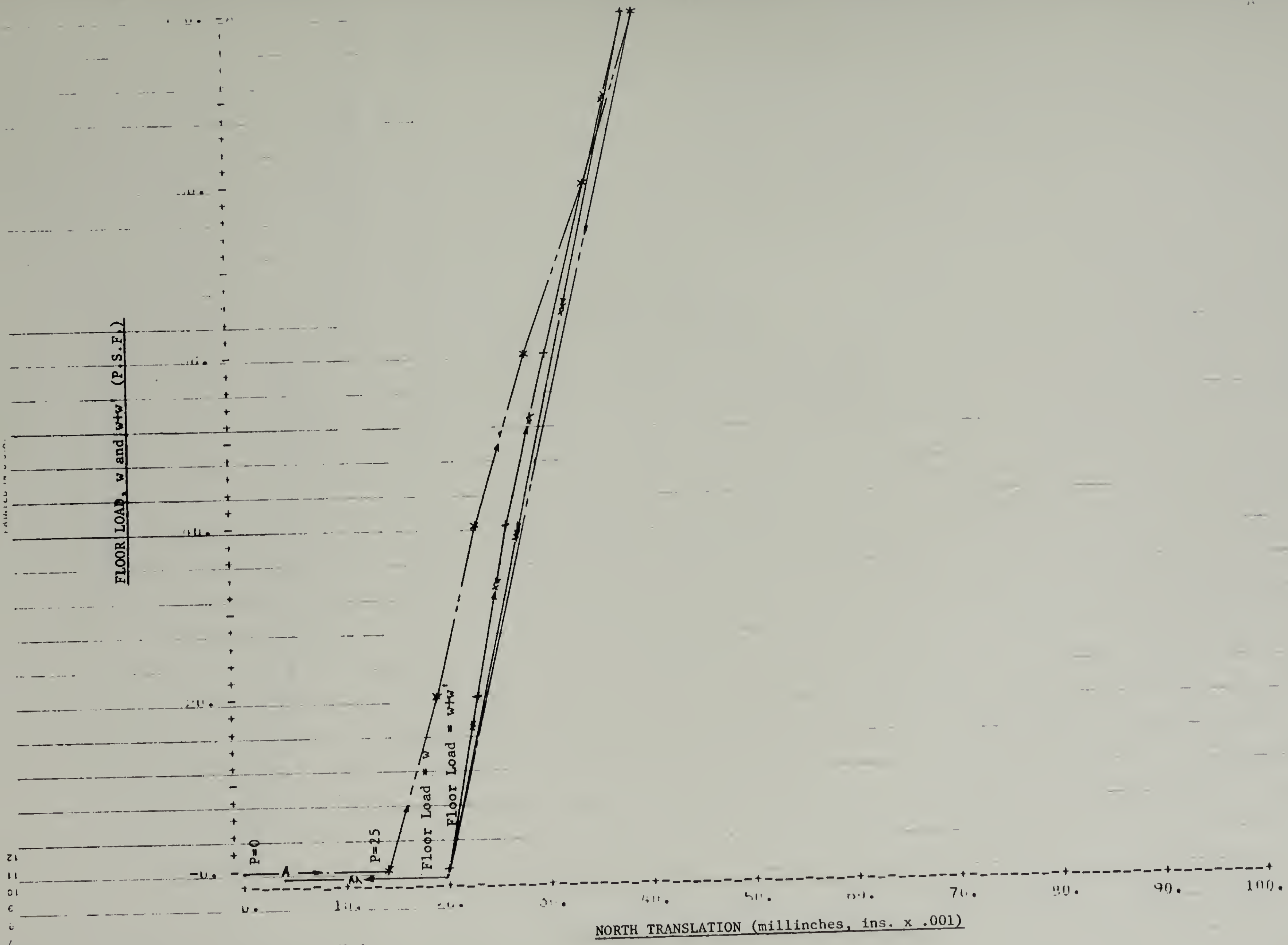


FIGURE NO. A.57 - TEST NO. 12A and 13A, VERTICAL LOAD VS. NORTH TRANSLATION

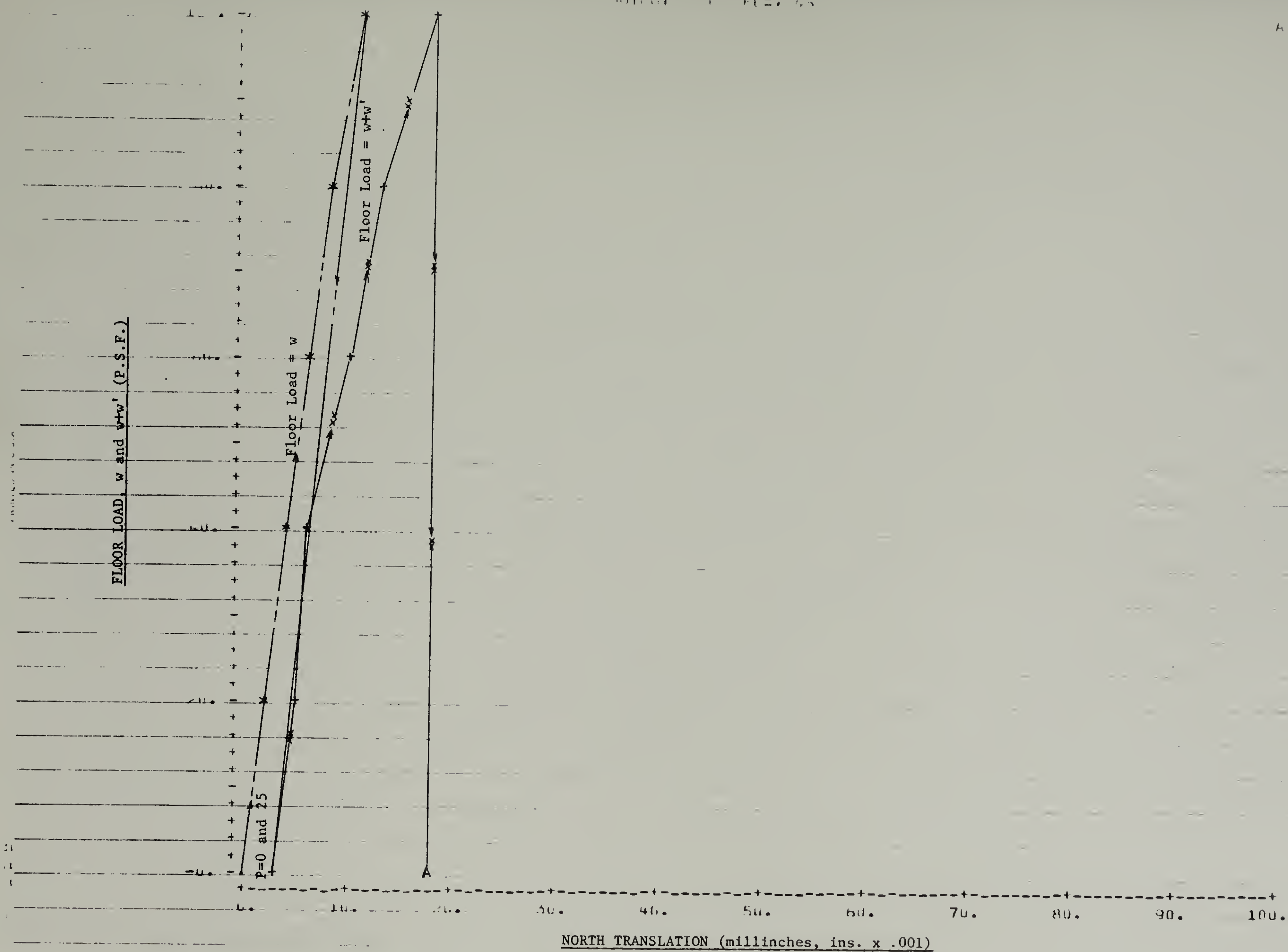


FIGURE NO. A.58 - TEST NO. 12A AND 13A, VERTICAL LOAD VS. NORTH TRANSLATION

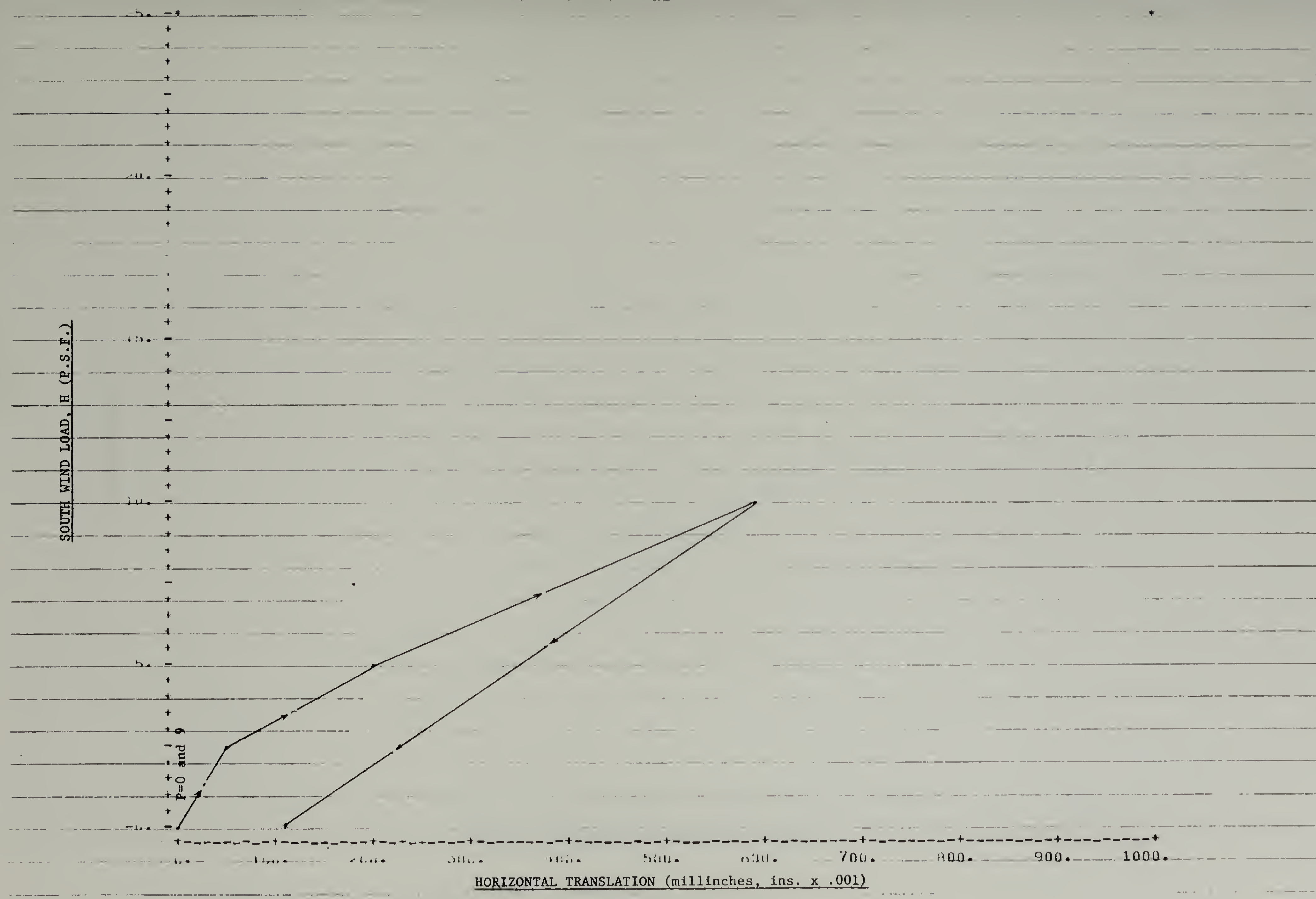


FIGURE NO. A.59 - TEST NO. 14, SOUTH WIND LOAD VS. TRANSLATION

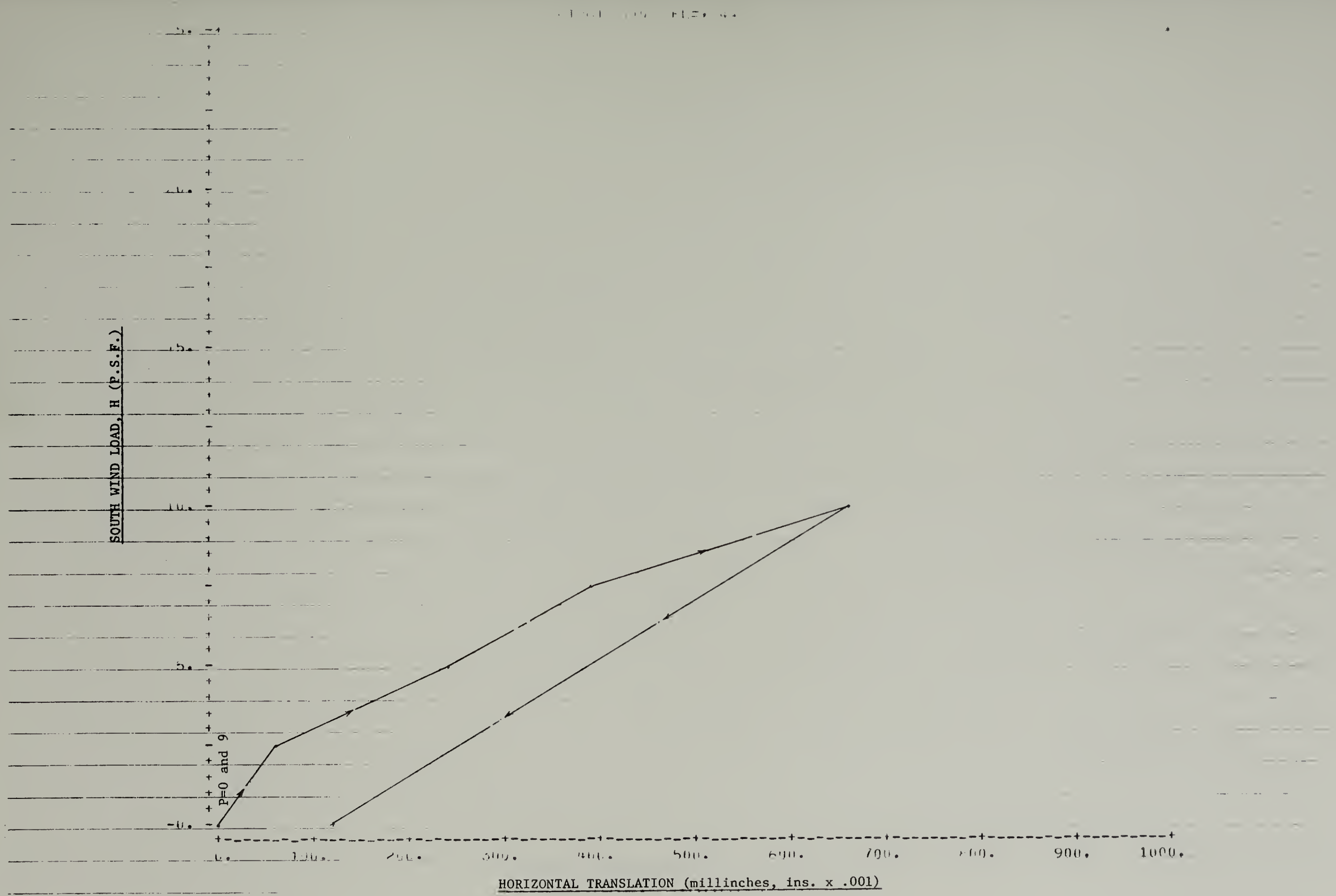


FIGURE NO. A.60 - TEST NO. 14, SOUTH WIND LOAD VS. TRANSLATION

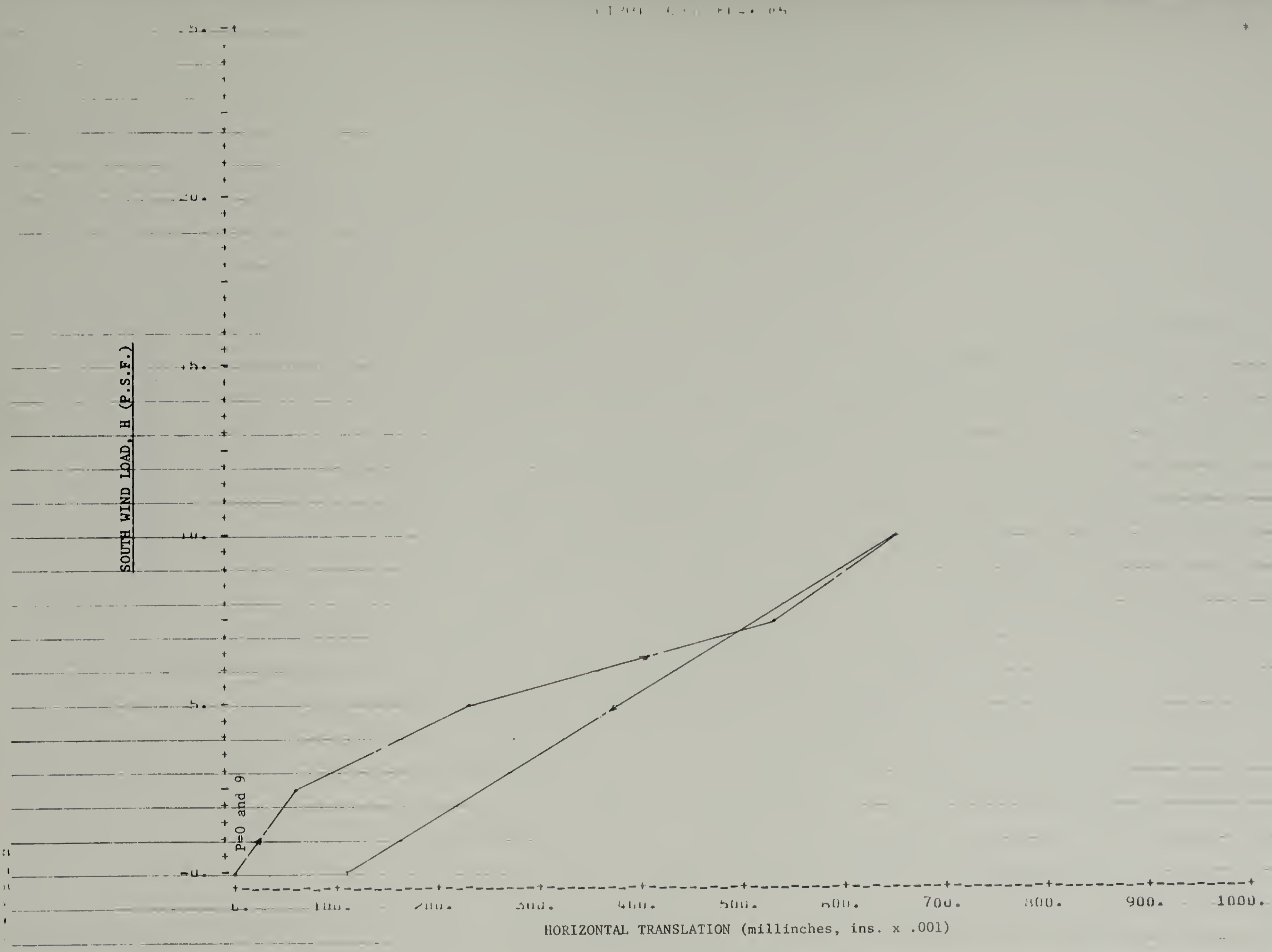


FIGURE NO. A.61 - TEST NO. 14, SOUTH WIND LOAD VS. TRANSLATION

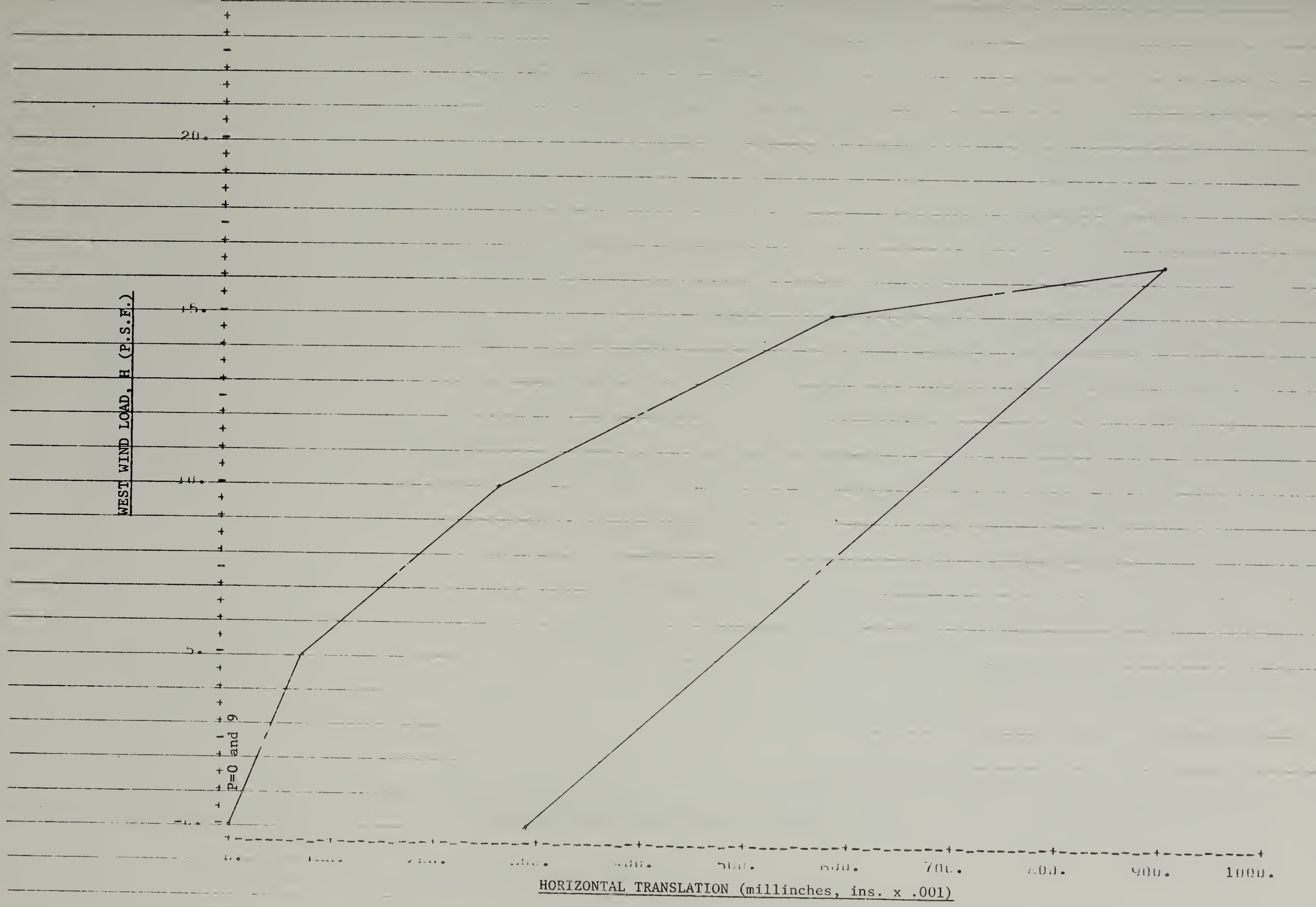


FIGURE NO. A.62 - TEST NO. 15, WEST WIND LOAD VS. TRANSLATION

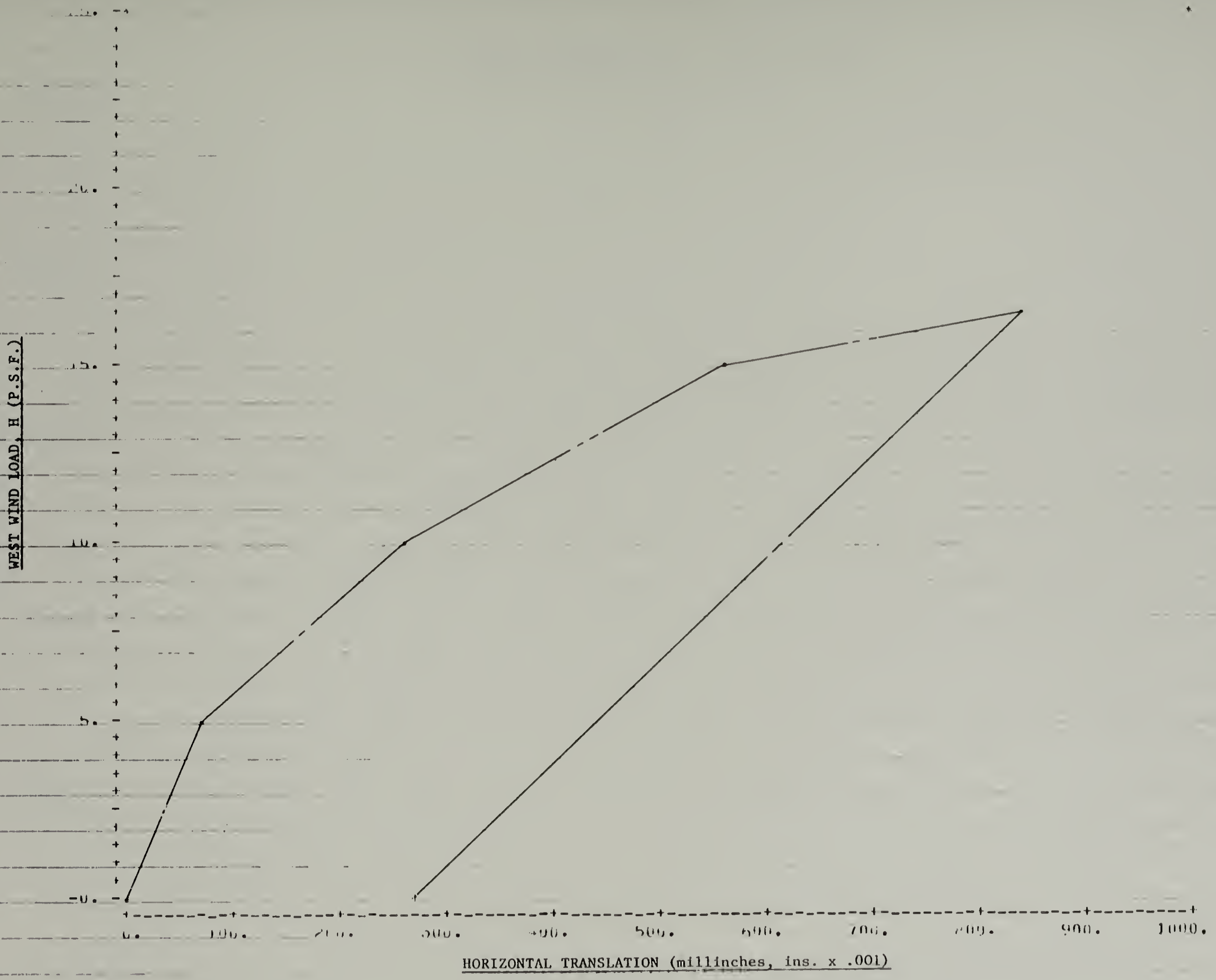


FIGURE NO. A.63 - TEST NO. 15, WEST WIND LOAD VS. TRANSLATION

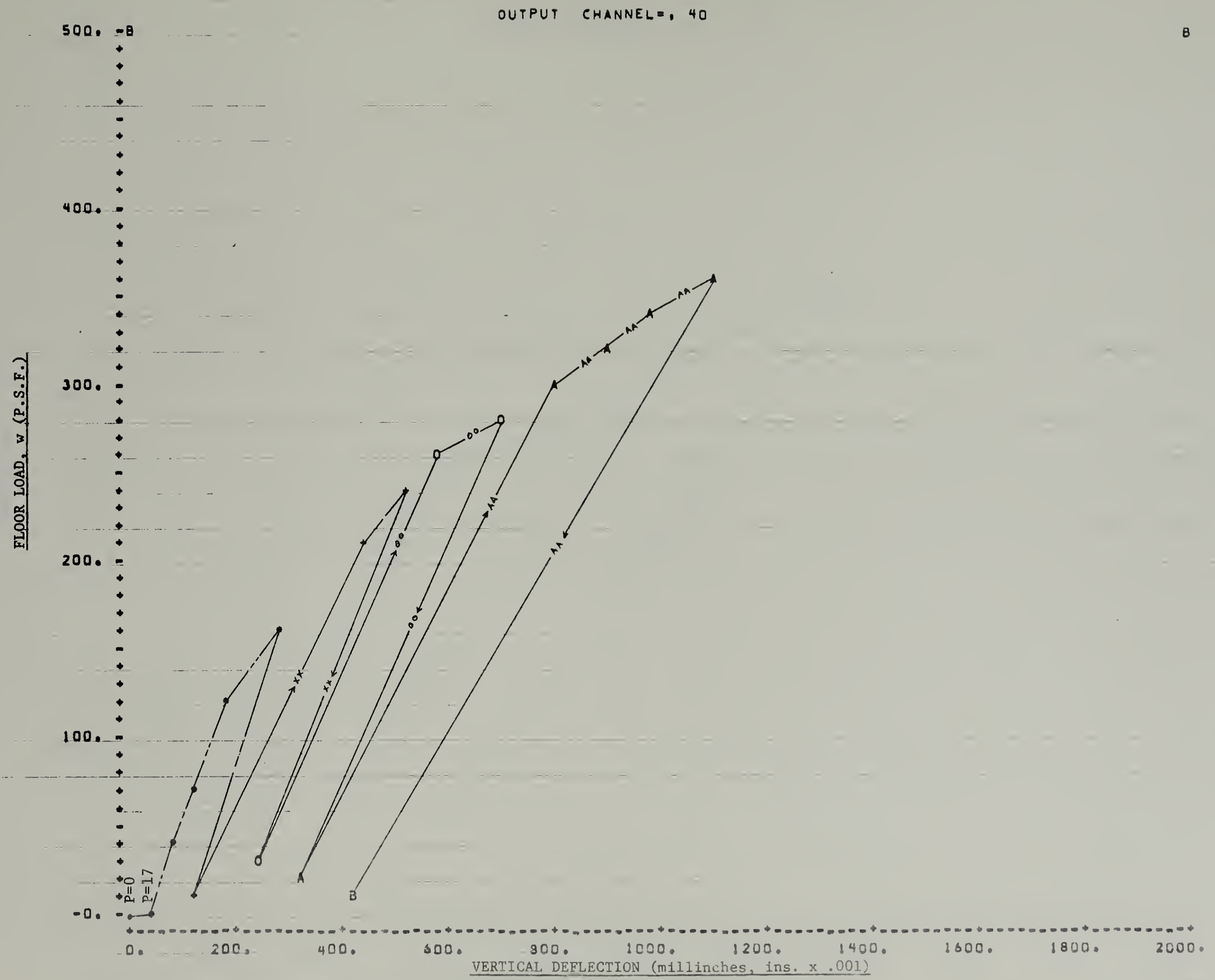


FIGURE NO. A.64 - TEST NO. 16, MAJOR FLOOR LOAD VS. BEAM DEFLECTION

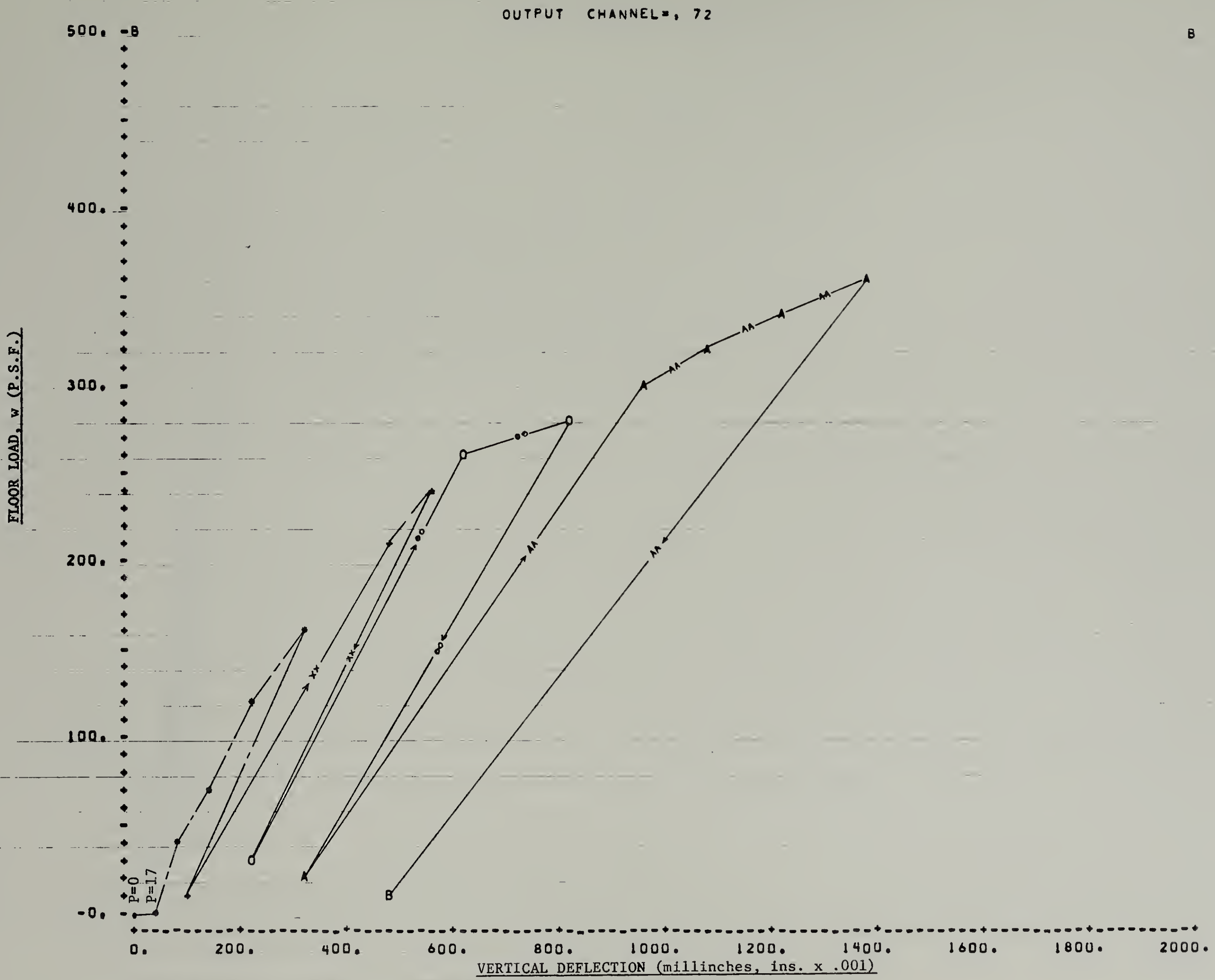


FIGURE NO. A.65 - TEST NO. 16, MAJOR FLOOR LOAD VS. BEAM DEFLECTION

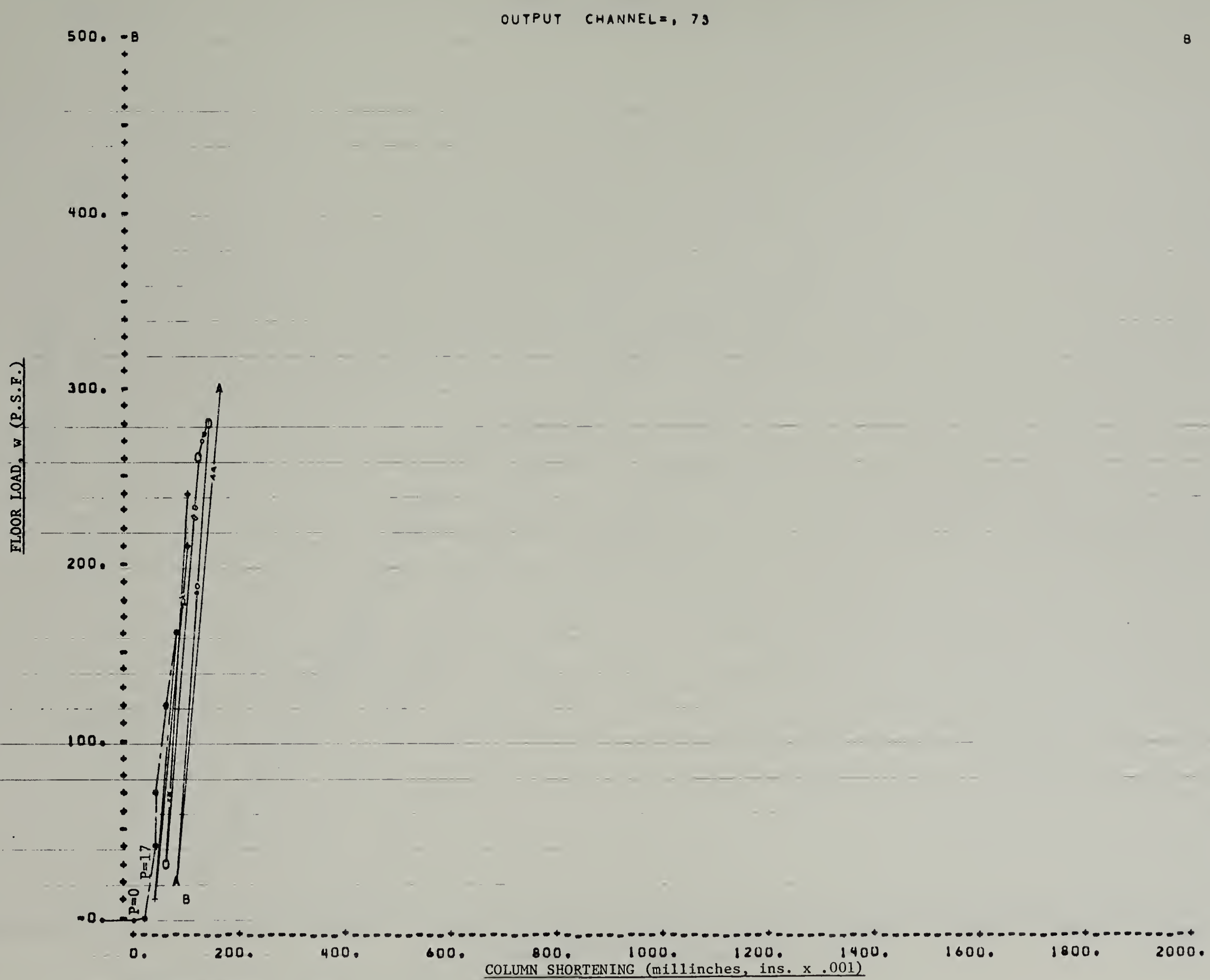


FIGURE NO. A.66 - TEST NO. 16, MAJOR FLOOR LOAD VS. COLUMN SHORTENING

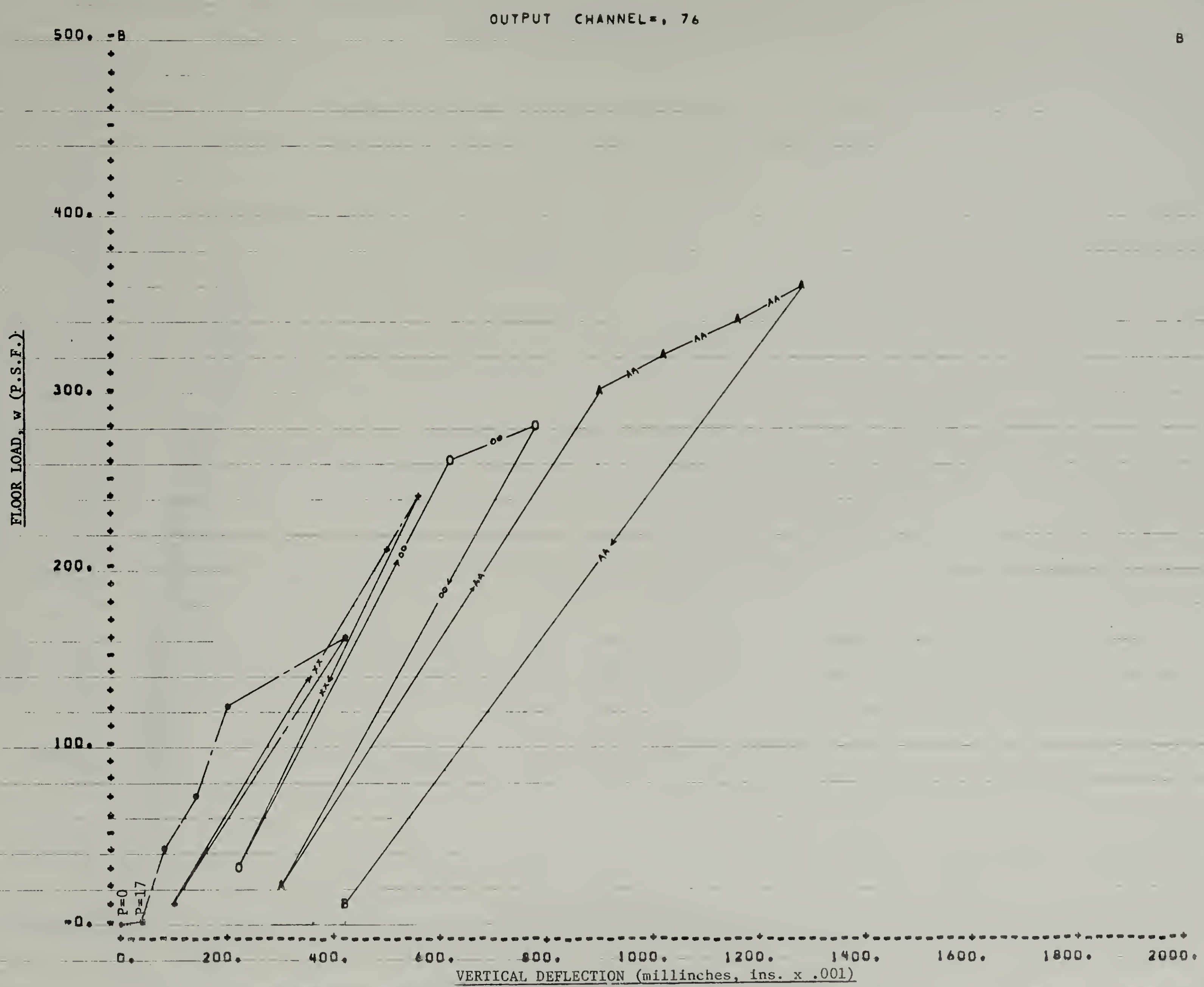


FIGURE NO. A.67 - TEST NO. 16, MAJOR FLOOR LOAD VS. SLAB DEFLECTION

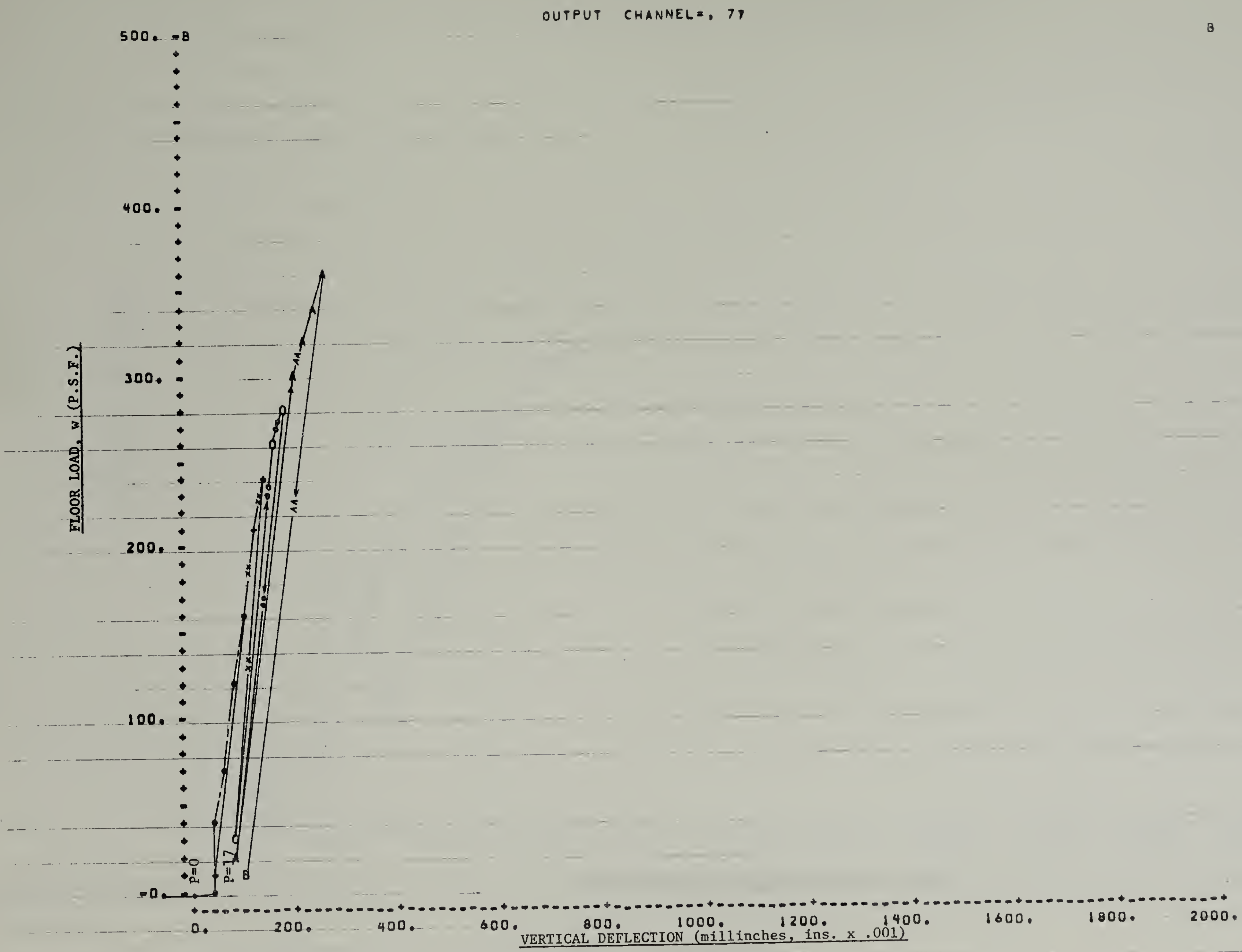


FIGURE NO. A.68 - TEST NO. 16, MAJOR FLOOR LOAD VS. SLAB DEFLECTION

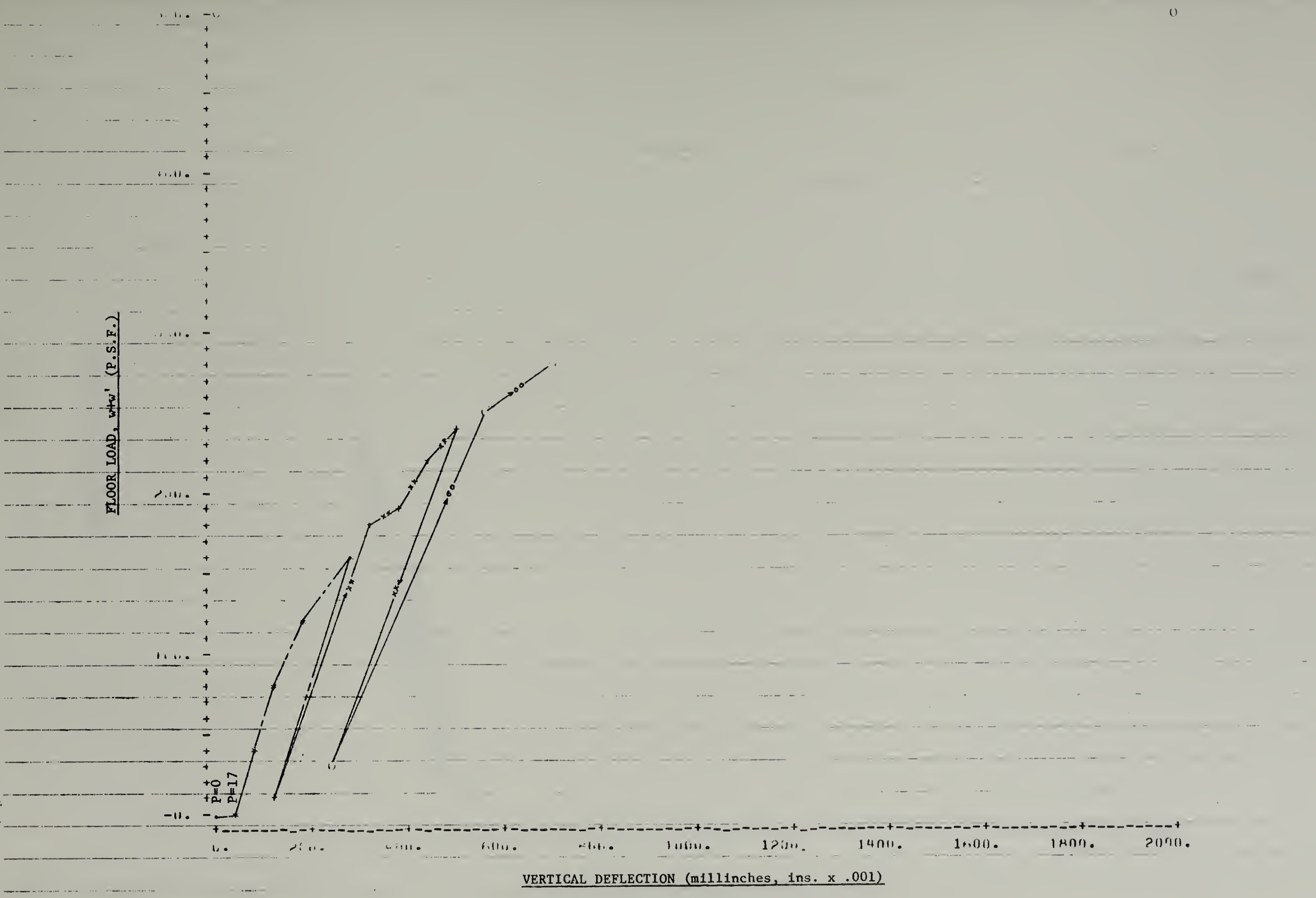


FIGURE NO. A.69 - TEST NO. 16A, FLOOR LOADS VS. BEAM DEFLECTION

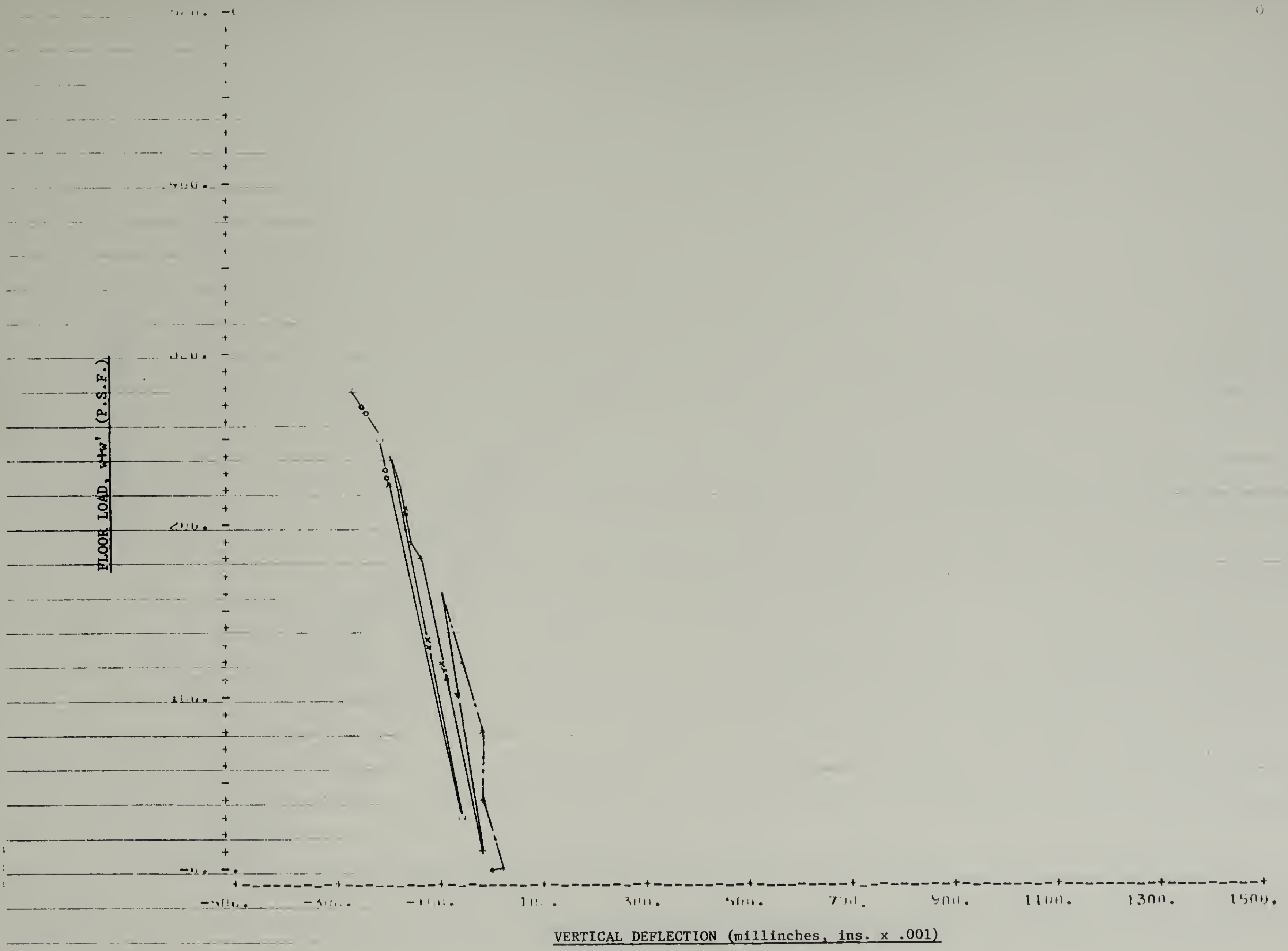


FIGURE NO. A.70 - TEST NO. 16A, FLOOR LOADS VS. BEAM DEFLECTION

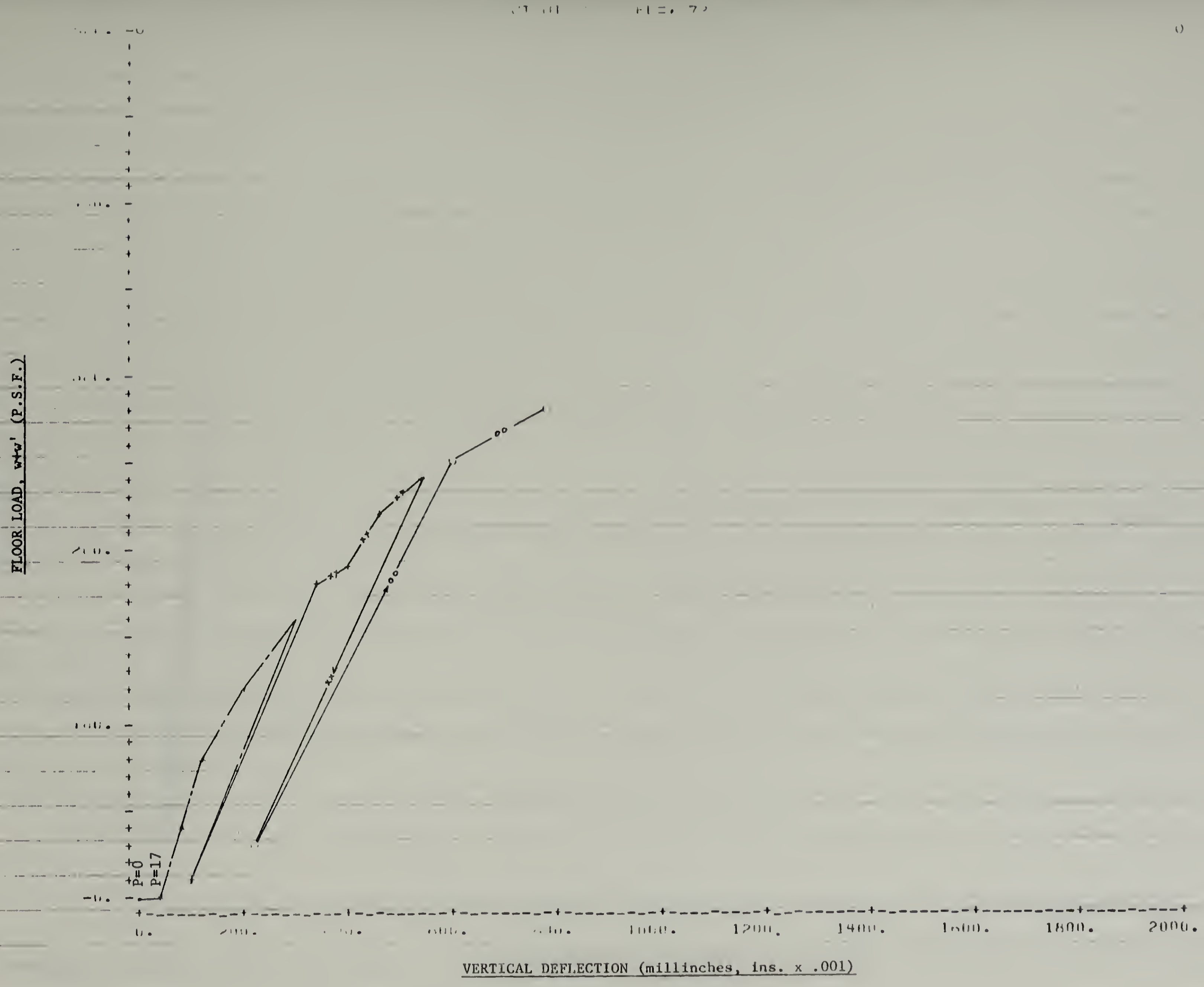


FIGURE NO. A.71 - TEST NO. 16A, FLOOR LOADS VS. BEAM DEFLECTION

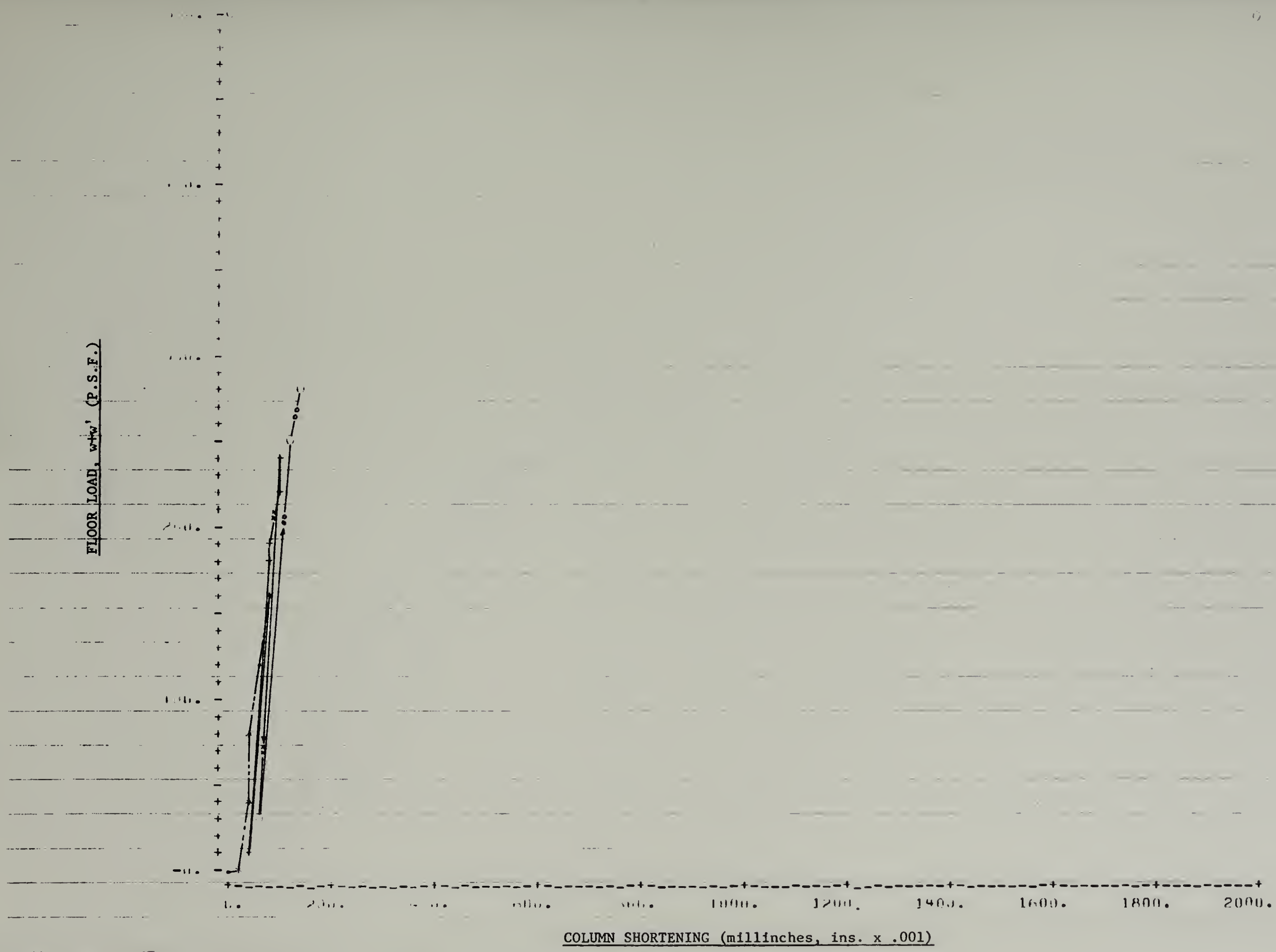


FIGURE NO. A.72 - TEST NO. 16A, FLOOR LOADS VS. COLUMN SHORTENING

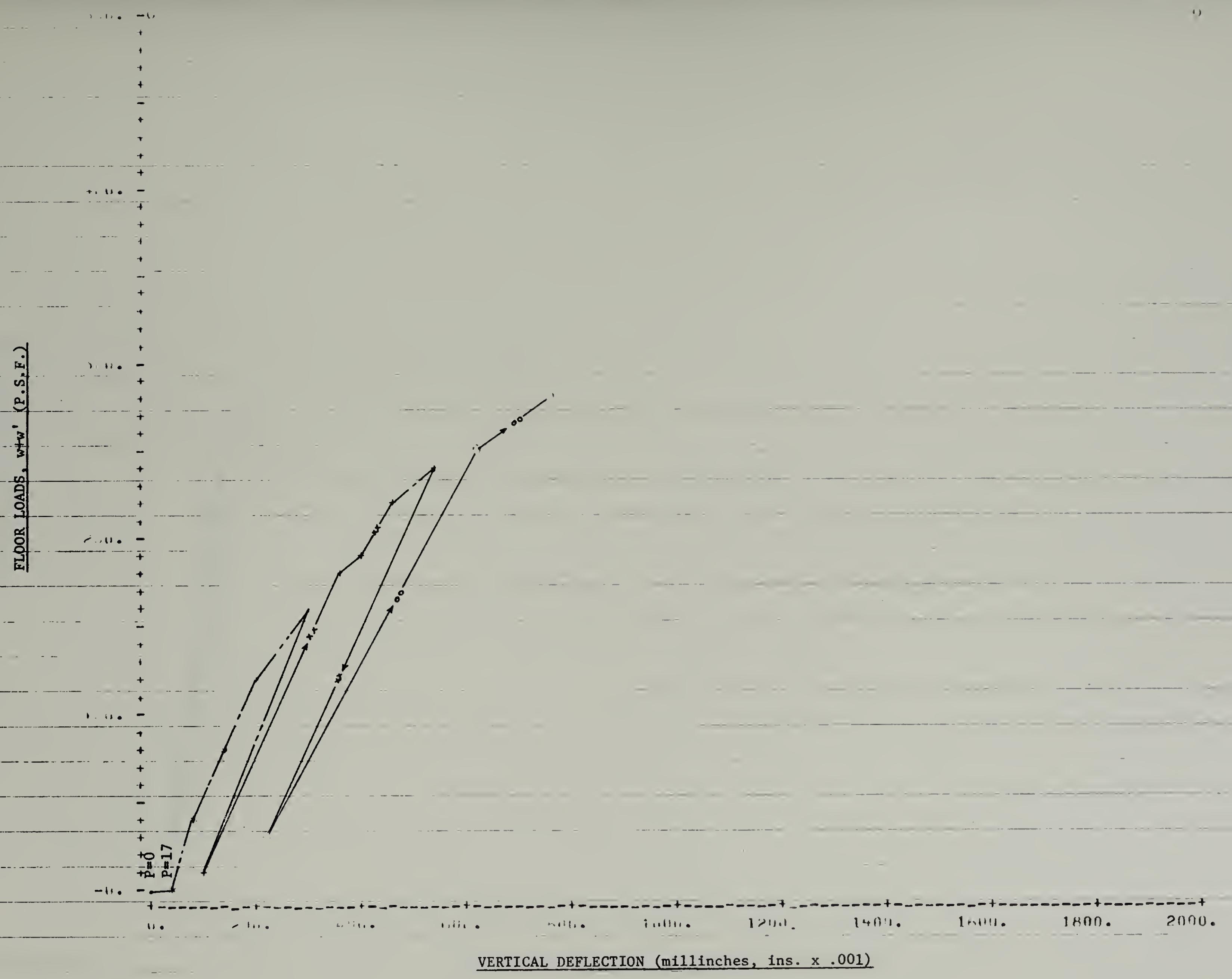


FIGURE NO. A.73 - TEST NO. 16A, FLOOR LOADS VS. SLAB DEFLECTION

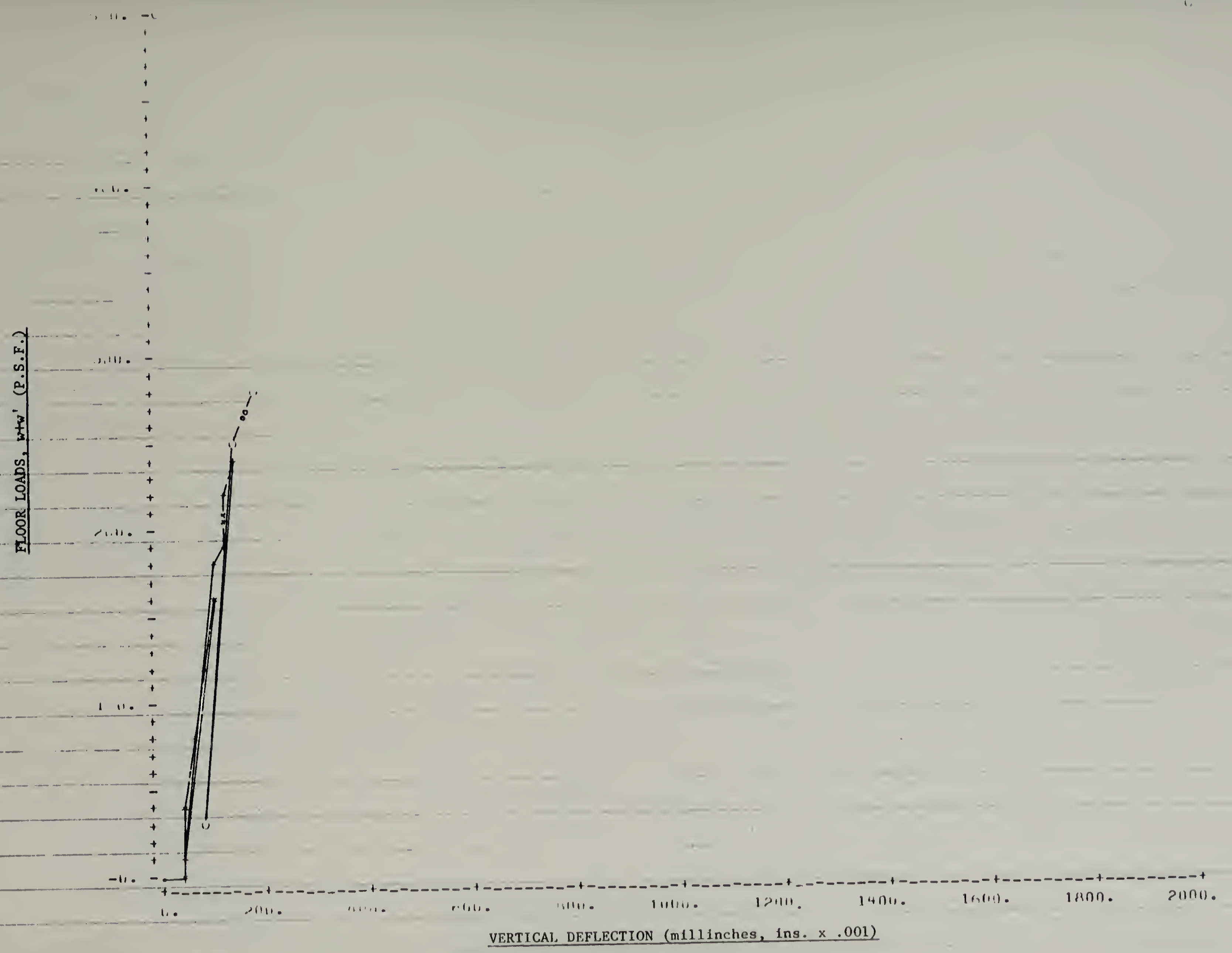


FIGURE NO. A.74 - TEST NO. 16A, FLOOR LOADS VS. BEAM DEFLECTION

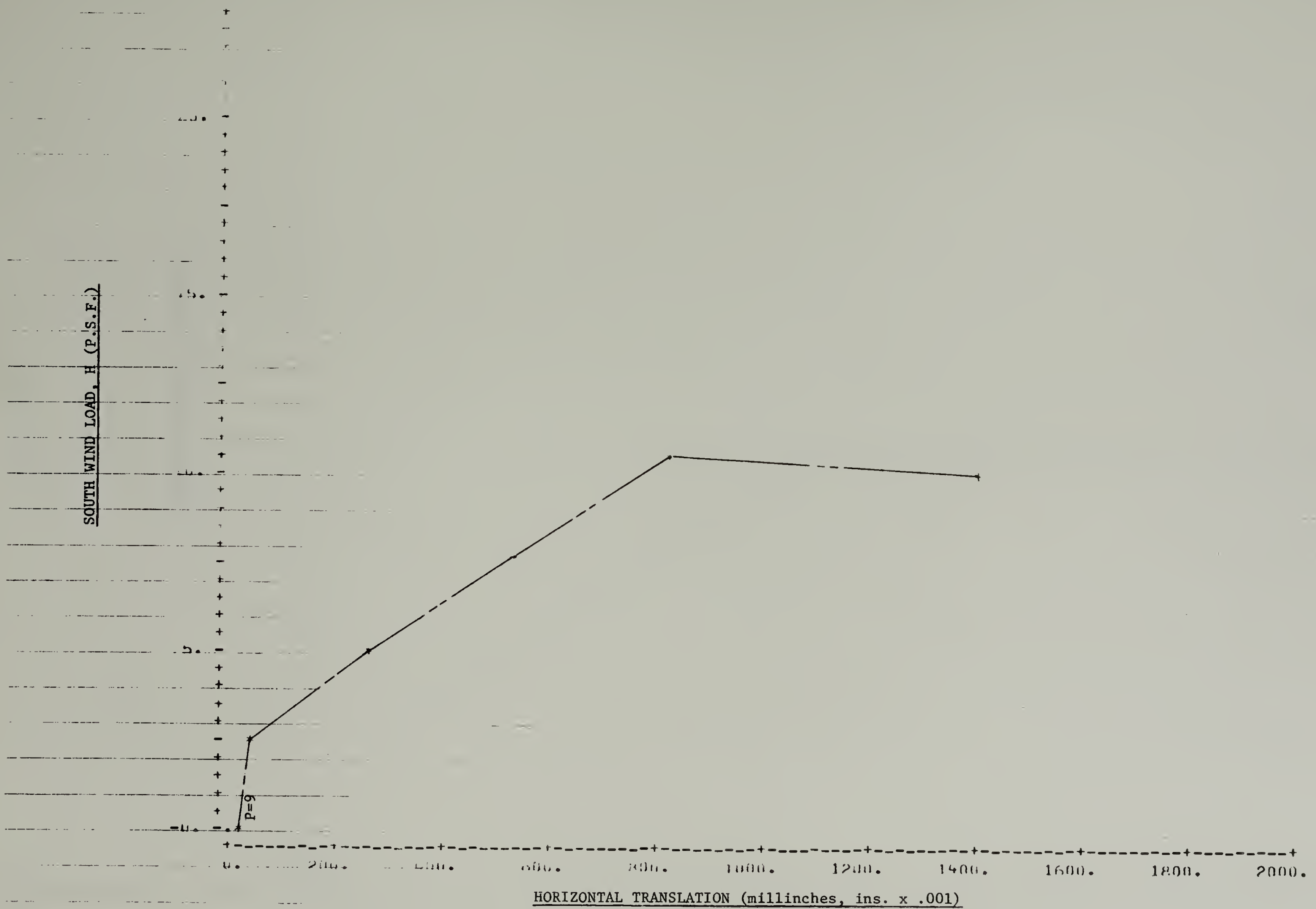


FIGURE NO. A.75 - TEST NO. 18, SOUTH WIND LOAD VS. TRANSLATION

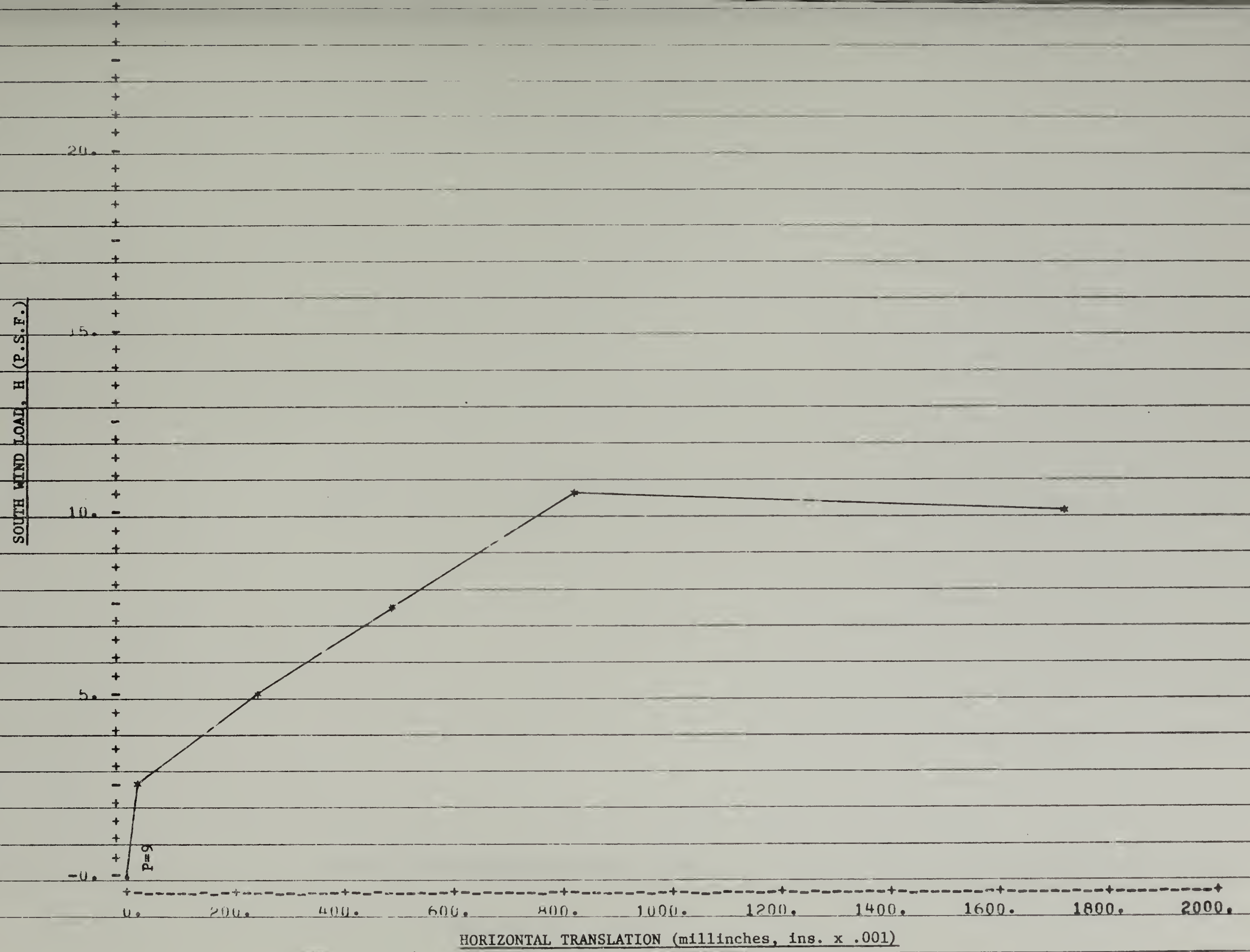


FIGURE NO. A.76 - TEST NO. 18, SOUTH WIND LOAD VS. TRANSLATION

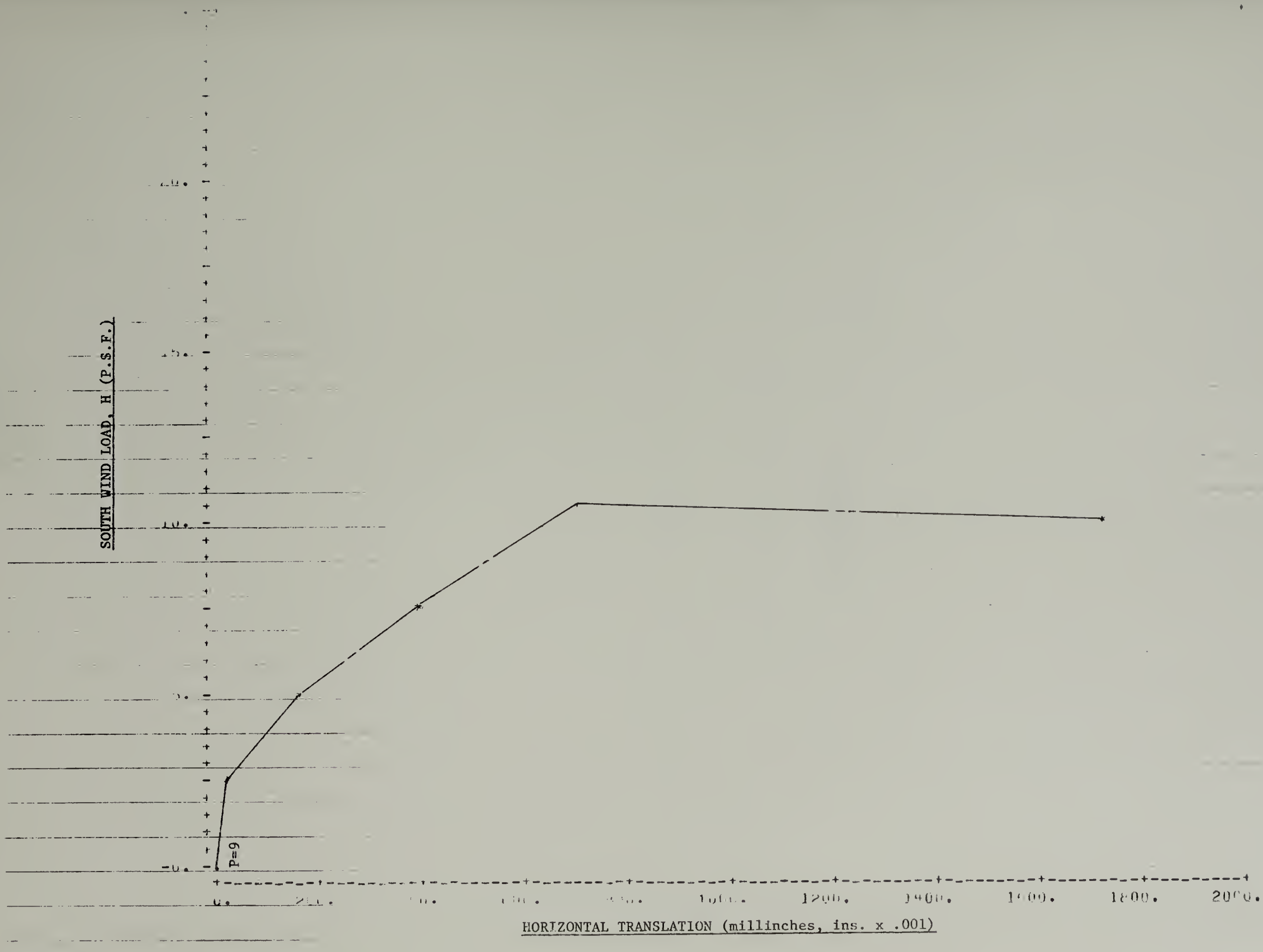


FIGURE NO. A.77 _ TEST NO. 18, SOUTH WIND LOAD VS. TRANSLATION

APPENDIX B

APPENDIX "B"

DISCUSSION OF COLUMN LOAD SIMULATION

Figure B.1 illustrates a comparison between beam-column 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 P.a 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).

To summarize, the upper story column load simulation, as used, 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 <u>increased</u>. Also, the negative main beam moment is <u>decreased</u>, while the positive main beam moment at the center of the span is <u>increased</u>. 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 <u>not</u> produce maximum negative moments at beam ends. However, these moments were adequately simulated in Test 16 where the floor loads were increased to 3% psf.

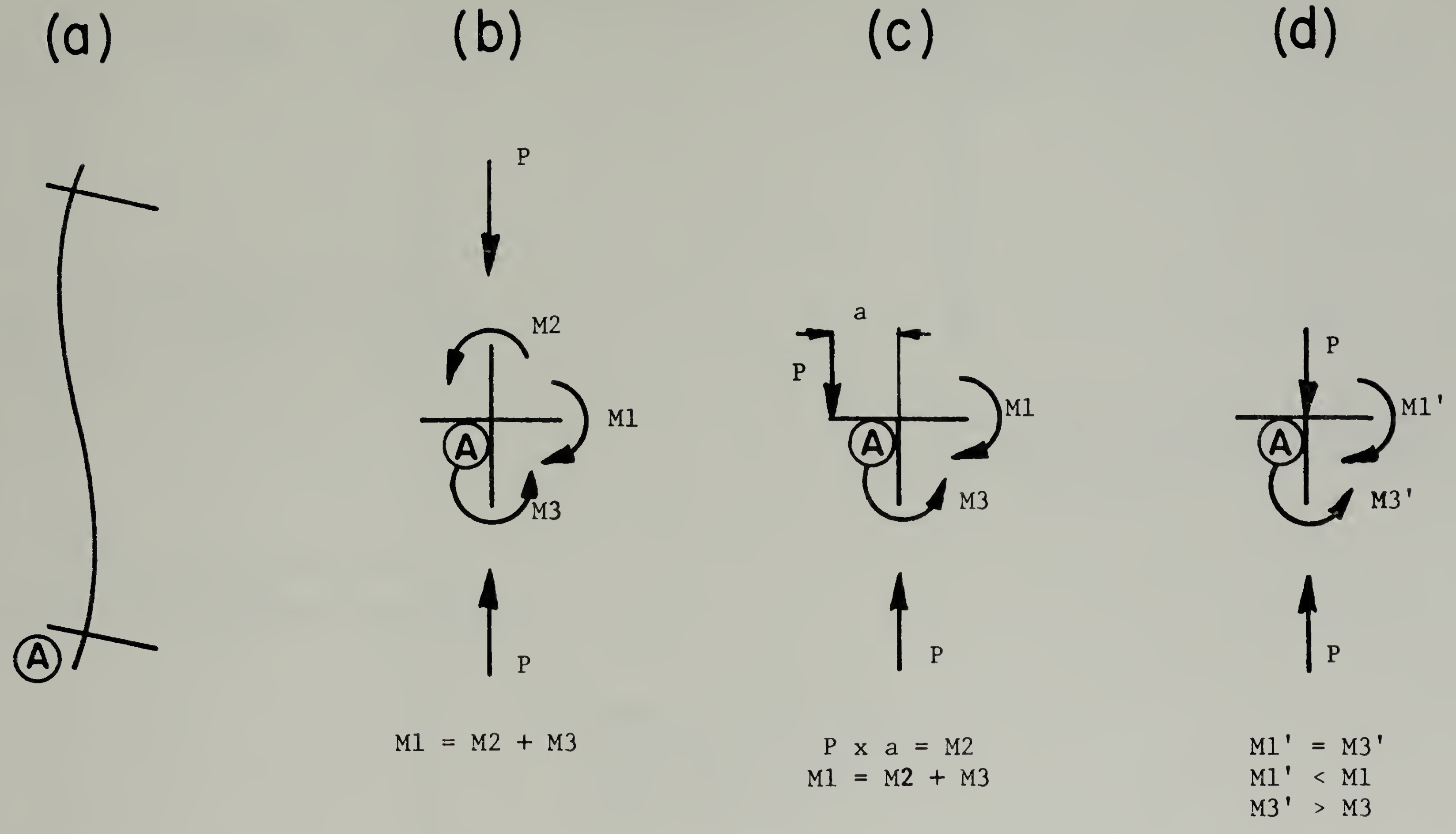
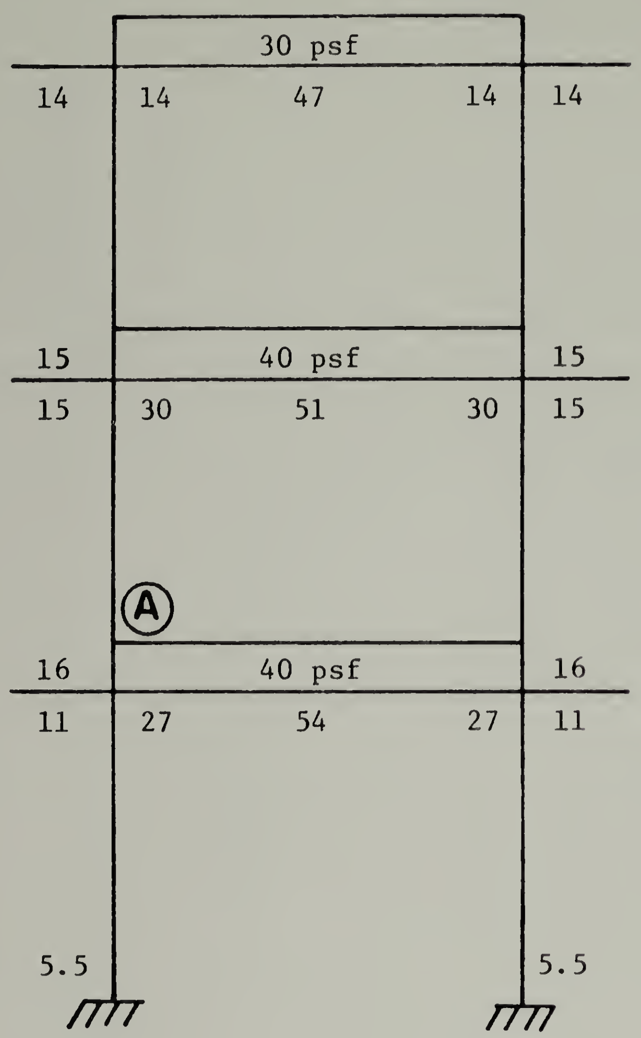
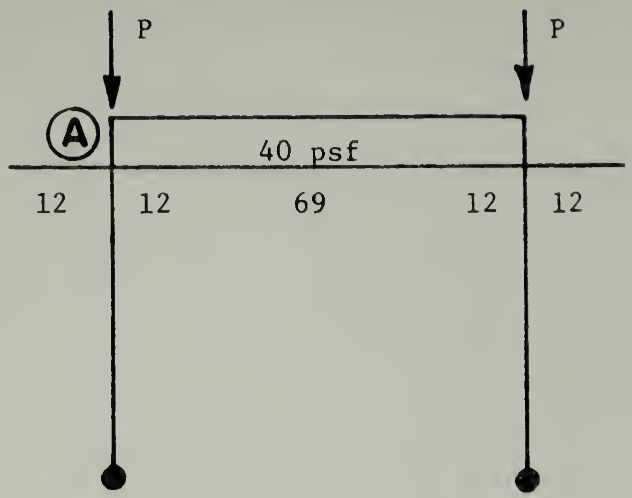


FIG. B.1 COLUMN SIMULATION EFFECT ON CONNECTION MOMENTS



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



(b) Moment Distribution (kip-ins.) due to one live load on test structure.

FIG. B.2 MOMENT DISTRIBUTION

APPENDIX C

APPENDIX "C"

LOAD COMPUTATIONS AND LOAD SCHEDULES

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 2nd Story Column Load
w,w' = Simulated Distributed Dead & Live Loads Exerted by Air Bags
Hs = Simulated S-N Wind Load, at Wall
H_s' = Simulated S-N Wind Load, between Walls
Hw = Simulated W-E Wind Load

II. Summary

(a) Simulated Loads on Test Structure

	D	L	.9D	H	.8H	1.25H	D+L	1.3D+1.7L	1.25(1.5D+1.8L)
P-kips	10	7	9				17	25	35
Hs-kips				4.05	3	5.1			
H_s'-kips				0.9	0.7	1.2			
Hw-kips				2.05	1.5	2.6			
w or w'- (psf)	9.3*	43					52	100**	145**

* Allowance for dead weight of partitions and fixtures.
**Includes dead weight of test structure multiplied by appropriate factor. (i.e., 43x1.07x.3)

(b) Loads on Lower Column in Complete Structure

	D+L	.9D+L	1.3D+1.7L	1.25(1.5D+1.8L)	D	L
P-kips	25	24	37	52	14.4	11

(c) Load on Main Beam for Fatigue Test in Kips 1/8 Point Loading, See Figure 10.14

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

III. Vertical Load Computations

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

area of bags = 2 (9.915x15) = 297 sf.
area of roof = 21.21 x 15.5 = 320 sf.
$\frac{320}{297}$ = 1.07 convert all loads applied by bags.

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

(a)	4" floor channels: $16.3 \text{ psf} \times \frac{18 \times 20}{19 \times 21.21}$	= 14.5 psf
(b)	Topping Slab: $2.5'' \times \frac{1'}{12''} \times 100\ \#/\text{ft}^3$	= 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

3rd 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 & beams prorated per unit area, say, 100 psf concrete (reinforcing weight).

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

Ties: $$\frac{42\ \text{in}^2}{144\ \frac{\text{in}^2}{\text{ft}^2}} = 20^{\text{ft}} \times 100\ \#/\text{ft}^3 \times \frac{1}{200\text{sf}} = 2.92 \text{ psf}$$

5.78 psf

Columns: $$\frac{(6\text{x}4.75)\,\text{in}^2}{144\ \frac{\text{in}^2}{\text{ft}^2}} \times 2 \times 7.83^{\text{ft}} \times \frac{1}{200}\ \text{sf} \times 100\ \#/\text{ft}^2 = 1.55 \text{ psf}$$

(4) Converted floor loads for air bags.

(a) Live Load = 40 psf x 1.07 = 43 psf

(b) Added Dead Load: 1D = 8.75 x 1.07 = 9.35 psf

(c) Combinations: 1.3 D = 1.3 x 9.35 = 12.1
+ 0.3 x 1.07 x 42.8 = 13.7
= 25.8

1.7 L = 1.7 x 43 = 73

1.3 D + 1.7 L = 98.8, say 100 psf

1.5 D = 1.5 x 9.35 = 14.1
+ 0.5 x 46* = 23
= 37.1

1.8 L = 1.8 x 43 = 77.5

1.25 (1.5 D + 1.8 L) = 143.5, say 144 psf

(5) Simulated second story column load.

(a) Dead Load: Tributary floor area = 10.39 x 9.5 = 99sf

Column + partition: 99 x 8.55	= 850 lb.
3rd floor: 99 x 51.58	= 5,150 lb.
roof: 99 x 41.28	= 4,050 lb.
	10,050 lb.

1.D = 10 kips

(b) Live Load: [40 psf (floor) + 30 psf (roof)] x 99 = 6,900 lb., say 7 kips

(c) Combinations: $.9D = 9^k$; $1D = 10^k$.

$1D + 1L = 17^k$.

$1.3D + 1.7L = 25^k$.

$1.25(1.5D+1.8L) = 35^k$.

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

(a) Dead Load:

2nd story	5,150 lb. (See 5a)
3rd story	5,150 lb.
roof	4,050 lb.
D. =	14,350 lb., say 14^k

(b) Live Load:

2nd story=99sf x 40 #/sf =	3,960 lb.
3rd story=99sf x 40 #/sf =	3,960 lb.
roof=99sf x 30 #/sf =	2,970 lb.
1L. =	10,890 lb., say 11^k

(c) Combinations: 1D + 1L

1.3D + 1.7L

1.25(1.5D+1.8L)

*42.8 x 1.07 = 46

(7) Main beam loads for fatigue test.

(a) 1/4 point loading.

Area: $1/2 \times 12.5^{ft} \times 10.385^{ft}$	= 65 sf
D. Slab: $6.25^{ft} \times 7.5^{ft}$(added slab width) x 21 psf	= $945^{\#}$ See (2)
Floor Channel Slab: 16.3 psf x 6.25ft. x 9'	= 920# See (2)
Walls, etc.: 8.75 psf x 65 sf	= <u>570</u> See (2)
1 Dead Load	= 2,435, say 2.5 Kips

1 L = 40 psf x 65 sf = 2.6 Kips, Say 2.5 Kips

1/4 Point Loading - Kips:

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

(b) 1/8 Point Loading - Kips: (See Fig. 10.14)

1D + 1L	2.50
1D + 1.5L	3.125
1D + 2L	3.75
1D + 2.5L	4.375
1D + 3L	5.00

IV. Wind Loads

(1) South wall (Hs, H_s') (Refer to Fig. C.1 (1))

<u>a</u> = Story height x panel width = 8.62 x 10.39 = <u>90 sf</u>

@ point A = Hs. @ point B - H_s'.

Hs is at firewall.

H_s' is midway between two firewalls.

Assume stiff floor distributes shear between two walls over width of structure, then tests assembly carried 1/2 the wind load.

$\therefore$ @ H = 20 psf: $Hs = 1/2 \times (4.5 \times 90)^{ft^2} \times 20^{\#/ft^2} = 4{,}050$ lb. = 4.05 kips.

$H_s' = 1/2 \times 90^{ft^2} \times 20^{\#/ft^2} = 900$ lb. = 0.9 kips.

Therefore, 0.8 Hs = 3.24 kips*
1.25 Hs = 5.1 kips.

---

*3 kips was used in test, based on H = 15 psf rather than 0.8H. This is conservative (See Sec. 5.3).

(2) West Wall (Hw) (Refer to Fig. C.1 (2))

a' = story height x 9.5' = 8.62 x 9.5 = 82 sf.

Assume 1/2 the wind load carried by wall panel (conservative)

$\therefore$ @ H = 20 psf: Hw = 1/2 x $(2.5 \times 82)^{sf} \times 20^{\#/sf}$ =2,050lb. = 2.05 kips.

Therefore, 0.8 Hs = 1.64 kips*
1.25 Hs = 2.56 kips.

---

* 1.6^k was used in test, based on H = 15 psf.

2. LOAD SCHEDULES

For an explanation of symbols, refer to Fig. 6.1, Table 6.I, 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^k	
Increments:	9 - 1^k increments	

(b) Test #2: Column Loads of 0.9D
South Wind Load to 25 psf
(0.9D + 1.25 H)

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

Notes: (1) Six cycles of loading and unloading were applied. Residual deflections were read 5 minutes after removal of all loads.

(2) P was applied initially and held constant.

(3) Hs and H_s' were applied simultaneously.

(c) Test #3: Column Loads of 0.9D
West Wind Load to 25 psf
(0.9D + 1.25 H)

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

Notes: (1) Six cycles of loading and unloading were applied at progressively larger loads. Residual deflections were read 5 minutes 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:	$P = 1.3D + 1.7L = 25^k$
	$w = 1.3D + 1.7L = 100$ psf
Increments:	P: 1st increment 9^k
	then 2^k increments
	w - 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 minutes 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 hours
(1.3D + 1.7L)

Start of test:	5/10/68 5:25 P.M.
Completion:	5/11/68 5:50 P.M.
Loading:	$P = 1.3D + 1.7L = 25^k$ - sustained for 24 hours
	$w = 1.3D + 1.7L = 100$ psf - sustained for 24 hours
Increments:	w - 20 psf increments

Notes: (1) After unloading, an additional cycle of loading and unloading was applied. Additional reading of recovery was taken 24 hours 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:	$P = 1.3D + 1.7L = 25^k$	
	$w = 1.3D + 1.7L - 100$ psf	
Increments:	w - 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:	$P + 1.3D + 1.7L = 25^k$	
	$w = 1.3D + 1.7L - 100$ psf	
	$H_s = 0.8H = 3^k$	
	$H_s' = 0.8H = 0.7^k$	
Increments:	$H_s + H_s'$ - 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 H_s and H_s'.

(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 + 0.8H)

Start of test:	5/14/68	3:10 P.M.
Completion:	5/14/68	4:15 P.M.
Loading:	$P = 1.3D + 1.7L = 25^k$	
	$w = 1.3D + 1.7L = 100$ psf	
	$Hw = 0.8H = 1.5^k$	
Increments:	P & 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 minutes after removal of all loads.

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

Test #9-A: Column loads of 1D
Major floor load to 160 psf
Minor floor load to 160 psf
(1D + 3.7L)

Start of test:	5/15/68	12:10 P.M.
Completion:	5/15/68	2:00 P.M.
Loading:	$P = 1D + 1L = 17^k$	
	$w' = w = 1D + 3.7L = 160$ 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 alternately. 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:	$P = 0.9D = 9^k$	
	Hs = 12 kips	
Increments:	Hs - 1^k increments	

Notes: (1) Three cycles of loading and unloading with progressively increased loads were applied. Loading of residual deflection was taken 5 minutes after removal of all loads.

(2) Walls were racked until 0.35" 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:	$P = 0.9D = 9^k$	
	Hw = 7 kips	
Increments:	Hw - 0.5^k increments	

Notes: (1) Four cycles of loading and unloading were applied at progressively increased loads. Reading of residual deflection was taken 5 minutes after removal of all loads.

(2) Walls were racked until 0.3" 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:	$P = 1.3D + 1.7L = 25^k$	
	w = 1.3D + 1.7L = 100 psf	
Increments:	w - 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 kips	
	w = 1.3D + 1.7L = 100 psf	

Increments: w - 20 psf increments

Notes: (1) Rollers under P oriented to permit N-S sway.

(2) 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:	$P = 1.3D + 1.7L = 25^k$	
	$w' = w = 1.3D + 1.7L = 100$ 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 minutes 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:	$P = 1.3D + 1.7L = 25$ kips	
	$w' = w = 1.3D + 1.7L = 100$ 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 minutes 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:	$P = 0.9D = 9^k$
	$Hs = 2^k$
Increments:	Hs in 0.5^k increments

Notes: (1) Racking load was carried to 2^k 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:	$P = 0.9D = 9^k$
	$Hw = 2.5^k$
Increments:	Hw - 0.5^k 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:	$P = 1D + 1L = 17^k$
	w = 370 psf
	w' = 280 psf
Increments:	w & 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 kips 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:	$P = 60^k = 1D + 7L$	
Increment:	Continuous increase of load.	

Notes: (1) Only the 4 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)

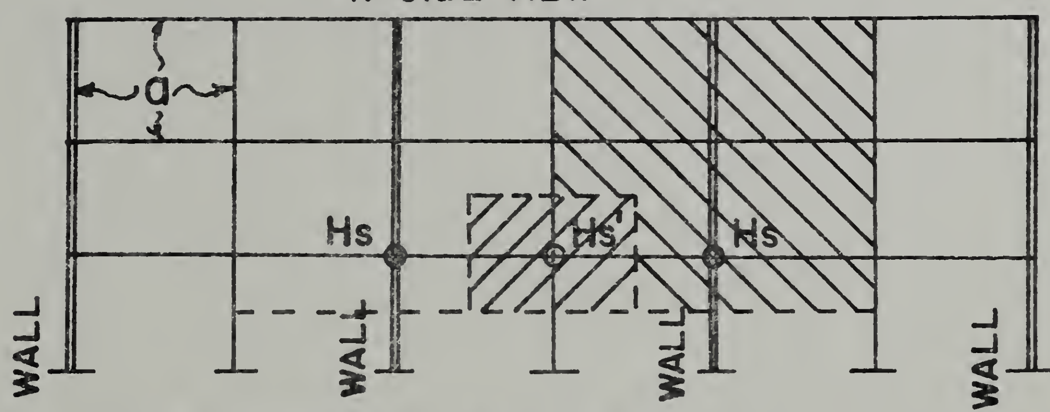
Start of test:	5/22/68	10:50 A.M.
Completion:	5/22/68	11:20 A.M.
Loading:	$P = 0.9D = 9^k$ $Hs = 2^k$	
Increments:	Hs - 0.5 kip increments	

Notes: (1) Racking load could not be further increased.

(2) Loud crack was heard in S-E column at maximum deflection.

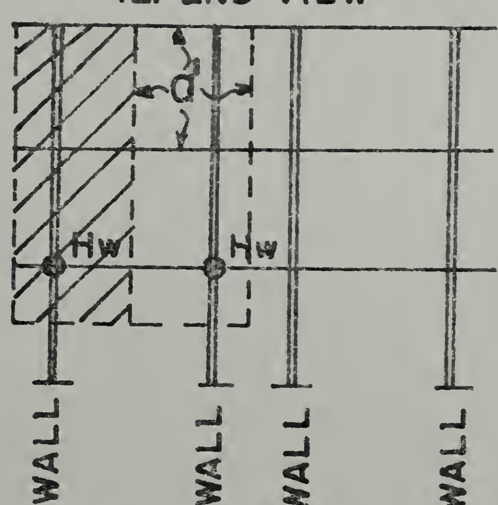
ASSUMPTIONS: (1) WIND FORCES RESISTED BY WALLS
(2) WIND FORCES BETWEEN WALLS TRANSMITTED TO WALL BY STIFF FLOORS.

(1) SIDE VIEW



AREA TRIBUTARY TO Hs = 4.5a
AREA TRIBUTARY TO Hs' = a

(2) END VIEW



AREA TRIBUTARY TO Hw = 2.5a'

FIGURE C.1 – WIND LOADS

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