NATIONAL BUREAU OF STANDARDS REPORT

9973

STRUCTURAL PERFORMANCE OF A BUILDING SYSTEM

NBS

U.S. DEPARTMENT OF COMMERCE
NATIONAL BUREAU OF STANDARDS

J.R. Wright, Chief
Building Research Division, IAT

February 19, 1969

E.O. Pfrang, Chief
Structures Section
Building Research Division, IAT

Phoenix Report

I am attaching a copy of the final report to HUD on the Project Phoenix.

cc: Thompson
Benjamin
Blake
Rowland
Achenbach

use of available technology and to facilitate technological innovation in industry and government. The principal elements of this Institute are:

—Building Research—Electronic Instrumentation—Technical Analysis—Center for Computer Sciences and Technology—Textile and Apparel Technology Center—Office of Weights and Measures —Office of Engineering Standards Services—Office of Invention and Innovation—Office of Vehicle Systems Research—Clearinghouse for Federal Scientific and Technical Information[3]—Materials Evaluation Laboratory—NBS/GSA Testing Laboratory.

[1] Headquarters and Laboratories at Gaithersburg, Maryland, unless otherwise noted; mailing address Washington, D. C., 20234.

[2] Located at Boulder, Colorado, 80302.

[3] Located at 5285 Port Royal Road, Springfield, Virginia 22151.

NATIONAL BUREAU OF STANDARDS REPORT

NBS PROJECT 4215415

January 10, 1969

NBS REPORT 9973

STRUCTURAL PERFORMANCE OF A BUILDING SYSTEM

FINAL REPORT

Prepared For
Department of Housing and Urban Development

By

E. O. Pfrang and F. Y. Yokel

NBS

U.S. DEPARTMENT OF COMMERCE

NATIONAL BUREAU OF STANDARDS

TABLE OF CONTENTS

	Page
SYNOPSIS	i
1. INTRODUCTION	1
2. OBJECTIVE AND SCOPE	3
3. NOTATION	6
4. PERFORMANCE CRITERIA	7
5. TEST STRUCTURE	23
6. LOAD PROGRAM	43
7. INSTRUMENTATION	49
8. RESULTS	52
9. INTERPRETATION OF RESULTS	55
10. COMPONENT TESTS	76
11. MATERIAL TESTS	97
12. SUMMARY AND CONCLUSIONS	102
TABLES	106
FIGURES	117
APPENDIX A	A-1
APPENDIX B	B-1
APPENDIX C	C-1
ACKNOWLEDGEMENT	

SYNOPSIS

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

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

The primary conclusions reached were:

(1) The Neal Mitchell Building System, as erected in the laboratory, satisfied the performance criteria which were set for its evaluation with a substantial margin. As a system,

it exhibited strength and stiffness in excess of service and ultimate load requirements.

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

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 simplified and conservative analyses which do not fully account for system interaction. Because of these simplifications, the strengthening effects of so-called nonstructural portions of a building system are, in general, neglected and the complex interaction of components is frequently overlooked. As a result, in those few cases where tests on 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 on these conventional concepts inhibits innovative solutions to the building problem.

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

2. OBJECTIVE AND SCOPE

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 analysis; 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 system's 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. under provisions of plans and specifications prepared by them for the Phoenix Housing Project in Detroit. The test structure was one story in height and was part of a three story high building, chosen and loaded in a manner that simulated the structural response of the complete building subjected to dead, live, and wind loads. The report also presents results obtained from tests on components and materials used in the test structure.

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 load
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 fire walls
Hs' = South wind load between fire walls
w = floor load between columns
w' = floor load - Cantilever Section

3.3 Deflections

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

3.4 Lengths

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

4. PERFORMANCE CRITERIA

4.1 Introduction

Some criteria for performance testing have been developed (ACI Committees 318 and 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 use some of the existing criteria 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 center lines 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

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

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

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

Gross deflection is the total deflection of a point.

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

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

4.2.3 Superimposed Loads

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

4.2.4 Failure

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

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

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

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

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

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

Commentary on Criterion 4.3.1

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.

<u>Commentary on Criterion 4.3.2</u>

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

4.4 <u>Performance Criteria</u>

4.4.1 Lateral Deflection under Dead and Wind Loads

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

$$Dh \leq 0.002h$$

where:

Dh = horizontal gross deflection

h = height above grade

Commentary on Criterion 4.4.1

Generally a structure will experience its most severe lateral deflection under a condition of minimum vertical load and maximum lateral load. This criterion is designed to prevent excessive deflection under this condition of loading, and provides a margin of 10% over the maximum lateral loads likely to occur under service conditions.

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

4.4.2 Lateral Deflection Under Dead, Live and Wind Load

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

$$Dh \leq 0.002h$$

<u>Commentary on Criterion 4.4.2.</u>

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

4.4.3. Vertical Deflections Under Service Live Load

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

$$dv \le \frac{\ell}{480}$$

where:

dv = vertical net deflection.

<u>Commentary on Criterion 4.4.3.</u>

Criterion 4.4.3. is based on the proposition that $\frac{\ell}{480}$ represents a reasonable maximum allowable instantaneous deflection under service loads. Prevailing codes usually set $\frac{\ell}{360}$ as a deflection limitation, however studies* have indicated that this deflection is excessive in terms of user comfort and causes minor distress to finishes and partitions. The proposed $\frac{\ell}{480}$ deflection limitation reasonably represents present day consensus based on limited knowledge in this area.

*FHA Housing Research Paper #30, April 1954

4.4.4 Sustained Load Deflection

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

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

(b) $Dh \leq 0.002h$

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

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

If $dv \leq \frac{\ell^2}{20{,}000t}$ $\qquad dvr \leq \frac{\ell^2}{80{,}000t}$

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

where:

Dv = vertical gross deflection

Dvr = residual vertical gross deflection

dv = vertical net deflection

dvr = residual vertical net deflection

ℓ = length of member

t = depth of member

Commentary on Criterion 4.4.4.

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

Under most codes 1.3D + 1.7L is about 90% of the ultimate design load. This is therefore the highest load which should be reasonably expected to cause no permanent structural damage. The deflection limitation in 4.4.4. (a) represents an extrapolation of the service load deflection requirement of $\frac{\ell}{480}$ with an additional allowance for creep deflection.

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

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

Criterion 4.4.4. (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 provides reasonable tolerances for creep and system slack. Lower limits for recoverable deformations cannot be set in this case before further studies are conducted.

4.4.5 Ultimate Strength

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

(a) 1.25 (1.5D + 1.8L)

(b) 0.9D + 1.4H

<u>Commentary on Criterion 4.4.5.</u>

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

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

4.5. Summary of Test and Performance Criteria

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

4.3.1 Model Selection

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

4.3.2 Loading

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

4.4.1 Lateral Deflection under Dead and Wind Load

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

$$Dh \leq 0.002h$$

4.4.2 Lateral Deflection under Dead, Live and Wind Load

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

$$Dh \leq 0.002h$$

4.4.3 Vertical Deflections under Service Live Load

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

$$dv \leq \frac{\ell}{480}$$

4.4.4 Sustained Load Deflection

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

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

(b) $Dh \leq 0.002h$

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

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

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

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

4.4.5 Strength

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

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

(b) $0.9D + 1.4H$

5. TEST STRUCTURE

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

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

5.1. Proposed Structure

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

1. Precast components

2. Cast-in-place topping slabs.

3. Gypsum walls, and

4. Foundations, grade-beams and slabs on grade.

5.1.1. Precast Components

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

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

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

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

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

1 part Type I cement

2 parts of masonry sand

3 ounces per 1 lb. of cement of "Euco Weld", a polyvinyl acetate emulsion produced by "Euclid Chemical Company."

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

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

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 #4 deformed intermediate 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 specified "nominal" thickness of 2". 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 by 2 # 4 bars on the first floor and

2 # 3 bars on all other floors, as shown in Fig. 5.8. 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.

The location of these walls in the structural system is illustrated in Fig. 5.17.

5.1.3.(1) Fire walls

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

The fire walls are standard dry-wall construction. Metal channels are attached to the concrete members with power-actuated fasteners at a 6" to 8" 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 1/2" gypsum backing board (ASTM-C442) and one 5/8" gypsum wallboard (ASTM-C36).

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

5.1.3.(2) Exterior Walls

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

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

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 (E-W direction in the test structure). Each building thus has two interior walls in the long directions. These walls fill the 10' panels between columns (See Fig 5.17.). A three ft. door may be expected in every second panel.

Several types of interior walls are used in the Mitchell System, of these the standard 2 1/2" "structicore" partition wall construction was deemed to have the least resistance to lateral load and was thus chosen for the laboratory structure. Figure 5.21. shows typical sections of a "structicore" wall, and Figure 5.22. shows a typical interior "structicore" wall partially dismantled.

5.1.4. Foundations, Grade Beams and Slabs on Grade

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

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

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

5.2 The Test Structure

5.2.1. Structural Simulation

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

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

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

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

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

5.2.2. Description of the Test Structure

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

The test structure consisted of:

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

5.2.2.(1) Precast components consisted of:

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

(b) Three main beams (See Figs. 5.8. and 5.9.).

(c) Four tie beams (See Fig. 5.10.).

(d) Eight 2' wide and two 1' wide floor channels. (See Fig. 5.16) The narrow channels were placed along the north and south edge of the structure. Reinforcement in the channel legs consisted of # 5 deformed intermediate grade (ASTM-A15) bars, instead of the # 4 bars specified in the plans.

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 ± 1/8" measured from the top of the main beams. The top of the floor channels is irregular and tends to be somewhat higher, thus producing a lesser average slab thickness relative to the top of the channel slab. This slab was originally cast to an excessive thickness and was reduced to its final thickness by terrazzo grinders. The final average slab thickness was 2 1/2" ± 1/8" measured from the top of the main beam. This is an average thickness with respect to the top of the channel slabs in excess of 2".

5.2.2.(3) Walls

The test structure had the following walls:

(a) East and west walls were "fire walls" as described in Section 5.1.3.(1)., except that 3/8" gypsum backing boards and 1/2" wallboards were used instead of the thicker sizes called for in the plans.

(b) The south wall was an exterior wall as described in Section 5.1.3(2) except that the exterior siding was omitted and 1/2" thick wallboards were used instead of the 5/8" thickness shown in the plans.

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

All channels for the wall system were attached by power actuated fasteners to the floor slab and the structural frame. 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 doorframe on its west side. This represents the least stiff condition that may be encountered in the field.

The western panel of the north wall contained a 3' x 7' wooden doorframe 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 slab 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. Tests indicated that the floor slab concrete had a 17-day compressive strength of 5,600 psi. Slab reinforcement consisted of a 66-1010 mesh (ASTM-A185).

The slab was poured around the columns which were lined by 1/2" asphalt-impregnated fiberboard, thus forming full depth pockets at the column seats to permit column rotation at the base. A 1/8" thick neoprene sheet was inserted between the column base and the laboratory floor.

5.2.2.(5) Materials

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

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

5.3 Fidelity of Simulation of Field Conditions by the Test Structure

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

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

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

5.3.1. Interaction Between the Test Structure and the Complete Structure

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

(1) Columns

(2) Abutting tie beams and main beams

(3) A continuous topping slab

(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 on the upper stories.

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

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

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

5.3.1.(2) Abutting Tie Beams and Main Beams

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

5.3.1.(3) Continuity of Topping Slab

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

5.3.1.(4) Walls

The wind load is imparted to the wall by 1) shear along its upper connection to the beam above it, and 2) bearing of the windward column against the wall.

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

In the test structure, wind loads equal to 1/2 the wind loads generated by the entire contributory portion of the 3 story building were imparted at the end of each main or tie beam by a ram load, as illustrated by Fig. 5.26. In the case of the north direction, a wind load was also applied at the main beam on top of the column in-between the two fire walls. Due to the stiffness of the floor system these wind loads have a net effect equal to the effect that may be expected on a structure in the field. The reason for applying only 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 north direction and to more than two wall panels in the east direction.

Test results also indicate that the walls participate in the support of vertical loads. This was demonstrated by the fact that deflection of main beams connected to fire walls increased almost 5 fold when these walls were removed. As will be noted later, the loading applied in Test #9 more than compensated for any adverse effect of vertical loads on the walls under service load conditions. Column loads were computed without regard to possible wall participation in load support. It is therefore concluded that the simulation of wall action adequately represented the most adverse conditions that may be expected in a complete structure.

5.3.2. Simulation of Foundation Conditions

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

In the test structure, the columns were "cut off" at the bottom of the floor slab. The lower ends of the columns were provided on all sides with a 1/2" thick asphalt-impregnated fiberboard 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 resulting 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. (See Fig. 5.26) This created a uniformly distributed load which was able to follow the deflections of the slab. Air bags were made of 20 mil. polyethylene and were designed to withstand 300 psf (7 times live load). The live loads applied represented a valid simulation of "live load" conditions, as used in structural design.

Horizontal live loads were applied by horizontal 10-ton rams as illustrated in Figs. 5.26 and 5.29.
The validity of wind load simulation has been discussed in Section 5.3.1.(4).

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

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

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

6.3.1 Tests conducted on the structure with walls installed:

Test #1: Column loads to 0.9D
(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 + 3.5L)

Test #9-A: Column loads of 1D

Major floor load to 160 psf

Minor floor load to 160 psf

(1D + 3.5L)

Test #10: Column loads of 0.9D

South wind load to 60 psf

(0.9D + 3H)

Test #11: Column load of 0.9D

West wind load to 74 psf

(0.9D + 3.7H)

6.3.2 Tests conducted on the structure after the removal of walls

Test #12: Column load of 1.3D + 1.7L
Major floor load to 1.3D + 1.7L
Rollers under column loads oriented to permit sway in the east - west direction (1.3D + 1.7L)

Test #12-A: Column load of 1.3D + 1.7L
Major floor load 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
(0.9D + 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 the remaining LVDT, gage No. 53, had an interval of 0.00001 in.

Figure 7.2 illustrates the northwest view of model instrumentation. Diagonal deformations were recorded by LVDT 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 key punched and reduced by electronic computer.

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

8. RESULTS

A total of 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 model of the total building system. Tests No. 12 through No. 18 were carried out on the system 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 2000 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 ordinate of each curve measures the variable load. Load symbols are defined in Section 3. The abscissa measures deformation, where zero deformation is chosen prior to any load application. Thus in tests where an initial constant load is introduced, the abscissa 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 Structural System with the performance criteria in Section 4, the structural behavior under loads, and the interaction of structural components.

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

Data pertinent to the discussion in this Section are presented in Figs. 9.1 thru 9.23.

9.2 Vertical Forces

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

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

9.2.1 Structural Response to Vertical Loads

9.2.1.(1) General

Figure 9.1* 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

*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.

of this beam due to the same loading are also shown to permit evaluation of the order of magnitude of the "net deflections" as well as the column deflection. Examination of all test data indicates that from the point of view of magnitude of vertical deflections, this curve illustrates the most critical point in the structure.

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

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

(2) The 24 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).

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

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

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

Both Figs. 9.1 and 9.2 illustrate the case of interior span loading (load "w" acting alone), since this appeared to be the more critical loading configuration. The relative influence of these two loading patterns is illustrated by Fig. 9.4, which shows center main beam midspan deflection for Test 9 with interior loading (w) alone, and Test 9A with interior and cantilever loading (w + w'). As would be expected, the "w loading" is more critical in terms of deflection; however, only slightly so.

9.2.1.(2) Influence of Walls

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

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

9.2.1.(3) Slip between Main Beams and Topping

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

9.2.1.(4) Translation due to Vertical Loads (Walls Removed)

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

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

$$dv \leq \frac{\ell}{480} = \frac{144''}{480} = 0.30''$$

where: dv = vertical net deflection

Under vertical loading of 1D + 1L the most critical vertical deflection in the test structure occured at the midspan of the center main beam. Fig. 9.4 illustrates test #9 plotting total vertical deflection at midspan of the center main beam together with total vertical deflection at one of the column supports of the same beam. The vertical net deflection will be the difference between the midspan deflection and the deflection of the beam support. Fig 9.4 illustrates that at the level of 1D + 1L the critical vertical net deflection was 0.04", which is considerably less than the permitted 0.30" net deflection.

Criterion 4.4.3. was therefore satisfied.

9.2.2.(2) Performance Criterion 4.4.4 Sustained Load Deflections

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

(a) $dv \leq \frac{\ell}{360} \times \frac{0.3D + 1.7L}{L} = \frac{144''}{360} \times \frac{100\ \text{lb}^*}{43\ \text{lb}} = 0.93$

(b) $Dh \leq 0.002h = 0.002 \times 94'' = 0.19$

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

(c) If $dv > \frac{\ell^2}{40{,}000t}$ $\qquad dvr \leq 0.25\ dv$

if $dv \leq \frac{\ell^2}{40{,}000t}$ $\qquad dvr \leq \frac{\ell^2}{80{,}000t} = \frac{144^2}{80{,}000 \times 9.5} = 0.03''$

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

* $\frac{100\ \text{lb}}{43\ \text{lb}}$ represents the ratio of the simulated floor load used in this test to the simulated floor load corresponding to 1L

where:

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

(a) Under the vertical loading of 1.3D + 1.7L sustained for 24 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, during the sustained load portion of this test, the instruments measuring the support deflection malfunctioned and there is thus no complete record of the beam support deflection. Thus, only the short-term portion of this deflection has been subtracted to obtain dv. The maximum measured Dv, which was 0.14", is considerably less than the 0.93" allowed by Criterion 4.4.4(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. (See Figs. 9.6 and 9.7.) This is considerably less than the 0.19" permitted by Criterion 4.4.4(b).

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

Criterion 4.4.4 was therefore satisfied.

9.2.2.(3) Performance Criterion 4.4.5 Ultimate Strength

> The structure or any portion thereof shall not fail at a load smaller than the following:
>
> (a) 1.25 (1.5D + 1.8L) = w = 145 psf.

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

Criterion 4.4.5.(a) was therefore satisfied.

9.3 Horizontal Forces

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

9.3.1 Horizontal Loads in the North Direction

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

9.3.1.(1) Racking Tests with Minimum Vertical Loads

These racking tests were conducted with a superimposed column load of 0.9D acting alone. The results of the racking test in the north direction are illustrated in Fig. 9.8. This figure shows lateral deflection measured at the level of the 2nd floor of the test structure. Loads were applied to simulate a wind pressure of 25 psf acting from the south. It may be noted from this figure that while over-all deflection was small (0.091"), recovery was also small. Figure 9.9 shows the results of a later racking test which was carried to an equivalent of 60 psf wind load. These two tests are simultaneously plotted in Fig. 9.10* and show good agreement.

Figure 9.11 shows a plot of south wind load versus diagonal compressive deformation measured on one of the fire walls. The diagonal 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 is extremely

*only a portion of the test to 60 psf is shown here since it exceeds the scale of this figure small. It is interesting to note from this figure that the recovery of the walls was good for all levels of load. No signs of distress were observed in the walls or other parts of the structure during Test 2 in which a wind load equivalent to 25 psf was applied. However, at the upper limit of Test 10 at a wind load equivalent to 60 psf, some distress appeared in the form of bowing out (buckling) in compression areas near the corners of the wall panels. These signs of distress disappeared upon removal of the lateral load. After removal of the walls all connections between the walls and the frame were found to be in good condition, showing no dislocation of screws or anchorage devices. During Test 10 there was some opening up of the joints between the columns and the wall panels in regions which would normally be subjected to tension by the development of diaphragm action in the walls. These openings were all less 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 (0.8H) south wind load was applied to the structure. The results of this test are illustrated in Fig. 9.12. This test is also plotted in Fig. 9.13

along with Test 2 which had 0.9D and 25 psf wind load. The agreement between these two tests is good. The structure experienced a considerably larger lateral drift under the application of the larger vertical load acting alone than it did under the smaller vertical load (0.024" 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. Results are illustrated in Fig. 9.14. This test is compared with an identical racking test performed before removal of the walls in Fig. 9.10. This comparison clearly indicates that a major portion of the lateral stiffness is provided by the walls rather than by the frame.

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

9.3.2 Horizontal Loads in the East Direction

West wind forces in the test structure were resisted by one interior "structicore" wall and one exterior wall.

These walls would not normally be expected to be as strong as the fire walls, however, their rigidity in the lateral load tests appeared to be comparable to that of the fire walls. Both of the walls in this direction had openings, however, these walls also had a greater over-all 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)). Fig. 9.16 illustrates Test 3 which subjected the structure to 0.9D plus 25 psf wind load from the west. In this test, measured deformations were extremely small (0.012"). Recovery characteristics were similar to those observed for the fire walls. There appear to be two breaks in the load-deflection curve (Fig. 9.16), one at 10 psf and the other at 24 psf. Neither of these were accompanied by any visual signs of distress in either the concrete frame or in the gypsum walls. Fig. 9.17 shows the results obtained from a racking test (Test 11) carried to a wind load in excess of 70 psf. A portion of this test along with the results of Test 3 are shown in Fig. 9.18. It is interesting to note that a definite break developed in the load-deflection curve of Test 11 at 6 psf; again, this break was not associated with any visual signs of distress. These breaks in the load deflection curve are not considered to be particularly significant since, for example in Test 11, even at a wind load of 25 psf the lateral drift of the structure is still less than 0.04". Figure 9.19* shows a plot of load versus wall diagonal compressive deformation for a wall resisting west wind load during Test 11, which was carried to 74 psf. As was the case for the walls resisting south wind load (Fig. 9.10) deformations and correspondingly average strains were extremely small and recovery was good. In this test, distress in the wall was not noted until the very upper range of the loading sequence was reached. At these loads distress was observed in the interior (north) wall in the form of shear cracks (Fig. 9.20). However, no noticeable distress was observed in the wall-frame connections. Some progressive opening of the joint between the wall panels and the columns was observed at loads in excess of 35 psf but the separation was not particularly pronounced and was similar to that discussed in Section 9.3.1.(1).

9.3.2.(2) Racking Test at High Vertical Loads

A racking test was performed in the east direction in Test 8 with 1.3D + 1.7L and 15 psf wind load. The results

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

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

9.3.2.(3) Frame Action versus Wall Action

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

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

In designing the test sequences to which the structure was subjected it was felt that only one meaningful racking test could be carried through to the point at which the structure was approaching collapse. The north direction, which is the narrow direction for this system, was chosen for this test and its results were reported earlier (Test 18) in Section 9.3.1.(3). In the east direction the maximum load to which the frame without walls was subjected was applied in Test 15, which is shown in Fig. 9.23. In this test the frame resisted a wind load in excess of 15 psf without collapse, although some minor flexural cracks were observed in the columns at maximum load. This load level was substantially higher than that sustained in the racking test without walls in the north direction. The conservative nature of the foundation simulation in the test structure, which provided a hinge at the lower column connection, undoubtedly affected the results obtained in this test in an adverse manner.

9.3.3 Compliance with Performance Criteria, Horizontal Loads

9.3.3.(1) Performance Criterion 4.4.1 Lateral Deflection under Dead and Wind Loads

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

$Dh \le 0.002h = 0.002 \times 94'' = 0.19''$

where:

Dh = horizontal gross deflection
h = height above grade

In Test 2 (Fig. 9.8) under 0.9D and a wind load of 22 psf (1.1H) from the south the maximum lateral drift was approximately 0.073" and in Test 3 (Fig. 9.16) with a west wind load the maximum lateral drift was approximately 0.007" 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 lateral deflection due to the superimposed load of 0.3D + 1.7L + 0.8H shall not exceed the following:

$Dh \le 0.002h = 0.19''$

In Test 7 (Fig. 9.24) under 1.3D + 1.7L and a wind load of 15 psf (0.8H) from the south the maximum lateral drift was 0.045" and in Test 8 (Fig. 9.21) with a west wind load the maximum lateral drift was 0.032"*, while the allowable

*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".

drift under this criterion is 0.19".

Criterion 4.4.2 is therefore satisfied.

9.3.3.(3) Performance Criterion 4.4.5.(b) Ultimate Strength

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

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

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

Criterion 4.4.4.(b) is therefore satisfied .

9.4 Summary

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

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

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

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

10. COMPONENT TESTS

10.1 Introduction

Load tests were conducted on the three principal 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 "short"* columns as shown in Fig. 5.6. Reinforcement was 60 ksi steel. Actual outside dimensions and concrete cover of individual test specimens are shown in Table 10.1. Values of concrete strength of the various specimens are reported in section 10.2.1 (1). The tests consisted of the following:

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

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

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

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

10.2.1 Short-Term Destructive Tests

10.2.1.(1) Specimens

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

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

The longitudinal reinforcing bars (No. 6 deformed bars) were approximately 3" shorter than their full required length in Columns 1 and 2, leaving a distance of about 1 1/2" between reinforcing bars and end fixtures, but in Columns 5, 6, 7, 8 and 9 these bars were only 1/4" shorter.

10.2.1.(2) Loading

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

10.2.1.(3) Results

Test results are shown in Figs. 10.4 through 10.10 as load-deflection curves. Ultimate loads are tabulated in Table 10.2. The average maximum load for Columns 1, 5 and 8 (minor axis bending, e=0.5 in.) was 78.9 kips. Fig. 10.11 shows these columns after testing. Column 1 failed near its end connection, and its mode of failure appeared to be partially due to the short reinforcement used. Columns 5 and 8 (which had longer reinforcement) experienced compression failures in the concrete at mid-height at about 12% 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, as illustrated in Fig. 10.12, with excessive bending of the channel-shaped, top-fixture and some spalling of the concrete near this fixture.

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

10.2.2 Sustained Loading (Creep) Tests

10.2.2.(1) Specimens

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

10.2.2.(2) Loading

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

The 25 kip load (1D + 1L) was applied by means of a 30-ton hydraulic ram and a load cell inserted between the top two plates of the test frame. The ram load was applied through the column to the spring, causing the spring to compress. Once the required load was applied, nuts on the 3/4" tie-bars were tightened against the top knife-edge plate. The deflection of the springs was 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.

10.2.2 (3) Results

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

10.2.3. Interpretation of Column Test results.

10.2.3.(1) Short-Term Destructive Tests.

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

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

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

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

*Values of P x dh were computed in accordance with "Proposals for Revision to Sections 915 and 916 of ACI 318-63" by MacGregor, Breen and Pfrang, (unpublished). These computations accounted for concrete cracking.

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

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

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

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

10.2.3.(2) Sustained Loading (Creep) Tests.

Figs 10.16 and 10.17 show the results of creep tests conducted on Columns 3 and 4.

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

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

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

The creep tests on Columns 4 and 3 respectively are also plotted in Figs. 10.20 and 10.21. Curves (D) in these figures are the interaction curves for maximum axial load and maximum total moment at which the steel carries the entire load without concrete participation. Curves (D') show the same interaction curves for the reduced moments when deflections are taken into account. Thus the curves marked (D') represent the combination of axial loads and applied end moments which can be supported by the column reinforcement without concrete participation as a limit condition. It is significant to note that curve (D') in Fig. 10.20 when compared with curve (C) indicates that creep buckling can not occur in this structure even under the assumed ultimate loading conditions. The creep tests of columns 3 and 4 simulate sustained loading of 1 live + 1 dead load in the structure. It can be seen from the plot of these tests in Figs. 10.20 and 10.21 that there is a considerable margin of safety against creep buckling in the direction of both, the major and the minor column axes.

10.3 Channel Slab Test

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

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

The test results are shown in Figure 10.22. The Load at the yield point (2.5k) was higher than the predicted load at the yield point of the reinforcement (2.25k) using the nominal specified reinforcement yield strength, (40ksi) and a 4,500 psi concrete strength. The tests on the laboratory structure also indicated satisfactory performance of these components. No material specimens were tested to determine the actual steel and concrete strength of the floor channels.

10.4 Beam Tests - Repeated Loading

10.4.1 Test specimens

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

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

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

Two types of shear connectors were used in the seven beams. These shear connectors are illustrated in Fig. 10.23. Beams Nos. 5, 7 and 9 used Star inserts spaced 19" on centers similar to the shear connectors used in the test structure. Beams Nos. 8 and 10 used Richmond (Kohler) inserts spaced 19" on centers. Beams Nos. 6 and 11 used the Richmond inserts spaced 9 1/2" on centers. All data on shear connector type and spacing in individual specimen are summarized in Table 10.3.

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 10.3.

10.4.2 Beam Loading

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

The beams were subjected to 1000 cycles of stress alternating between intensities corresponding to 1D and 1D + 1L, (for each ram 1D=2.5kips, 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 (1.25kips).

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 variable differential transducer

(LVDT). Both measuring methods can be seen in Fig. 10.24. In an effort to measure the relative horizontal slip between the beam and its topping slab, 0.001 in. dial gages were mounted on the beam ends. One of these gages can also be seen in Fig. 10.24.

10.4.3 Test Results

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

10.4.4. Interpretation of Results.

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

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

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

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

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

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

The results of the test on Beam 8 are shown in Fig. 10.27. 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. Beam 10 (the companion to Beam 8) 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.

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

When the repeated load tests were conceived, it was felt that from the standpoint of slip behavior, the beams should be capable of sustaining 1000 cycles of loading from 1D to 1D + 1L, and should be capable of sustaining a loading of at least 1D to 1D + 2L before ultimate failure. This component requirement was set for this particular test, even though it was realized that the performance of the main beam as a separate component does not necessarily simulate the behavior of the complete system. All of the beams tested which had partial column connections, except for Beam 9, satisfied this requirement. The reason that the presence of the partial connections had no effect on Beam 9 is not clear.

Beam strength was substantially improved by changing the insert spacing to 9 1/2", as in Beams 6 and 11.

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

11.2 Concrete Tests

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

11.2.1 Precast Component Concrete

The precast components were cast in two days (April 16 and 17, 1968) from five batches of lightweight aggregate concrete. This concrete was made from a 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. The slump was judged to be about 2 in. although it was not measured. The workability of the concrete was excellent with no indication of either segregation or bleeding.

Test specimens (6 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.

Since 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 coarse 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 at 2 days of age. The specimens were then air-dried until tested.

11.2.3 Concrete Test Results

The results from the strength tests are shown in Table 11.1. The unit weight and air-content determinations are presented in Table 11.2. Air-content determinations were made by ASTM method C-457. The values of the modulus of elasticity are shown in Table 11.3. By way of comparison, values for an average lightweight aggregate concrete are included in these tables. These values are averages from a total of 46 batches of concrete made from 21 different expanded shale, lightweight aggregates.[1/] The cement content for this average concrete was 6.5 sacks/yd and the average wet density was 100.3pcf.

The results indicate that; (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 justify four conclusions:

(1) The concrete strengths in the test structure were significantly higher than the strengths called for in the plans and specifications of the Neal Mitchell System.

[1/] T. W. Richard., "Creep and Drying Shrinkage of Lightweight and Normal-Weight Concretes," NBS Monograph 74, National Bureau of Standards, Washington, D. C., Mar. 64

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

(3) 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.

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

11.3 Reinforcing Steel

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

12. SUMMARY AND CONCLUSIONS

12.1 Summary

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

12.2 Conclusions

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

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

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

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

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

(3) Floor channel reinforcement size was increased

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

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

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

(1) The building system satisfied the performance criteria which were set for its evaluation with a substantial margin.

As a system it exhibited strength and stiffness in excess of services and ultimate load requirements.

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

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

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

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

appeared to play a major role in shear transfer between the precast beam and the cast in place topping slab.

TABLES

TABLE 6.1

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

Symbols for Loading:

D = Service Dead Load

L = Service Live Load

H = Service Wind Load

SIMULATED LOAD	SYMBOL	MAGNITUDE		
		1D	1L	1H
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 C)

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

TABLE 10.1

COLUMN TESTS
(Actual Dimensions and Reinforcement Cover of the Columns)

Column No.	Outside Dimensions in inches		Cover Measurements in inches							
	"X"	"Y"	A	B	C	D	E	F	G	H
1	4.76	6.05	11/16	3/4	11/16	13/16	11/16	3/4	3/4	11/16
2	4.77	6.07	1 1/8	1	1 1/16	7/8	1	1 1/8	1 1/16	1 1/8
3	4.76	6.05	-----	-----	-----	-----	-----	-----	-----	-----
4	4.76	6.15	-----	-----	-----	-----	-----	-----	-----	-----
5	4.79	6.16	1 1/4	1 1/4	1 3/16	1 1/4	1 1/16	1 1/16	1 1/8	7/8
6	4.81	6.06	1 1/8	1 1/16	1 1/8	1 1/16	7/8	1 1/8	1	3/4
7	4.78	6.05	1 1/4	1 3/16	7/8	1	1 1/16	1 1/8	1 3/8	1 3/16
8	4.74	6.15	1 1/4	1 1/8	15/16	1	11/16	1 1/4	3/4	1 5/16
9	4.75	6.20	1 1/8	1 1/8	7/8	7/8	1 1/16	1 1/8	1 1/8	1 1/8
Average	4.77	6.10	1.12	1.07	0.96	0.98	0.92	1.08	1.03	1.01
Specified (Fig. 5.13)	4.75	6.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

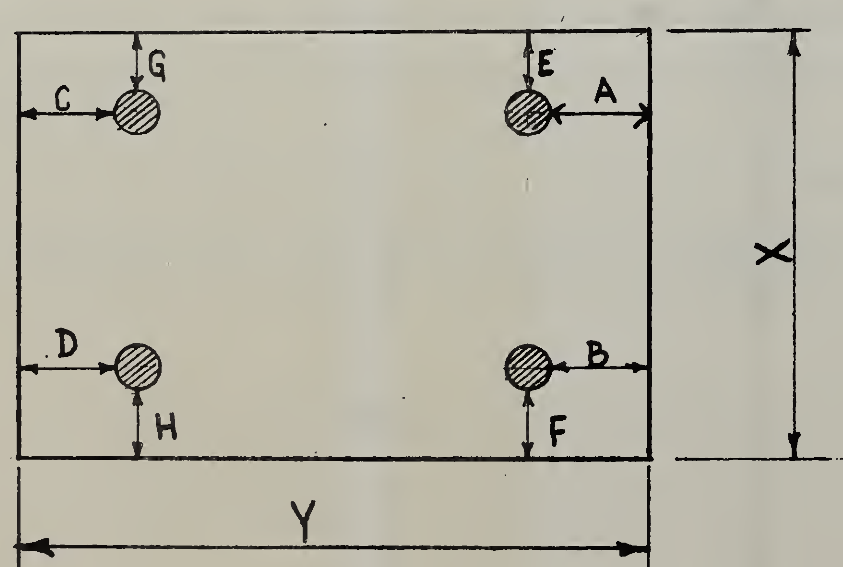


TABLE 10.2

COLUMN TESTS
(Ultimate Loads and Failure Modes)

Column No.	Major Axis Eccentricity (in.)	Minor Axis Eccentricity (in.)	Ultimate Load (kips)	Type of Failure
1	0.0	0.5	73.0	Bottom Fixture
5	0.0	0.5	82.0	Concrete Mid-ht.
8	0.0	0.5	81.8	Concrete Mid-ht.
			Avg.= 78.9	
2	2.0	0.0	56.6	Top fixture
6	2.0	0.0	51.0	Top fixture
7	2.0	0.0	45.9	Top fixture
			Avg.= 51.2	
9	1.5	0.0	89.5	Concrete Mid-ht.

TABLE 10.3

BEAM TESTS
(Insert Spacing and Concrete Strength of Specimens)

Beam No	Inserts	Date Cast Beam	Date Cast Topping	Date Beam Tested	Concrete Strength, psi* Beam	Concrete Strength, psi* 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.

*** All beams with 19" insert spacing between supports had one insert outside each support at 18" from the centerline of the support.

**** All beams with 9 1/2" insert spacing between supports had three inserts outside each support at 6", 18" and 30" from the centerline of the support.

Notes on Inserts:

1. Star inserts were 3/8" zinc base, die-casting alloy
2. Richmond inserts were 3/8" grey cast-iron "Kohler"
3. Jam nuts were used with Star inserts
4. Cross-bars on shear-studs were 8" long when inserts were spaced at 9 1/2" between columns.

TABLE 10.4

BEAM TESTS
(Repeated Load Test Results)

Beam No	Inserts Used	Beam Mid-Span Deflections, in.*										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
11	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

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

TABLE 10.5

BEAM TESTS
(Modes of Insert Failures)

Beam No.	Inserts	Mode of Insert Failure
5	Star at 19"	Inserts broken 1" below interface and pulled out
7	Star at 19"	Inserts broken 1" below interface and pulled out
9	Star at 19"	Bolts broken just above jam nuts
8	Richmond at 19"	Bolts sheared off at slab-beam interface
10	Richmond at 19"	Bolts sheared off at slab-beam interface
6	Richmond at 9 1/2"	Bolts sheared off at slab-beam interface
11	Richmond at 9 1/2"	Bolts sheared off at slab-beam interface

TABLE 11.1

CONCRETE TESTS
(Concrete Compressive and Splitting Strength*)

Concrete	Batch	Date Tested	Age at Test	Compressive Strength		Splitting Strength
			days	Specified psi	Actual psi	psi
Precast Component**	4/16 - B	May 10	24	3500	5100	346***
Precast Component**	4/16 - C	May 10	24	3500	4930	287***
Precast Component**	4/17 - A	May 10	23	3500	7160	369***
Precast Component**	4/17 - B	May 10	23	3500	7090	------
Floor Slab	---	May 10	17	2000	5600	------
Topping Slab	A	May 21	26	3000	3840	------
Topping Slab	B	May 21	26	3000	2560	------
Avg. Value for Typical Lightweight Aggregate Concrete****			28		5800	

* 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.

** Concrete for the long columns (first story) was specified to be 4500 psi strength.

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

**** Derived from NBS Monograph 74, March, 1964.

TABLE 11.2

CONCRETE TESTS
(Unit Weights and Air Contents of Concrete)

Concrete	Batch	Date Tested	Age	Air Drying Period	Unit Weight	Air Content
			days	days	pcf	%
Precast Component	4/16 - B	May 16	30	21	98.5	11.3
Precast Component	4/16 - C	May 16	30	21	96.4	----
Precast Component	4/17 - A	May 16	29	21	103.7	----
Precast Component	4/17 - B	May 16	29	21	103.4	8.2
Floor Slab	----	May 14	21	20	147.7	----
Topping Slab	A	May 16	21	20	117.7	----
Topping Slab	B	May 16	21	20	108.3	----
Average values for typical lightweight aggregate concrete*			28	27	97.0	4.7

*Derived from NBS Monograph 74, March, 1964.

TABLE 11.3

CONCRETE TESTS
(Modulus of Elasticity of Precast Component Concrete)

Batch	Age at Test	Date Tested	Compressive Strength (f'c)	Secant Modulus*
	days		psi	10^6 psi
4/16 - B	41	May 27	6400	2.1
4/16 - C	41	May 27	6030	2.2
4/17 - B	40	May 27	7380	2.4
Avg. value for Lightweight Aggregate Concrete**	28		5850	2.2

* Secant Modulus at 0.4 f'c after prior loading to 0.5 f'c several times.

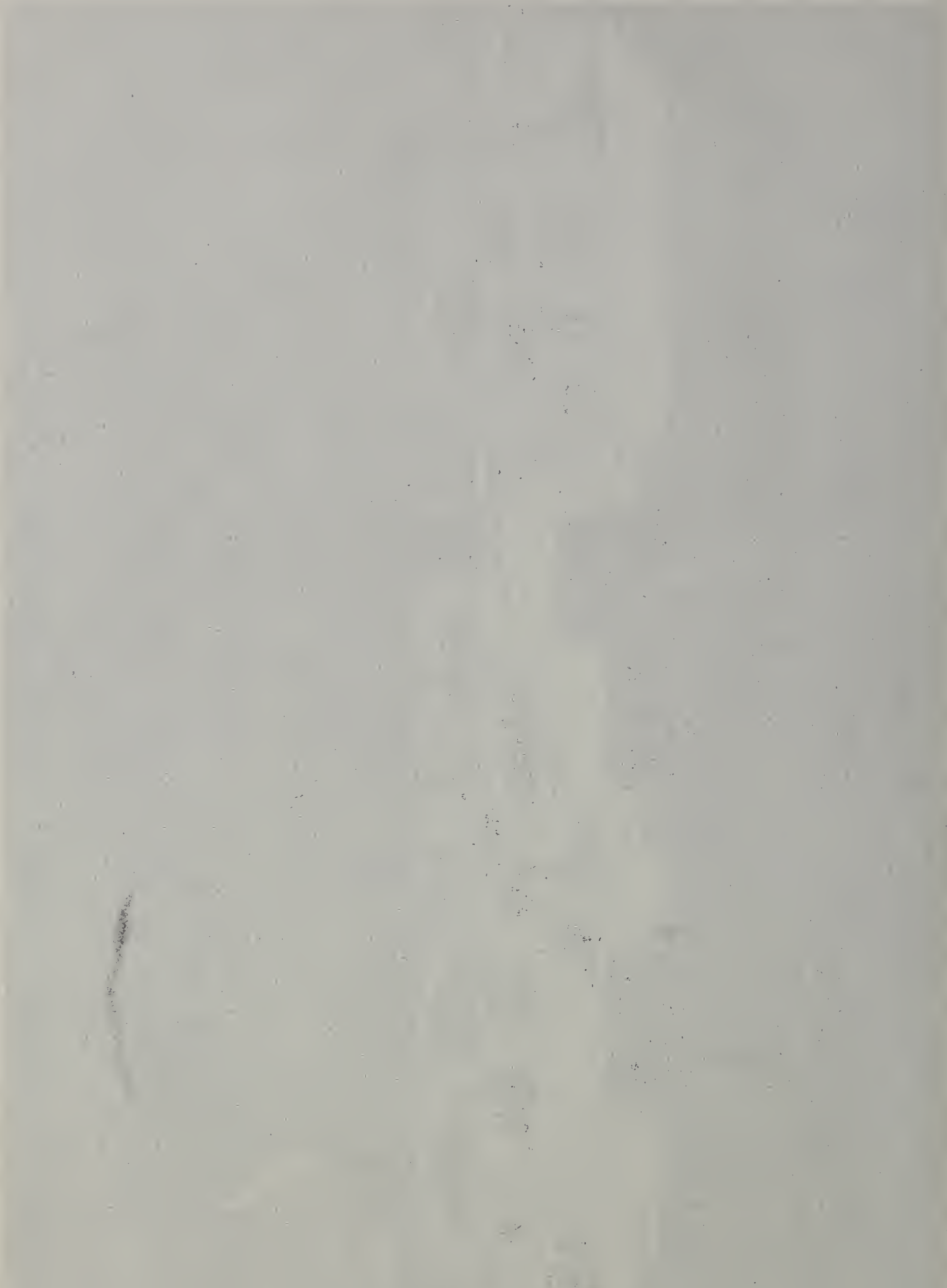
**Derived from NBS Monograph 74, March, 1964.

TABLE 11.4

REINFORCING STEEL TEST RESULTS

Specimen	Where Used	Yield Strength		Ultimate Strength
		Specified psi	Actual psi	psi
No. 5 Deformed Bar	Beams	60,000	65,000	96,000
No. 4 Deformed Bar	Cantilever Beams	60,000	73,000	117,000
No. 2 Plain Bar	Tie Bars in Columns	40,000	47,000	77,000

FIGURES



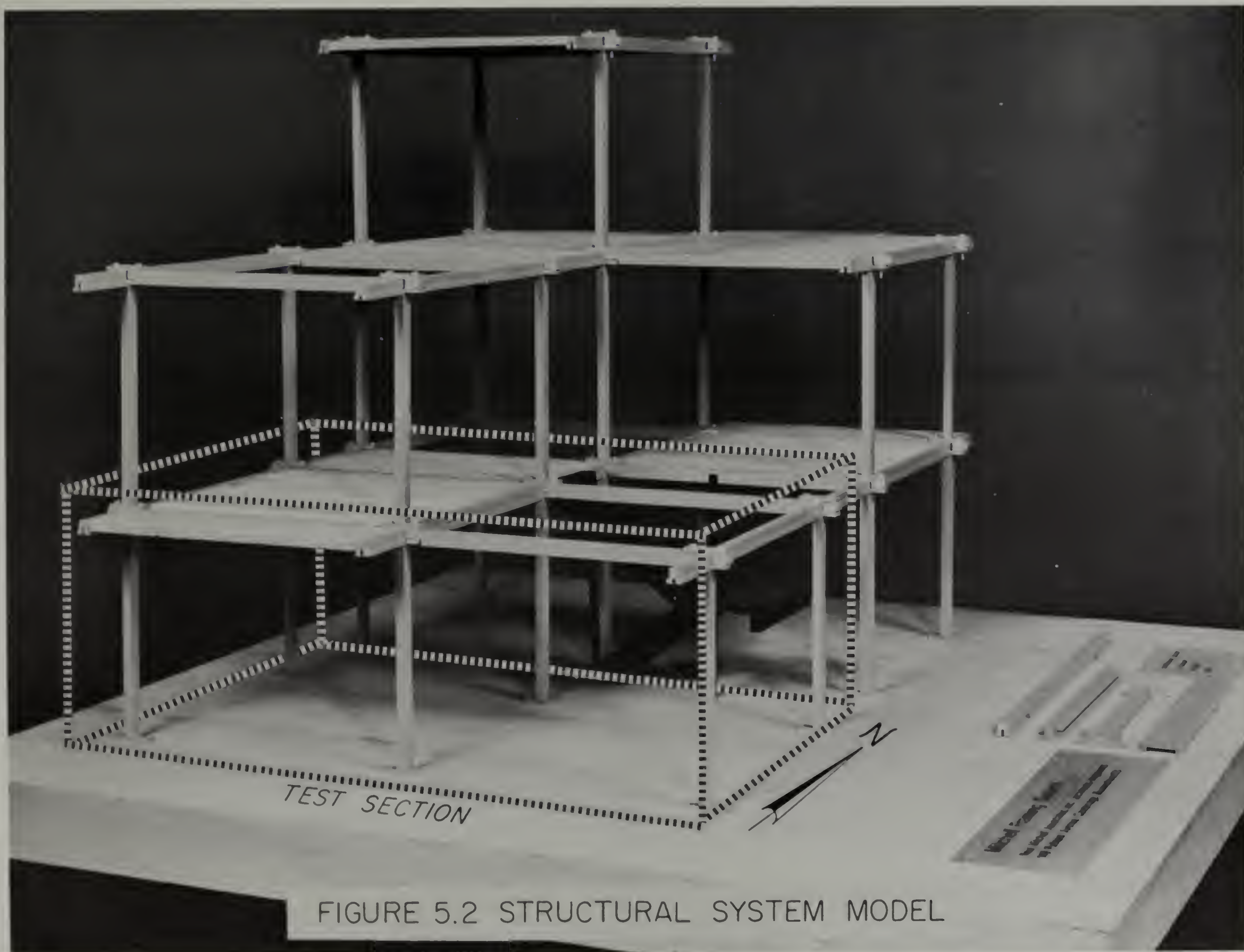


FIGURE 5.2 STRUCTURAL SYSTEM MODEL

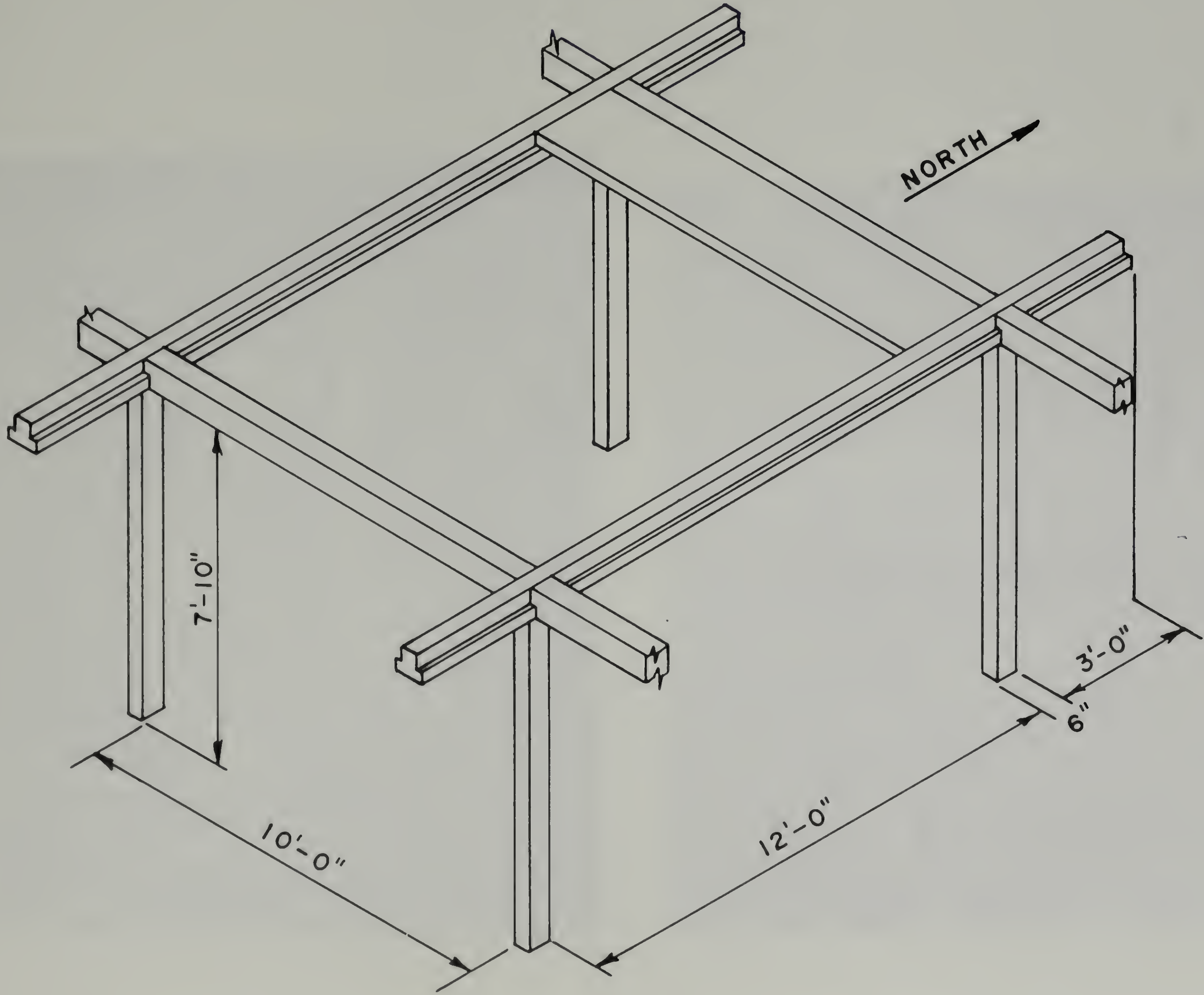


FIGURE 5.3 TYPICAL BAY OF TWO BAY FRAME TEST

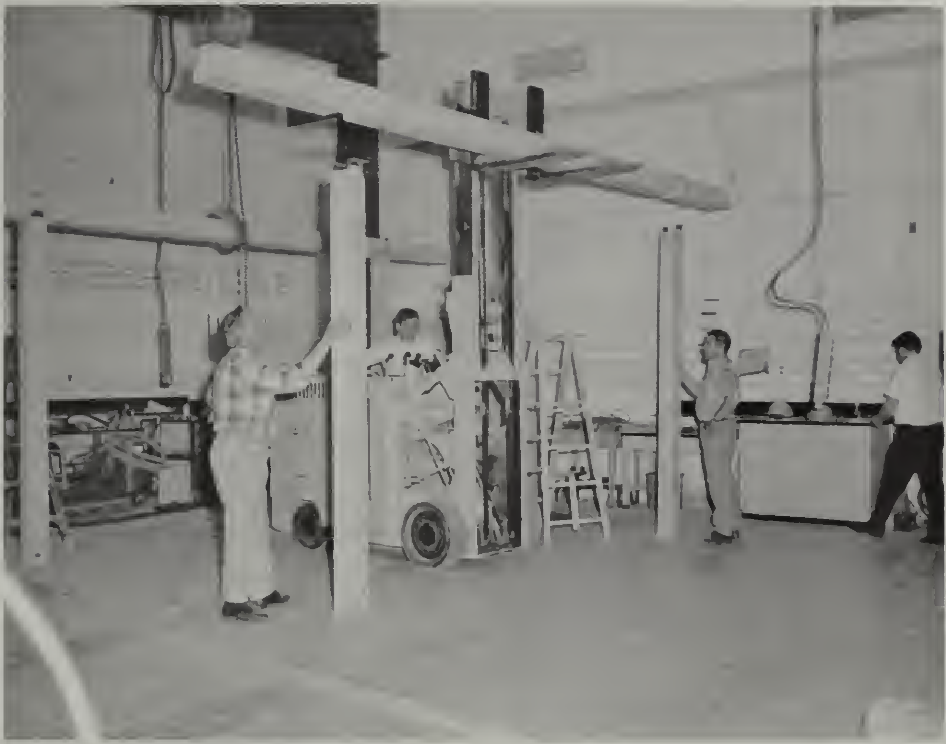


FIGURE 5.4 ASSEMBLING STRUCTURAL FRAME

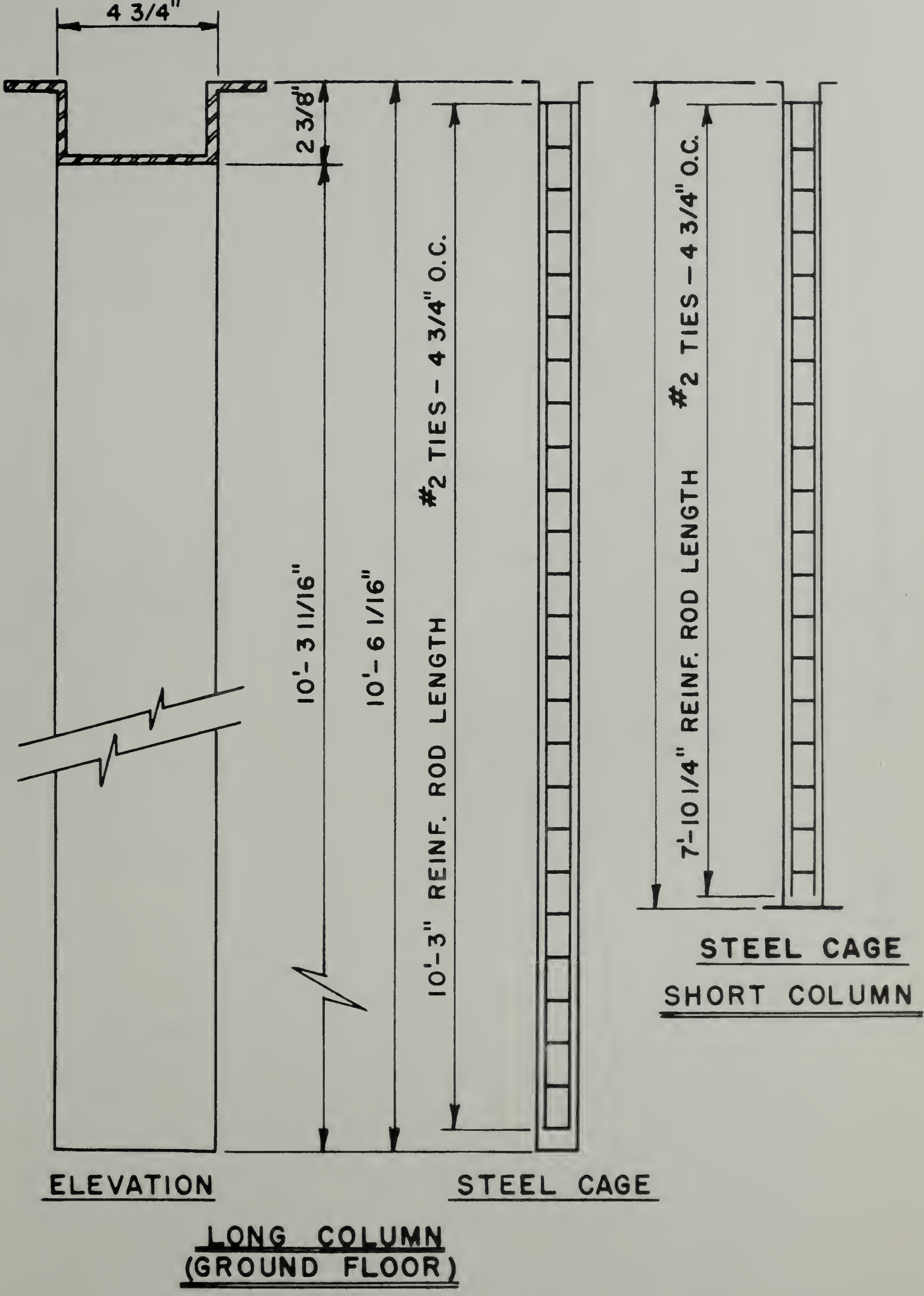


FIGURE 5.6 ELEVATION OF COLUMN REINFORCEMENT

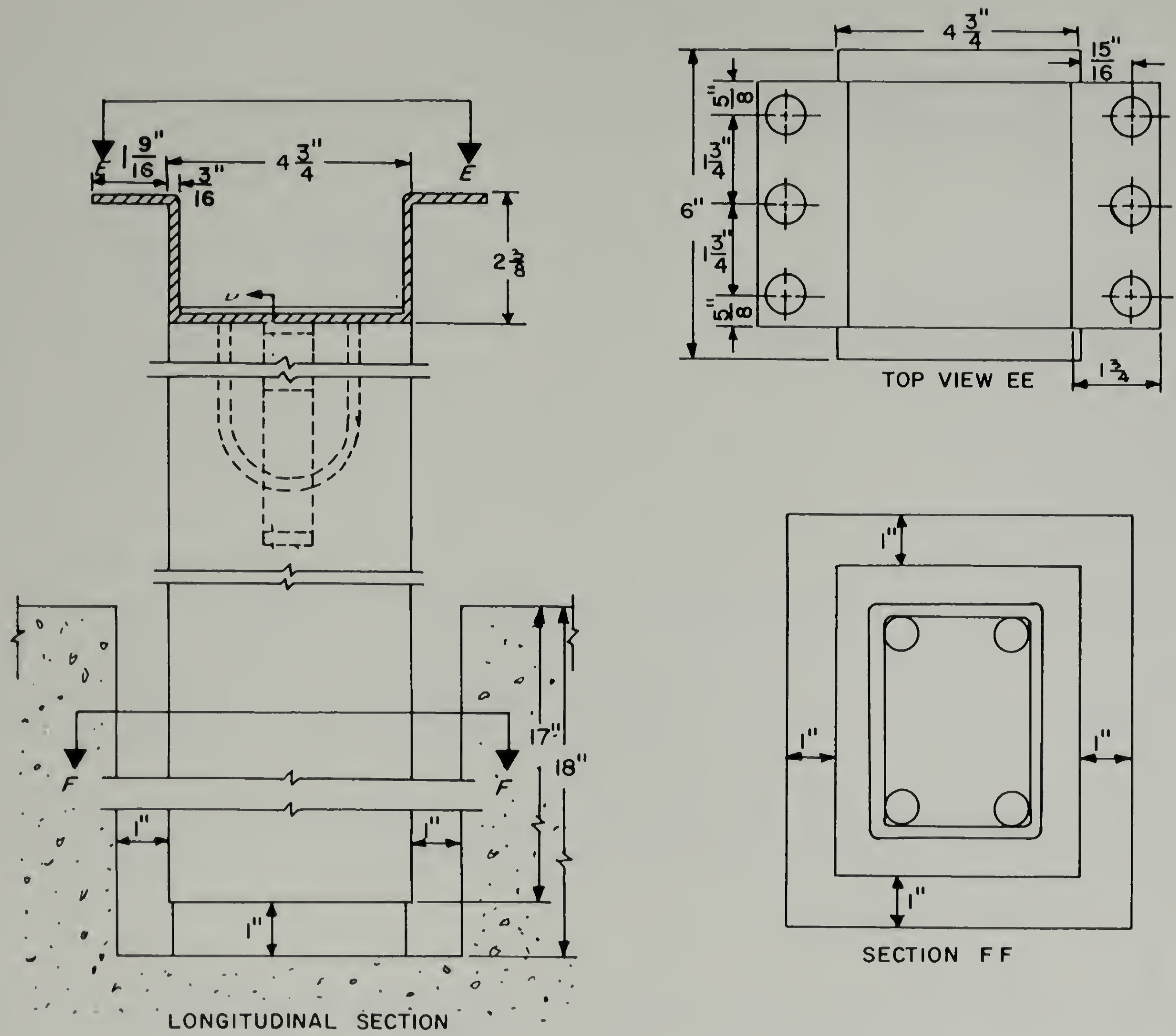


FIGURE 5.7 LONG COLUMN DETAILS

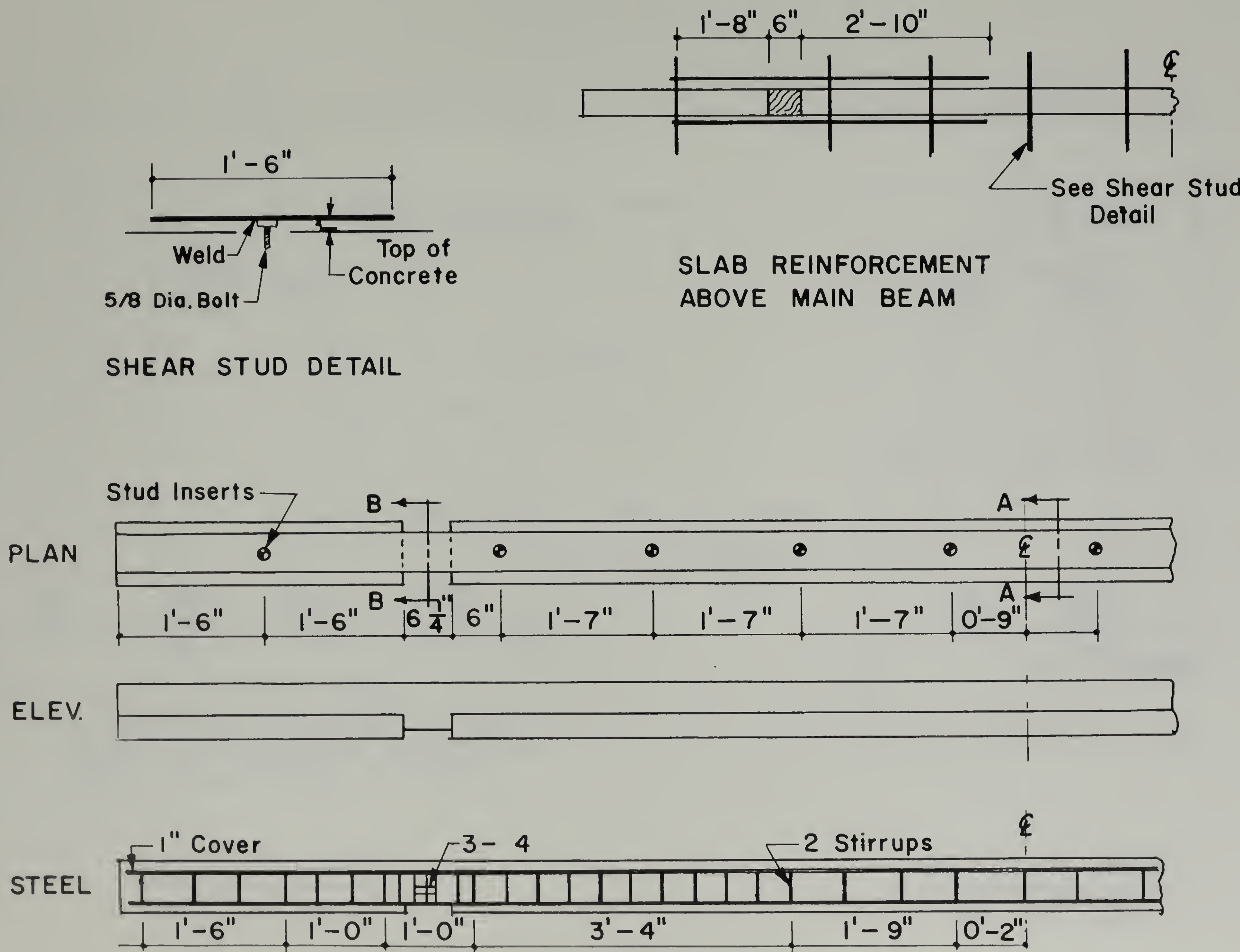
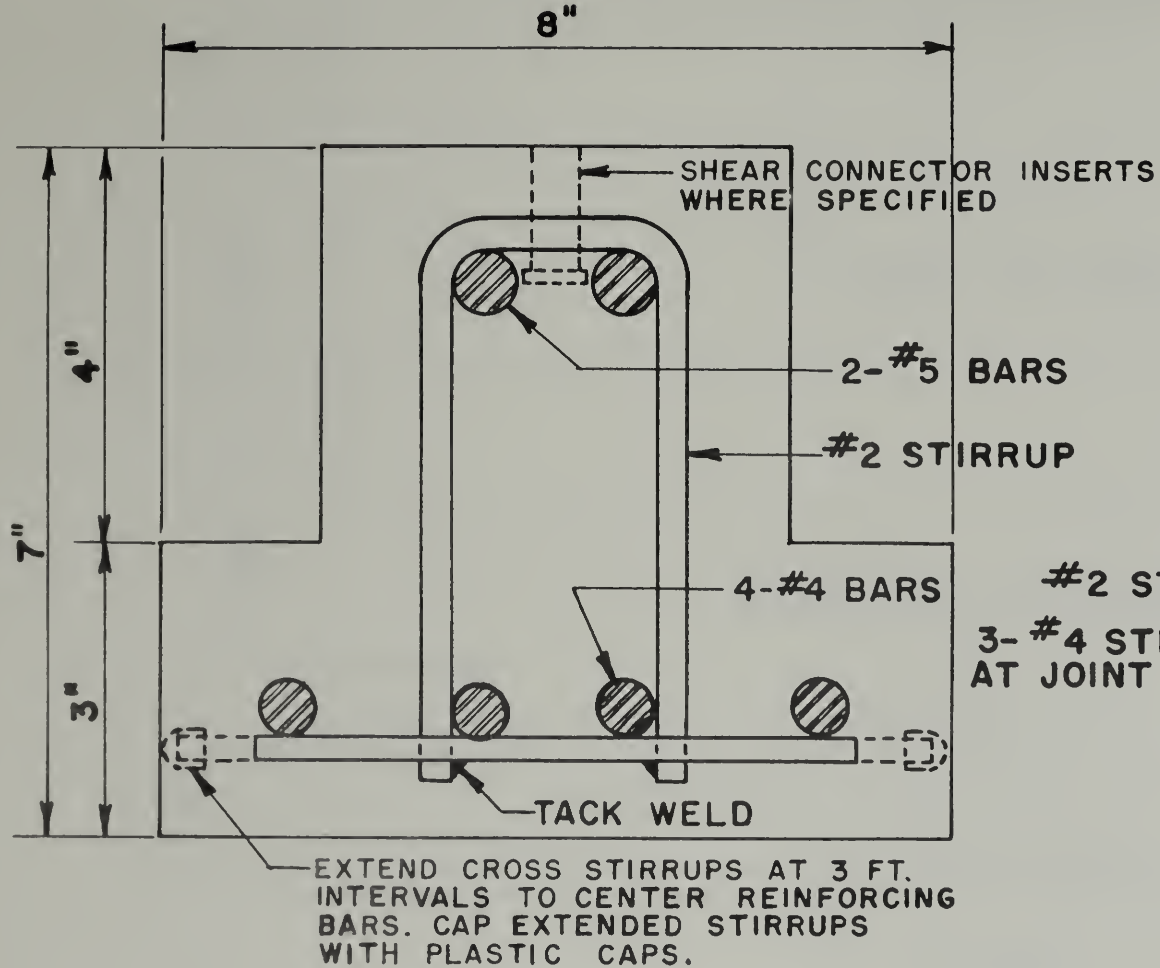
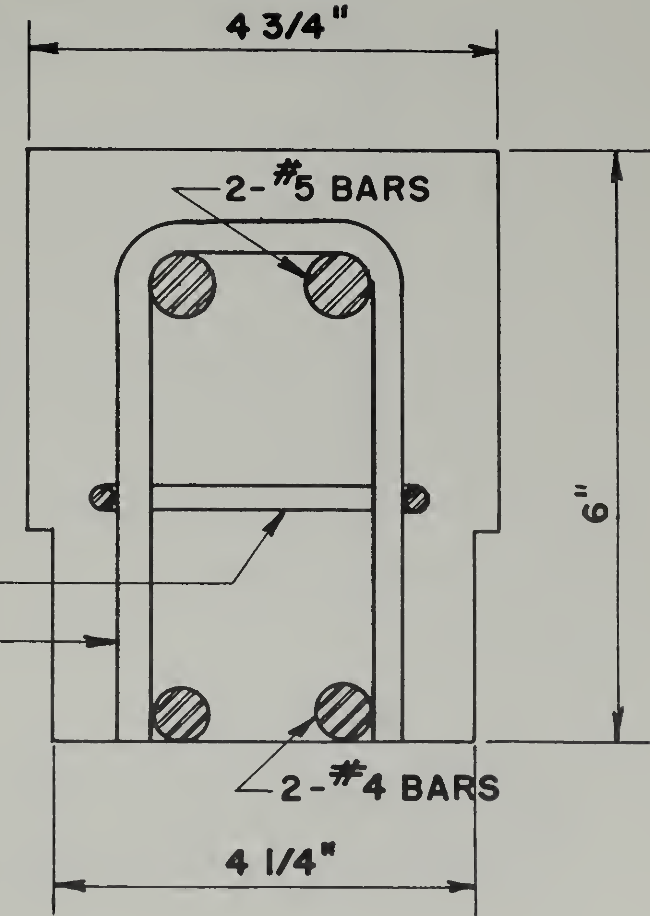


FIGURE 5.8 MAIN BEAM REINFORCEMENT



SECTION "A-A"

(SEE FIG. 5.8)



SECTION "B-B"

(SEE FIG. 5.8)

FIGURE 5.9 MAIN BEAM SECTIONS

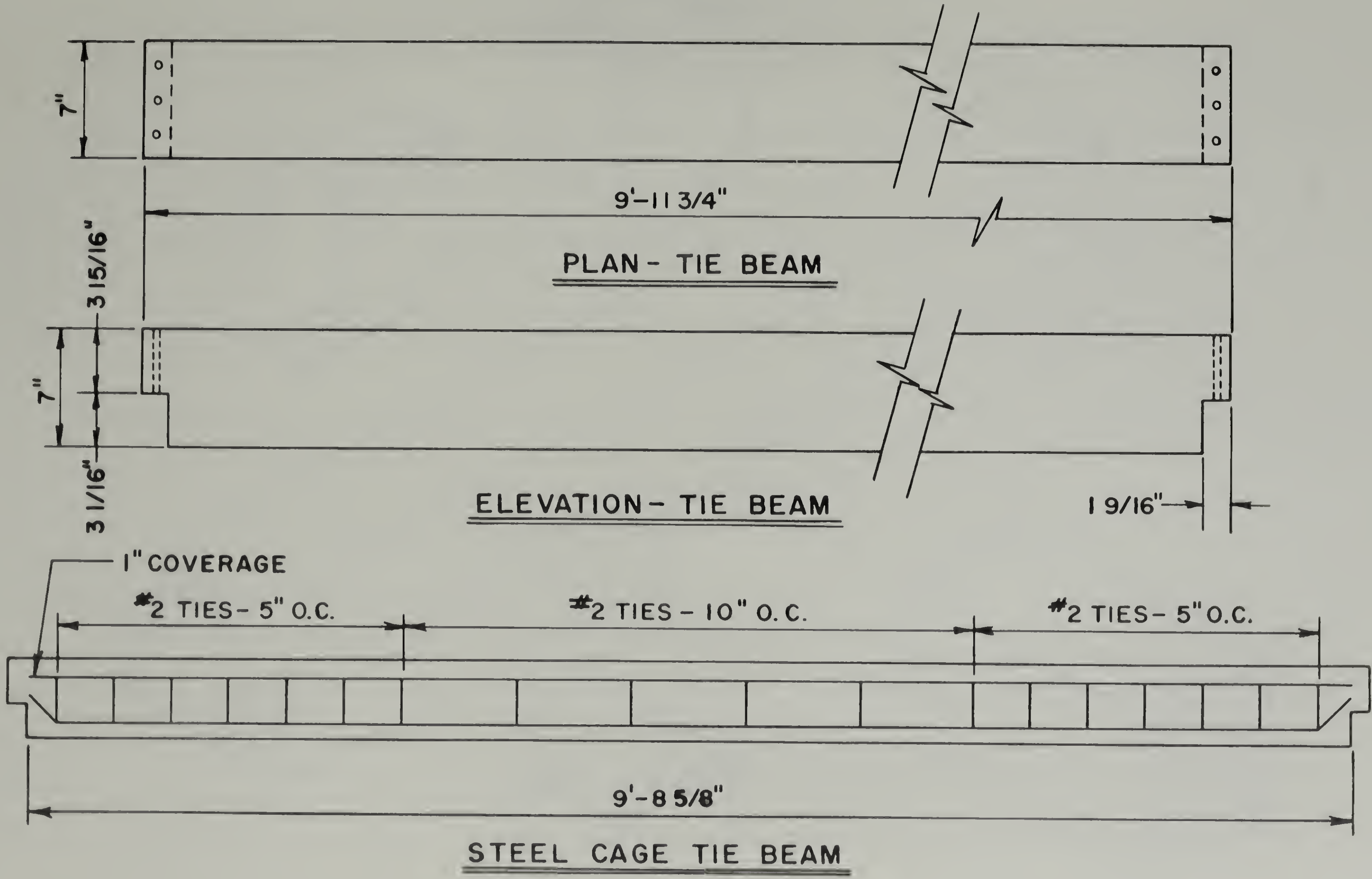


FIGURE 5.10 TIE BEAM PLAN AND ELEVATION

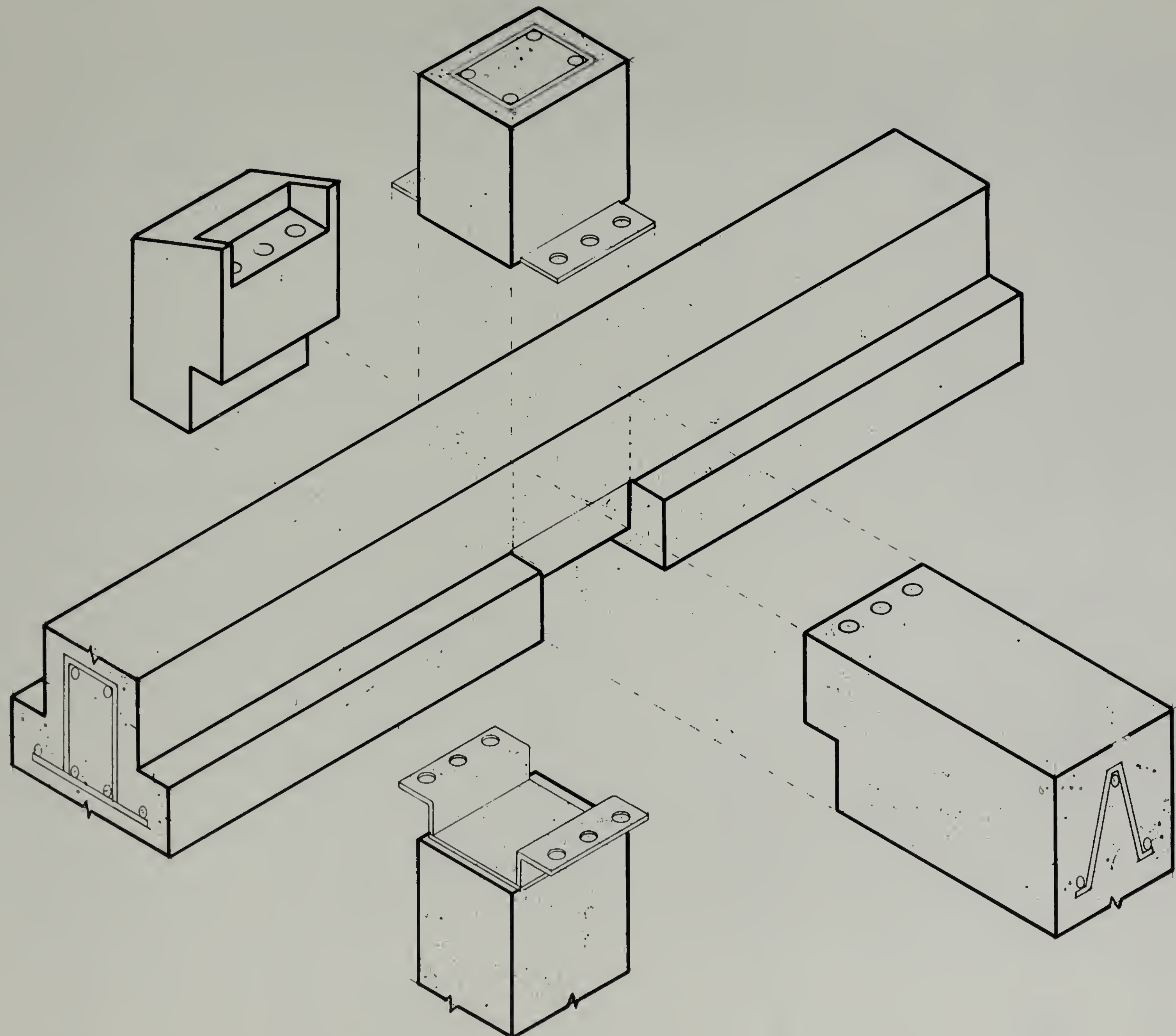


FIGURE 5.11 BEAM AND COLUMN CONNECTION

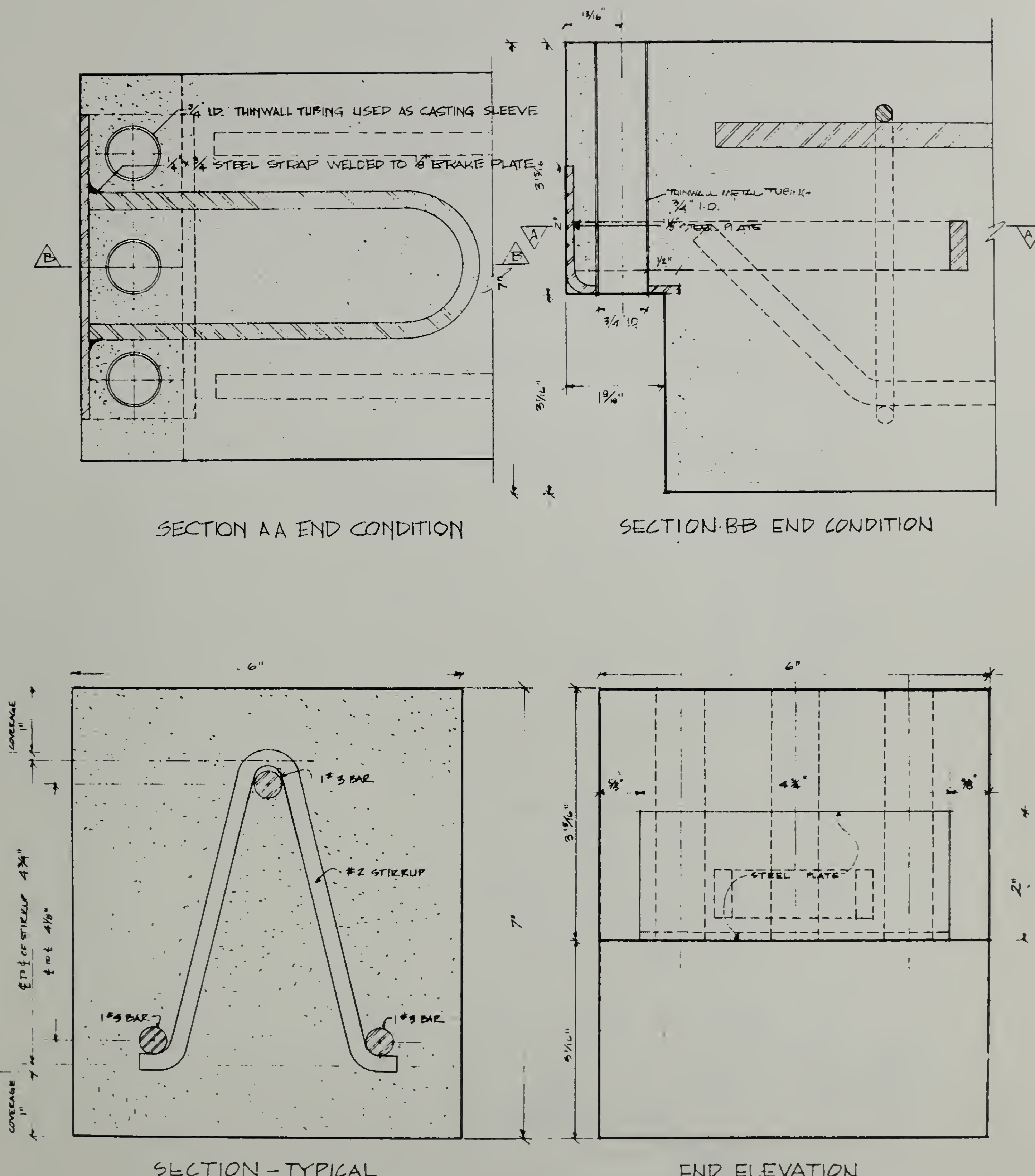


FIGURE 5.12 TIE BEAM END DETAILS

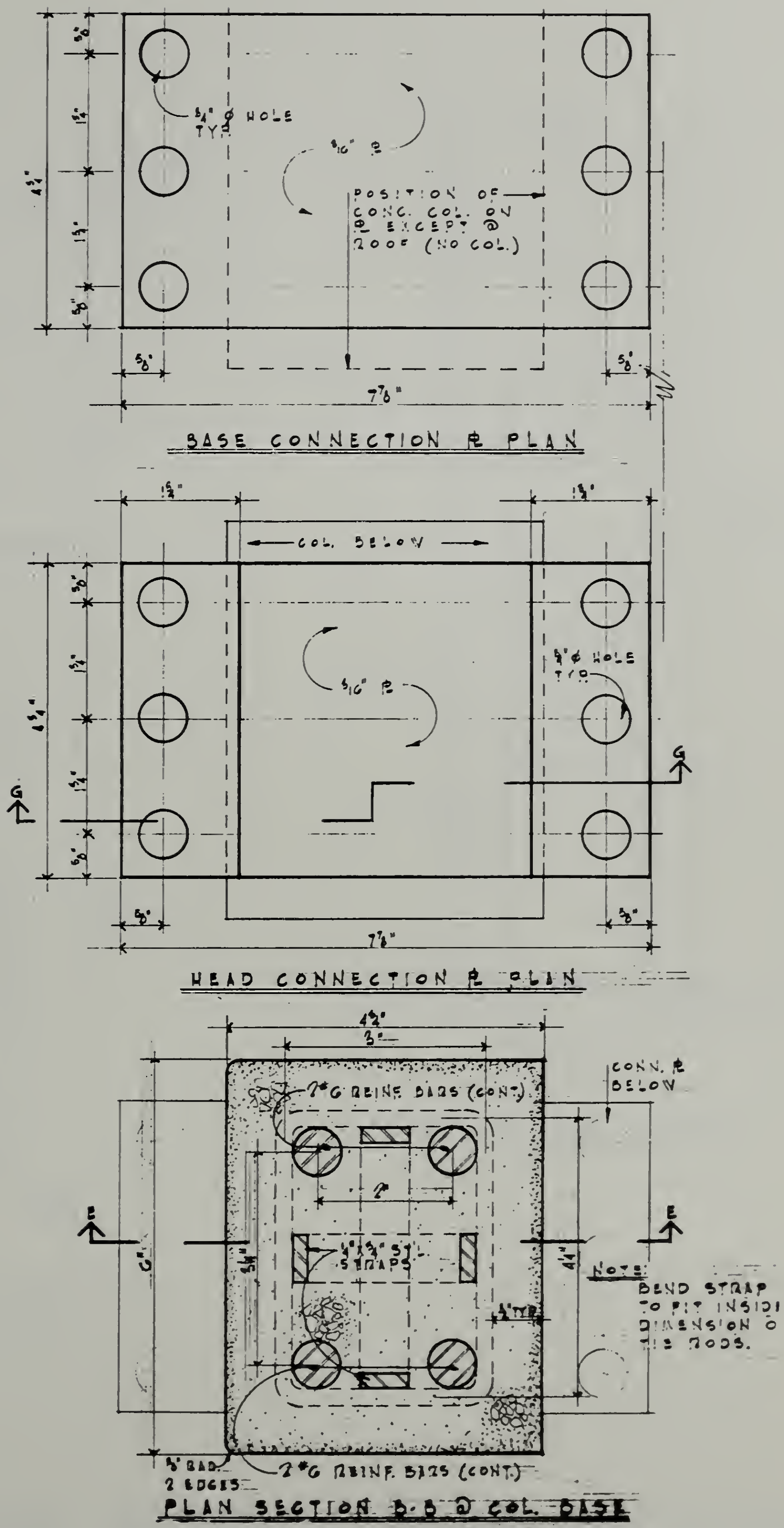


FIGURE 5.13 PLAN SECTIONS OF COLUMN CONNECTIONS

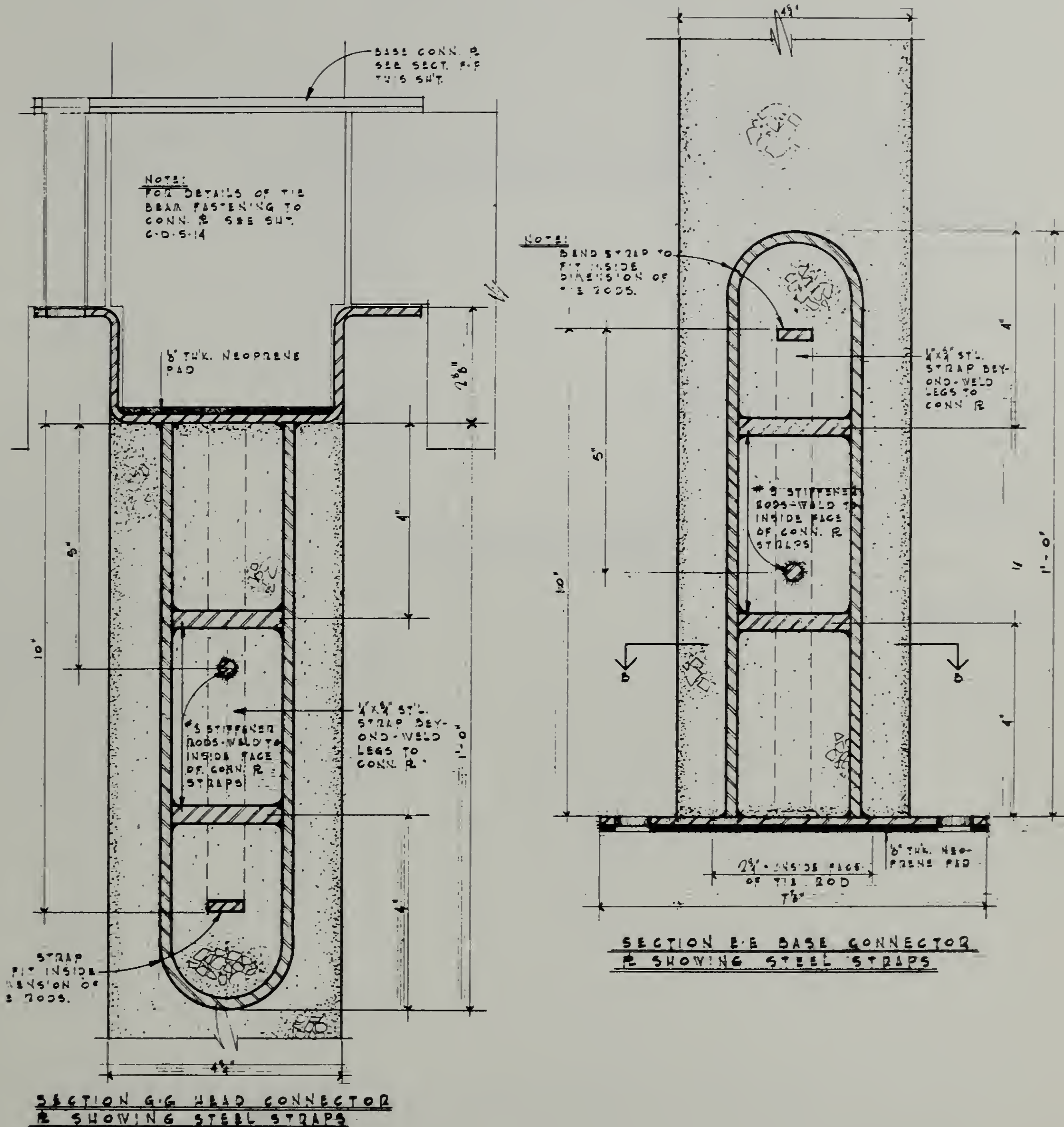


FIGURE 5.14 ELEV. SECTIONS OF COLUMN CONNECTIONS

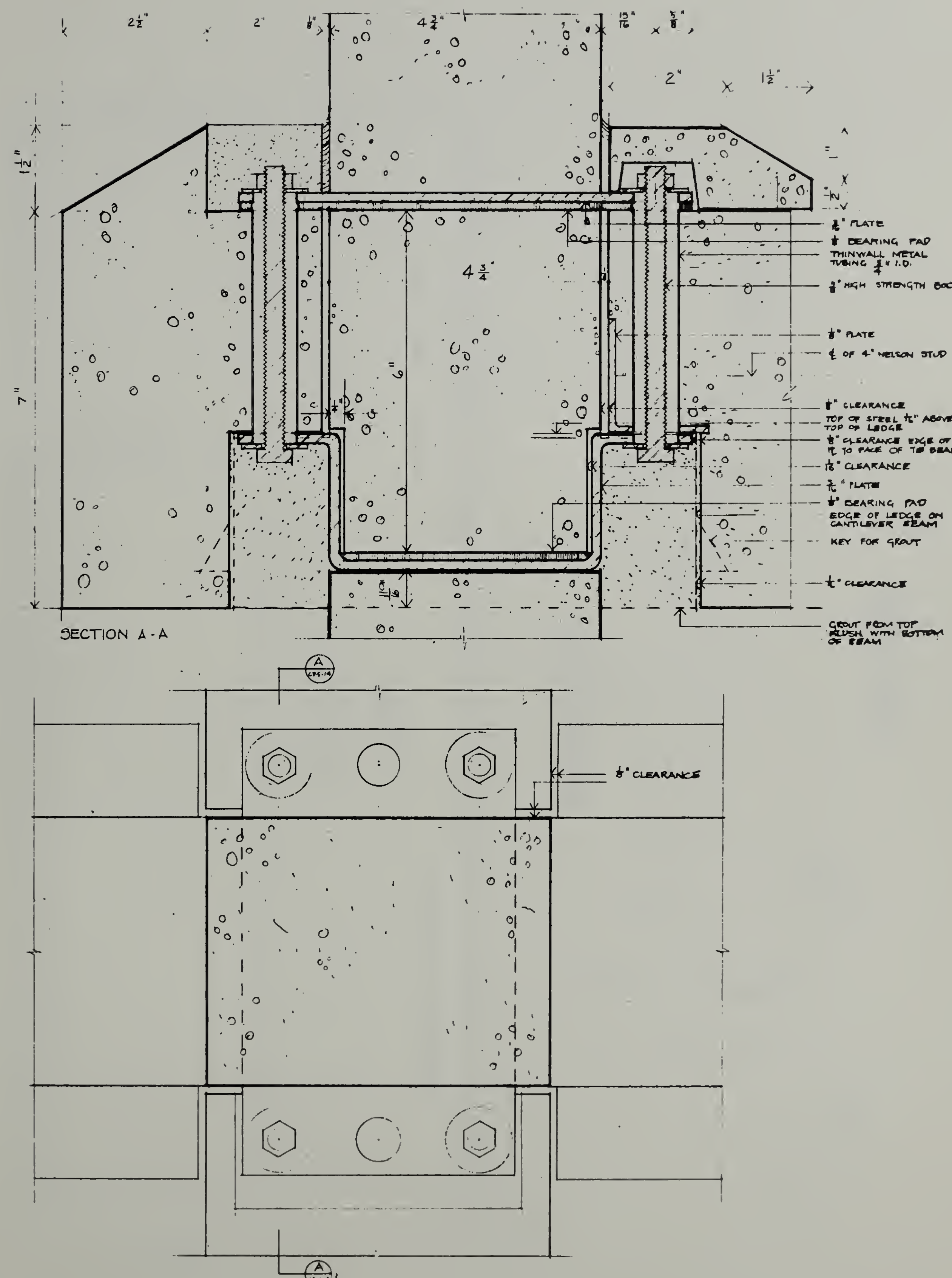


Fig. 5.15 Connection detail

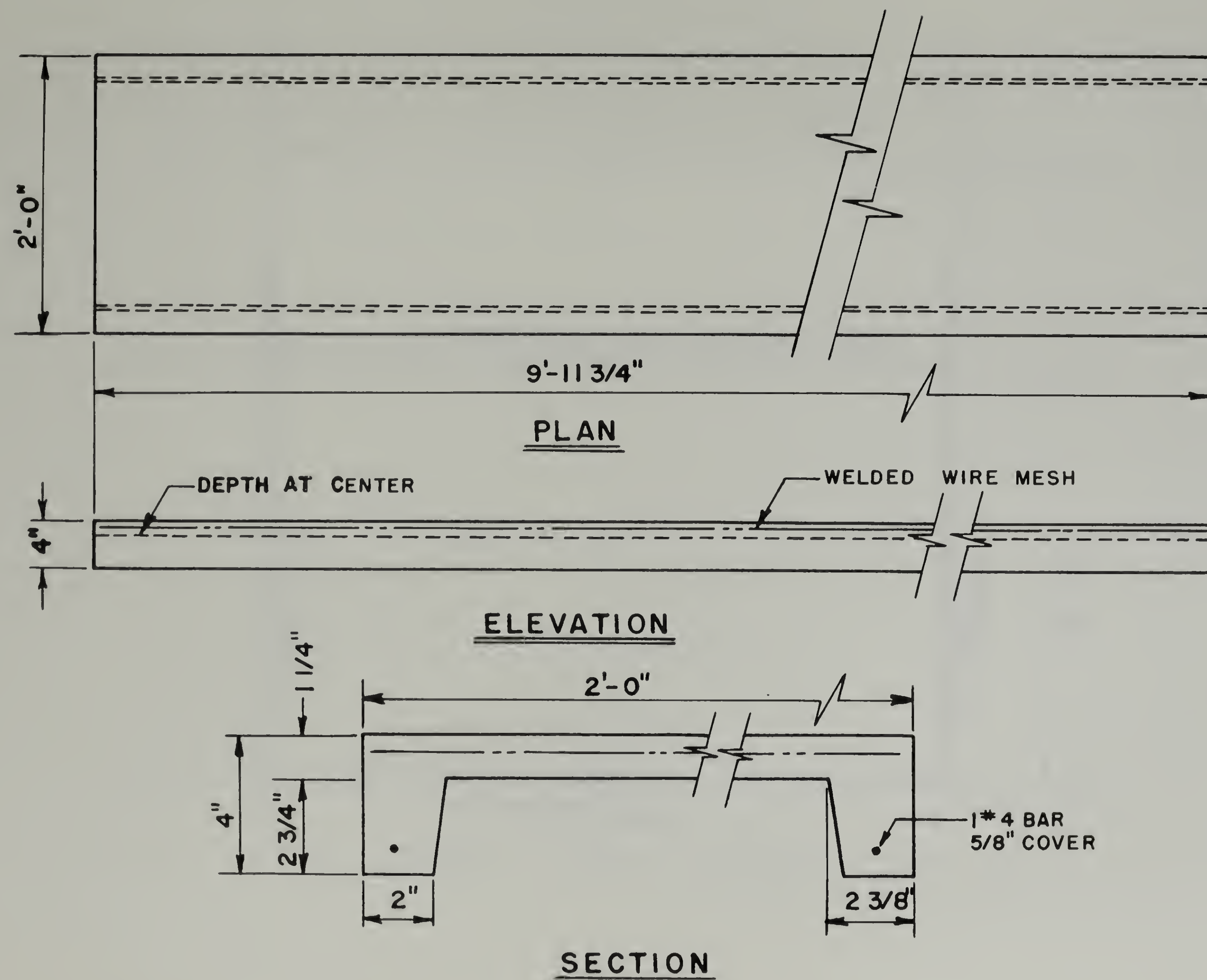


FIGURE 5.16 FLOOR CHANNELS

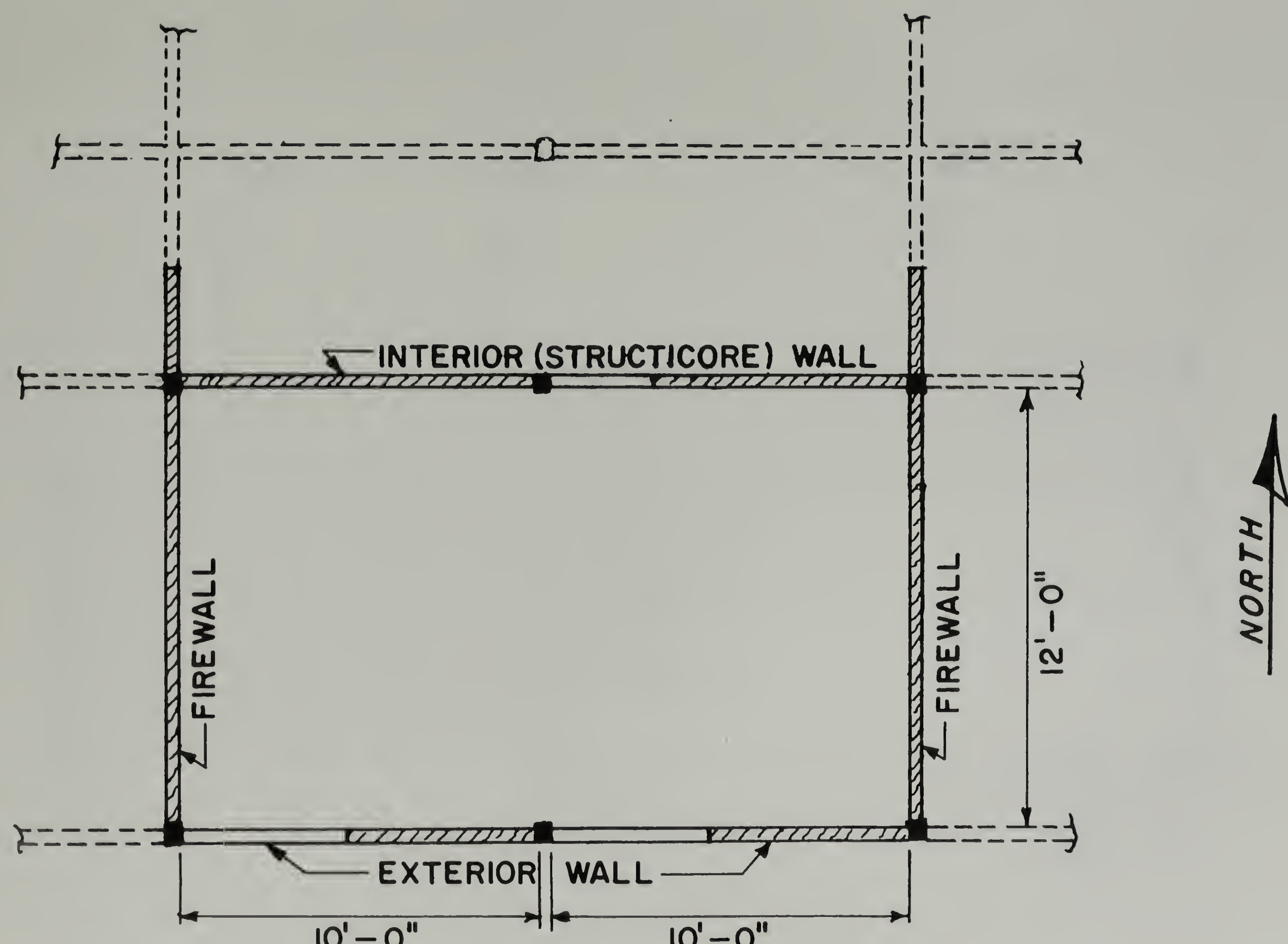
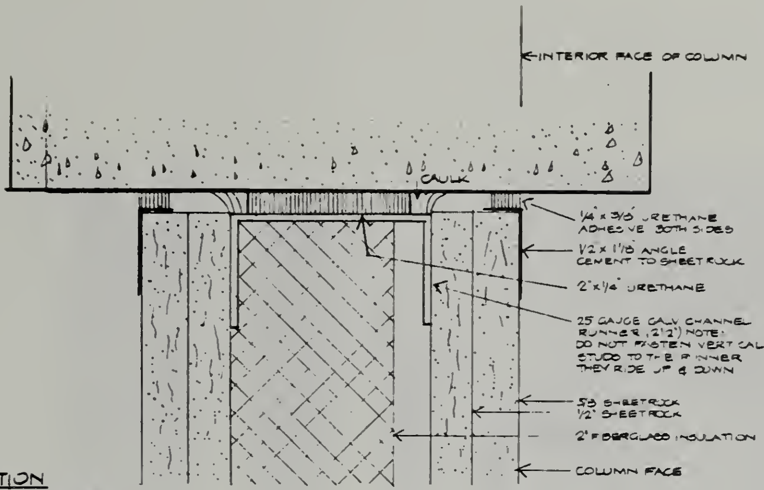
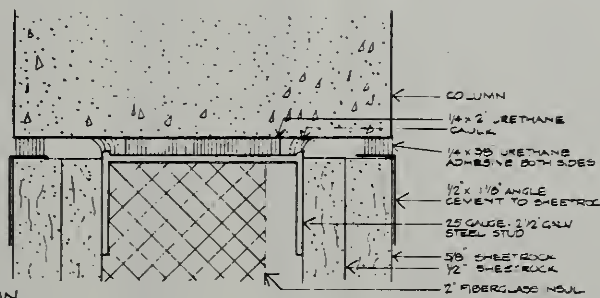


FIGURE 5.17 PLAN SECTION THROUGH WALLS



3 SECTION
DETAIL AT HEAD
2 HOUR RATING



2 PLAN
DETAIL AT COLUMN
2 HOUR RATING

1 SECTION
DETAIL @ SILL
2 HOUR RATING

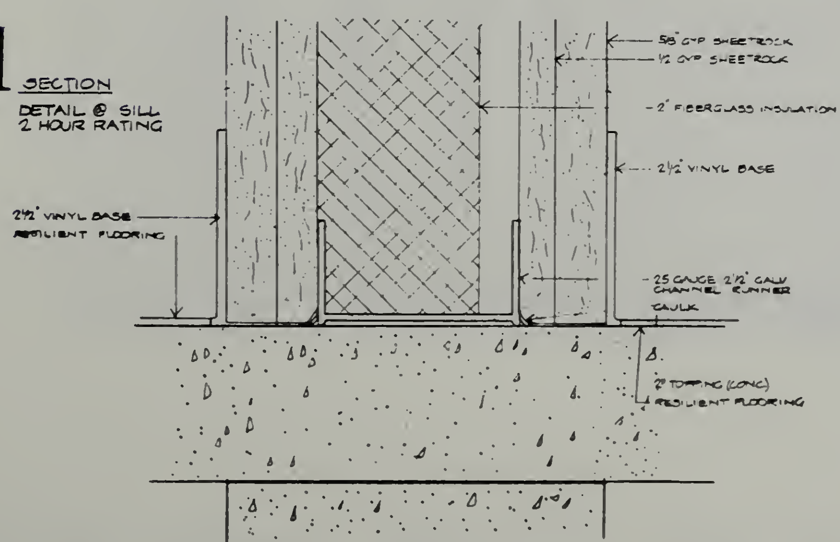
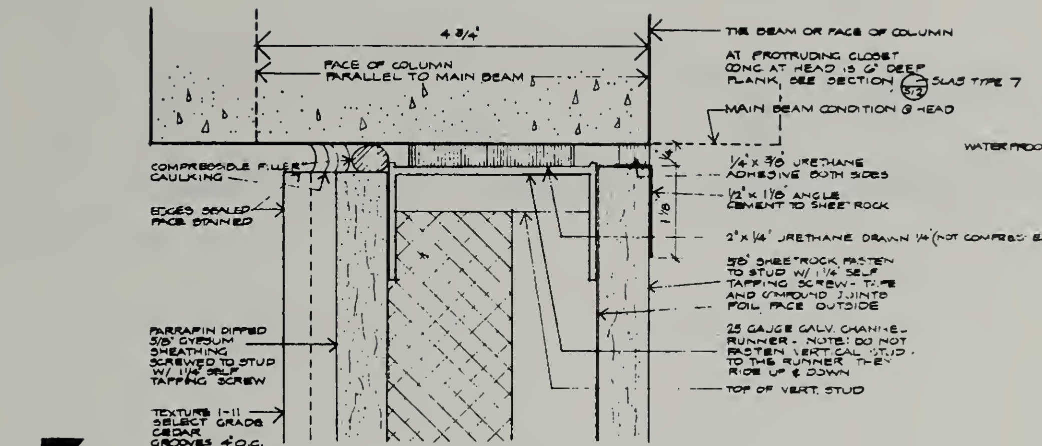


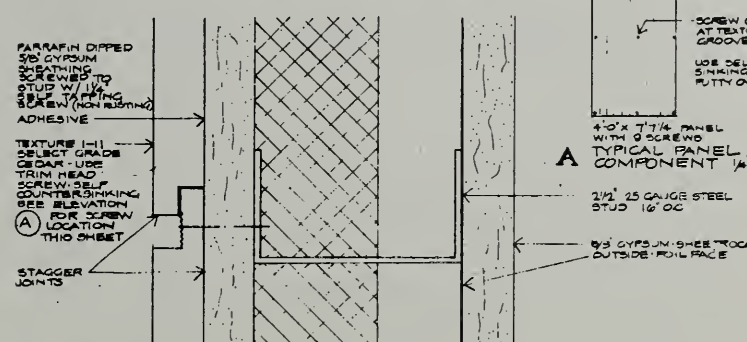
FIG. 5.18 FIRE WALL

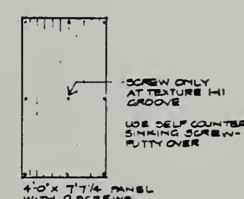
FIG. 5.19 FIREWALL (PARTIALLY DISMANTLED)



3

SECTION OR PLAN
DETAIL AT HEAD W/ SOLID PANEL OR CLOSET
DETAIL AT COLUMN W/ SOLID PANEL (JAMB)





A TYPICAL PANEL COMPONENT 1/4"=1'-0"

2

PLAN
DETAIL AT SOLID PANEL
JOINT IN TEXTURE 1-11

1

SECTION
DETAIL AT SILL
WITH SOLID PANEL
OR CLOSET

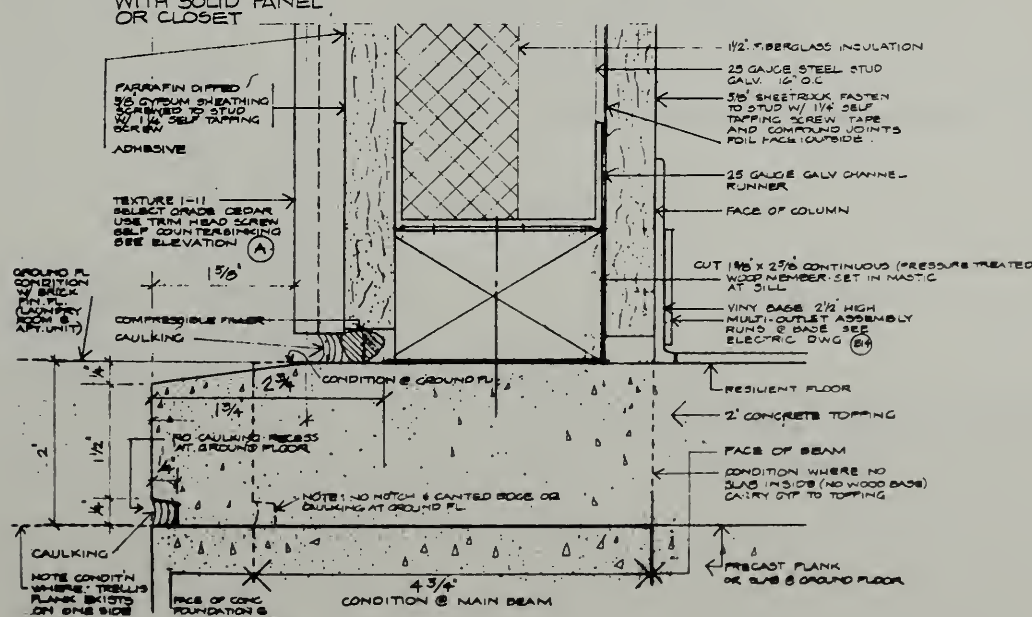


FIG. 5.20 EXTERIOR WALL DETAILS

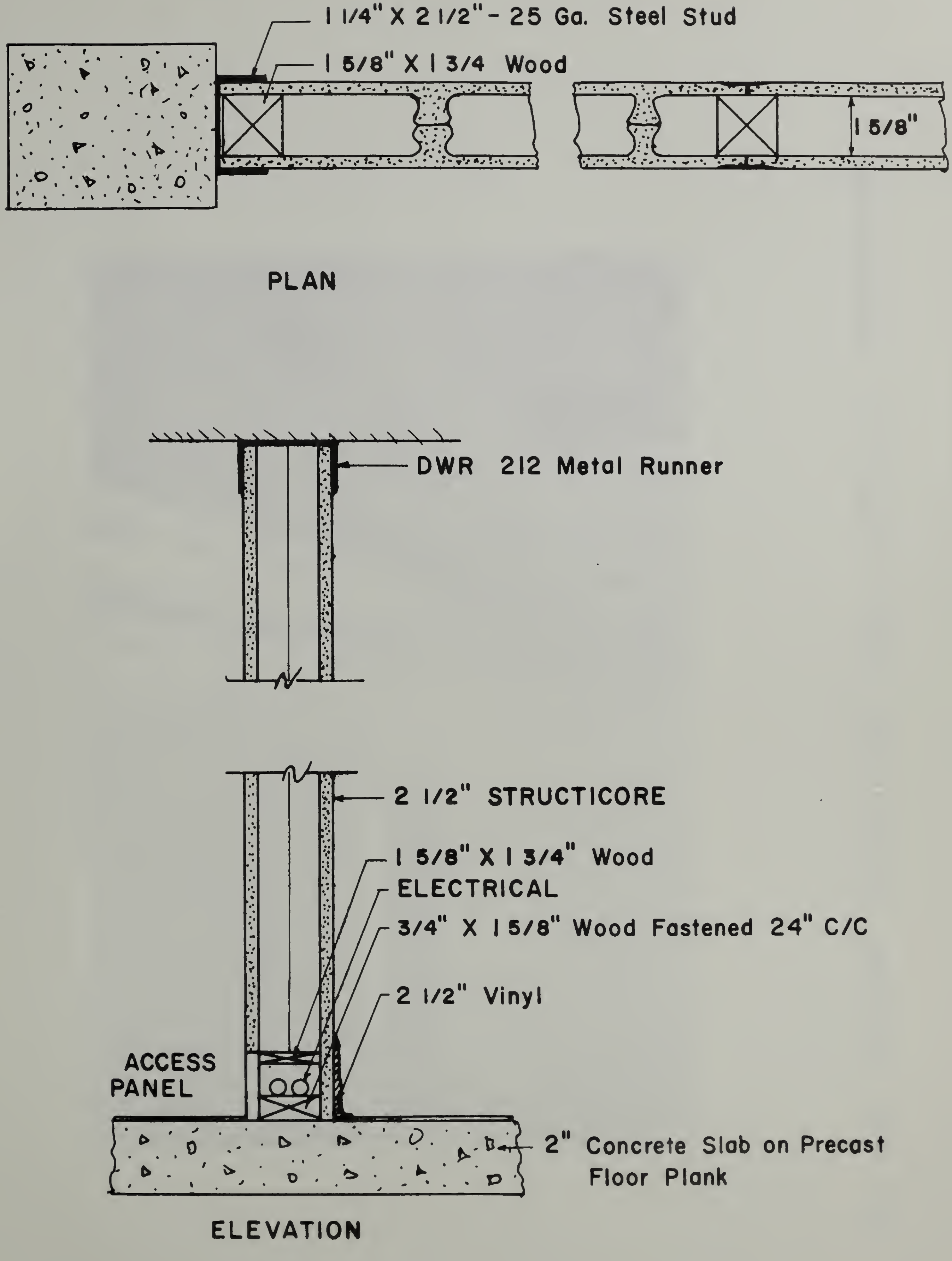


FIGURE 5.21 STRUCTICORE WALL SECTIONS

FIG. 5.22 INTERIOR "STRUCTICORE" WALL (PARTIALLY DISMANTLED)

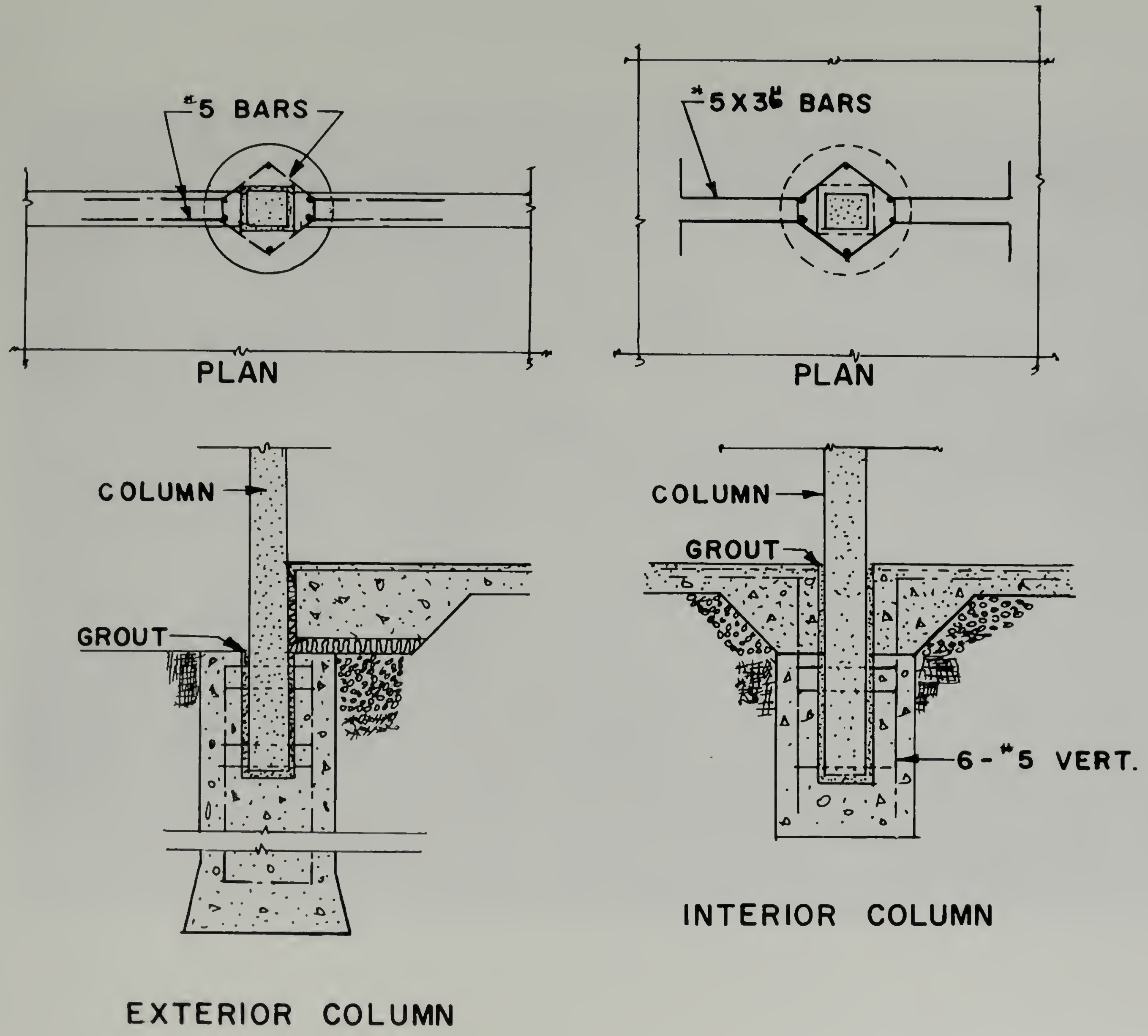


FIGURE 5.23 FOUNDATION DETAIL

FIG. 5.24 TEST STRUCTURE BEFORE THE INSTALLATION OF WALLS

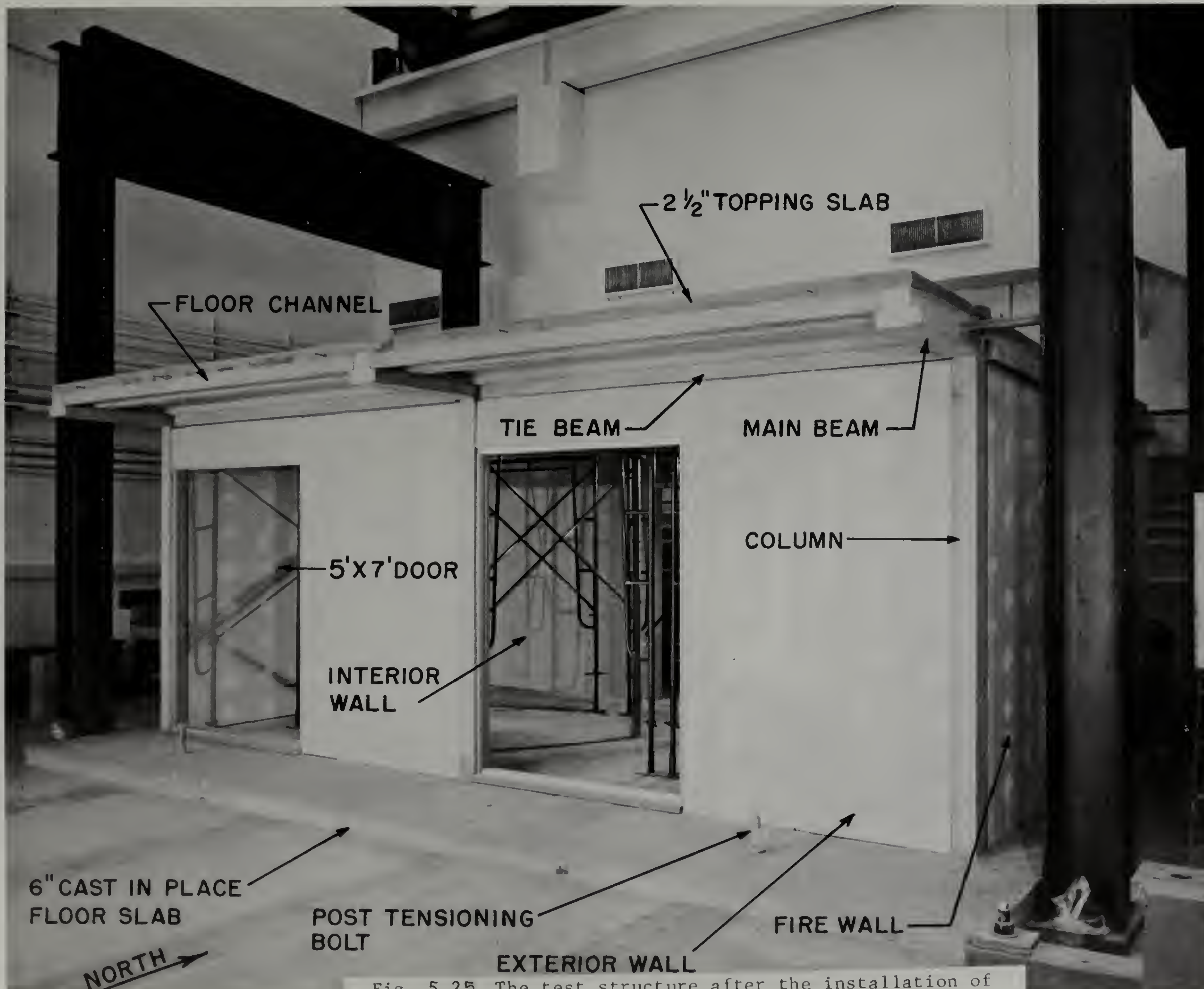


Fig. 5.25 The test structure after the installation of walls

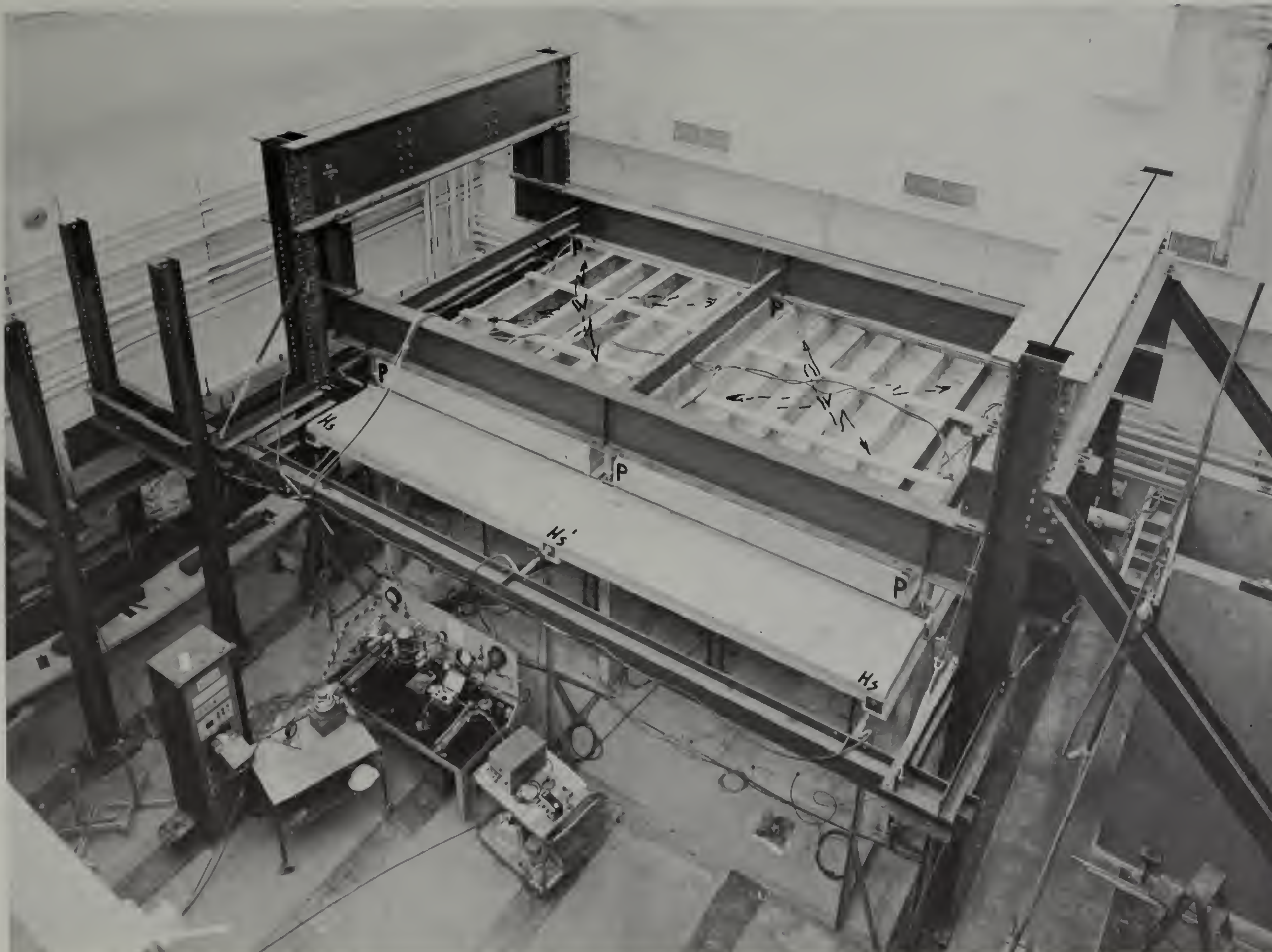


Fig. 5.26 The test structure with testing equipment installed (top view)

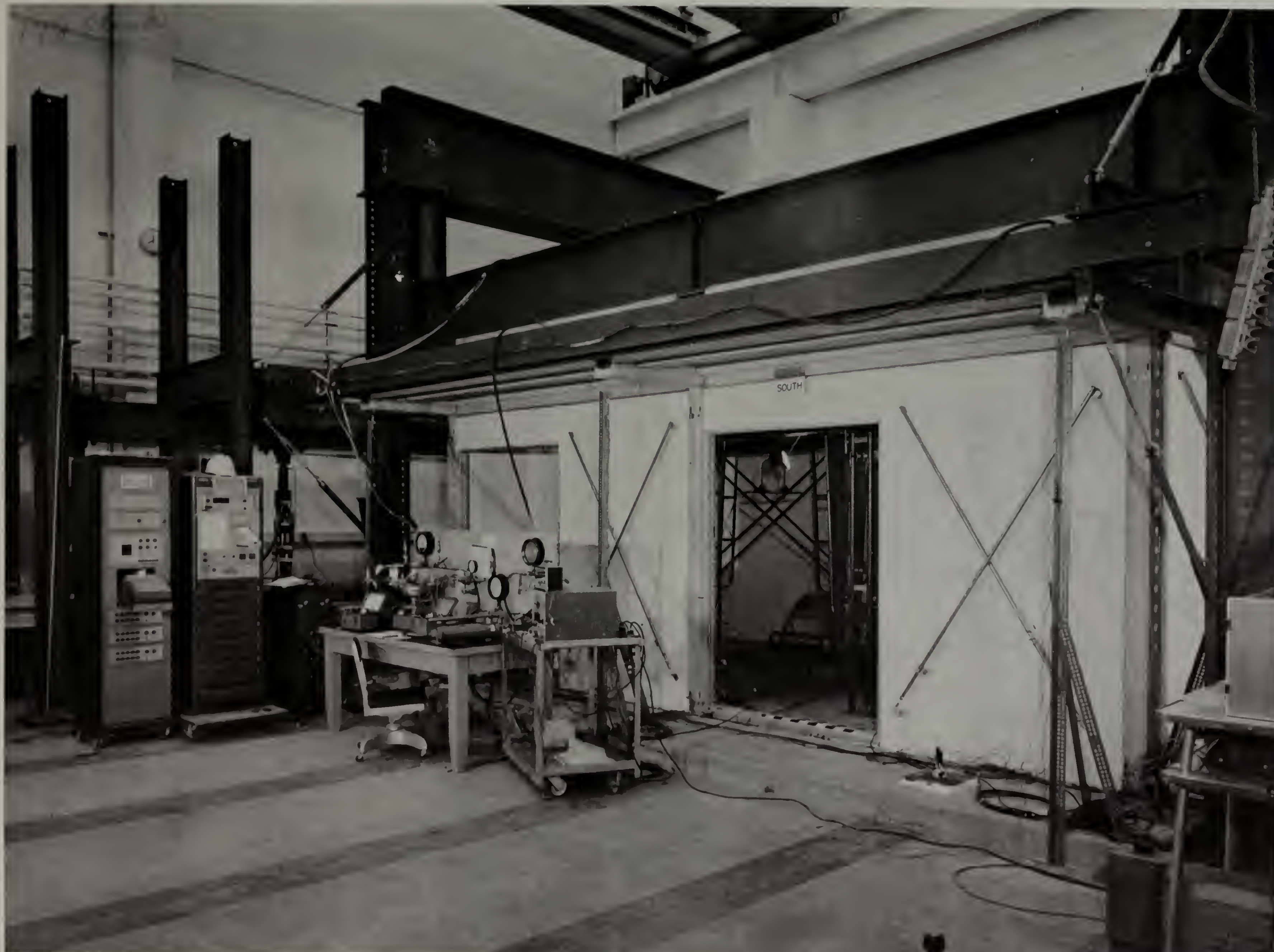


Fig. 5.27 Test structure with testing equipment installed (front view)

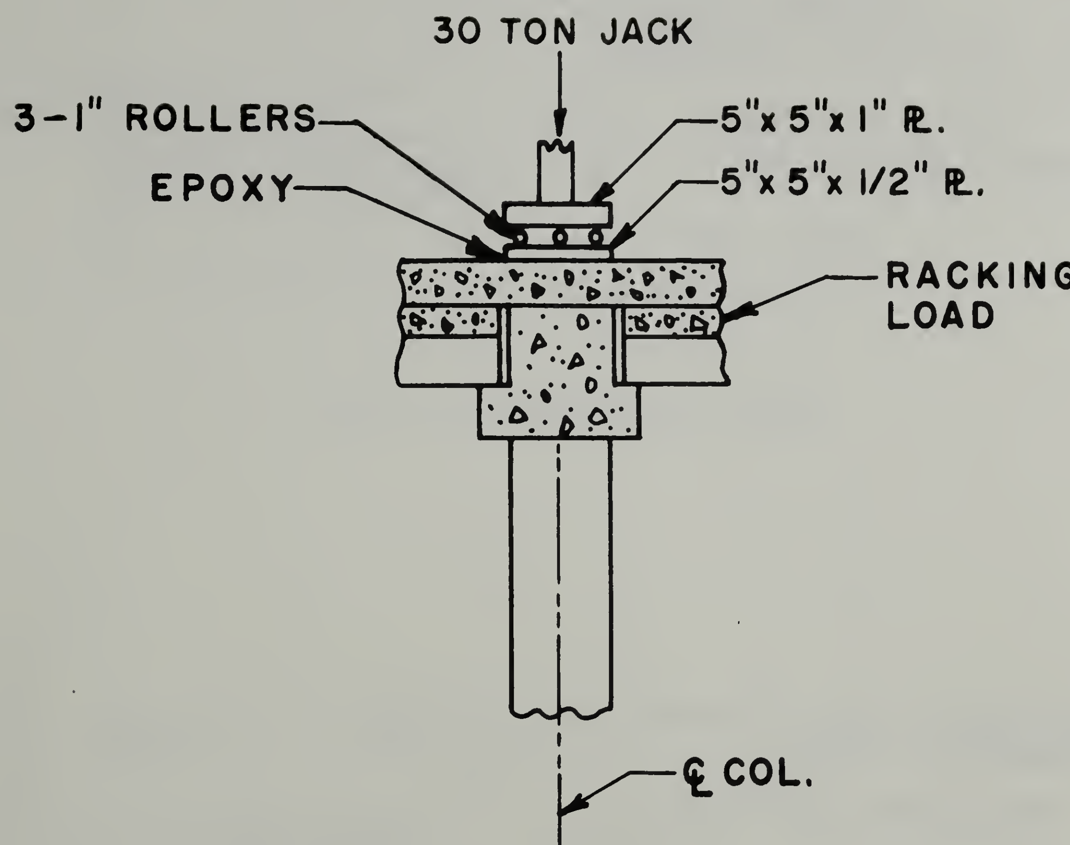
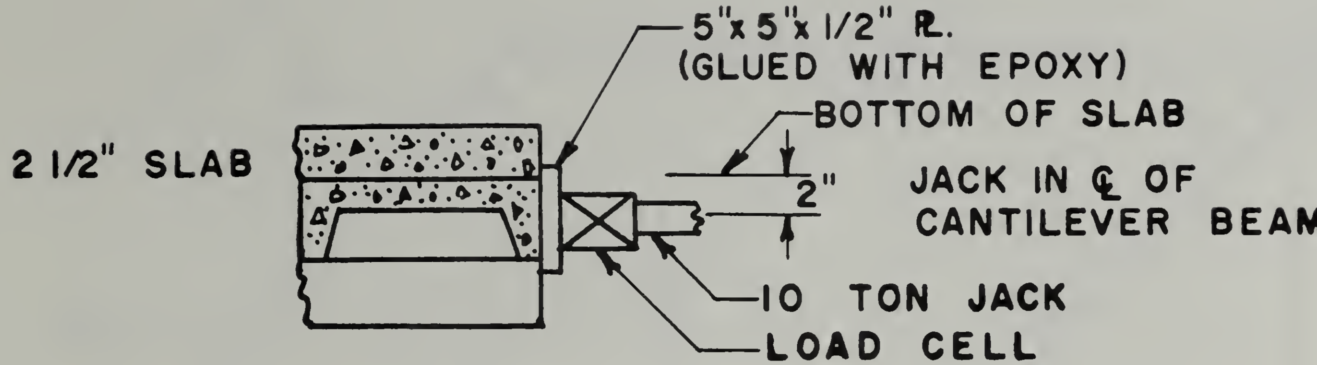
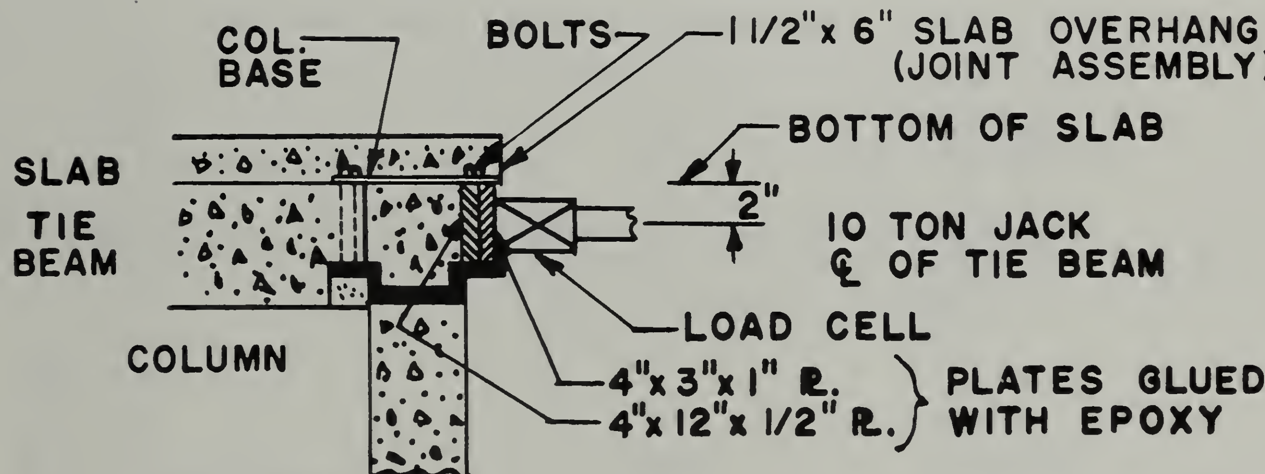


FIGURE 5.28 COLUMN LOAD EQUIPMENT CONNECTION

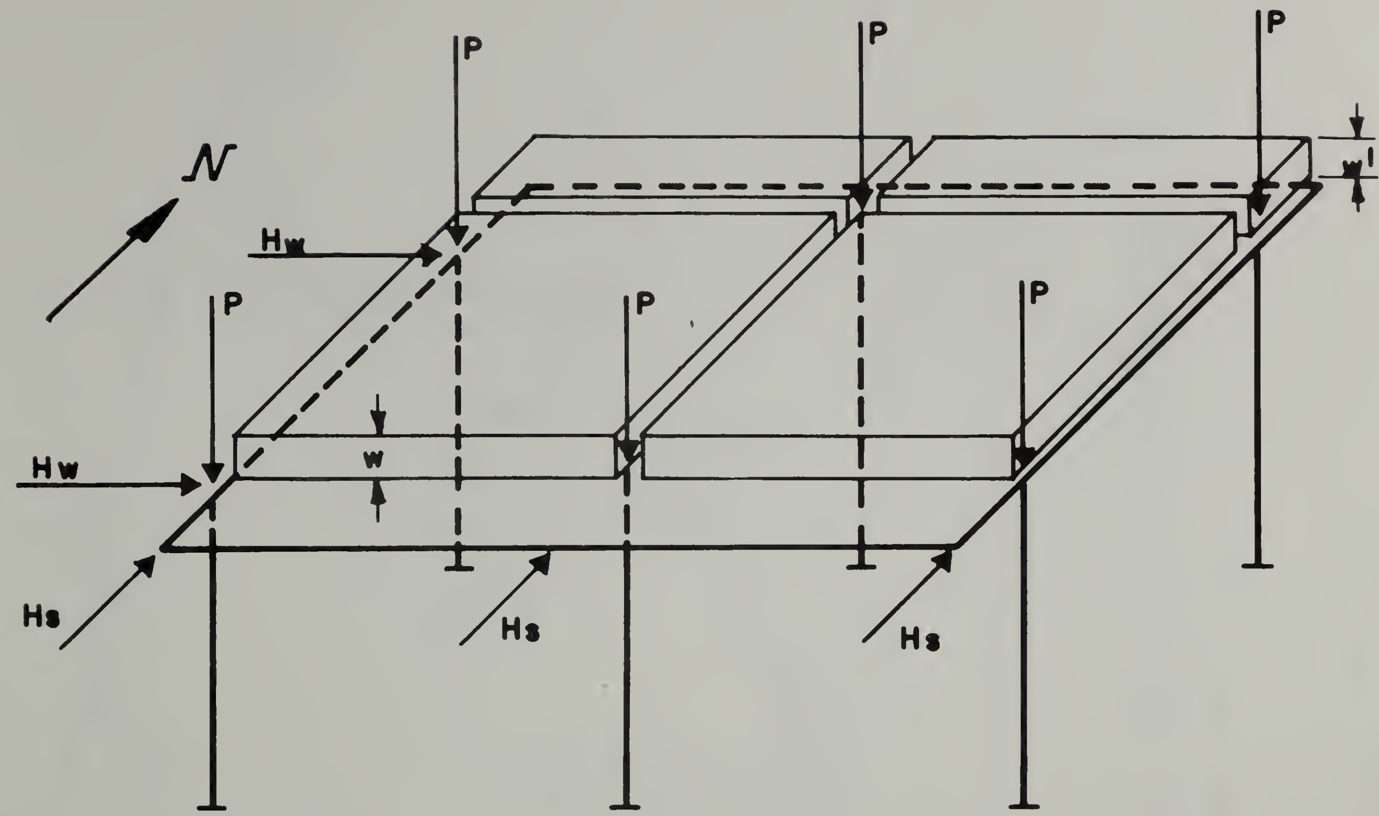


N→S RACKING LOAD



W—E RACKING LOAD

FIGURE 5.29 RACKING LOAD EQUIPMENT CONNECTION



TOP VIEW AND TYPICAL DIMENSIONS

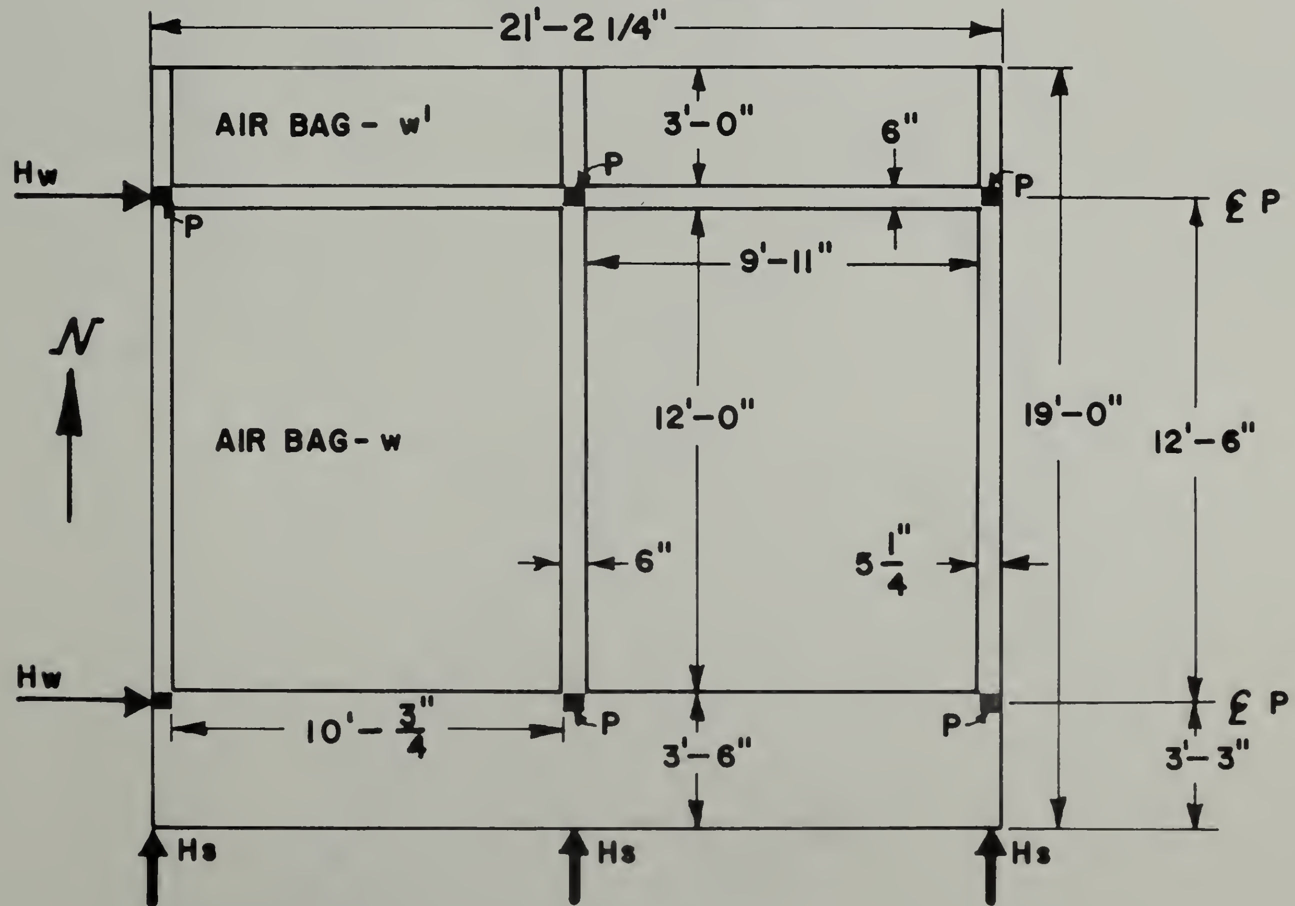


FIGURE 6.1 LOADING OF TEST STRUCTURE

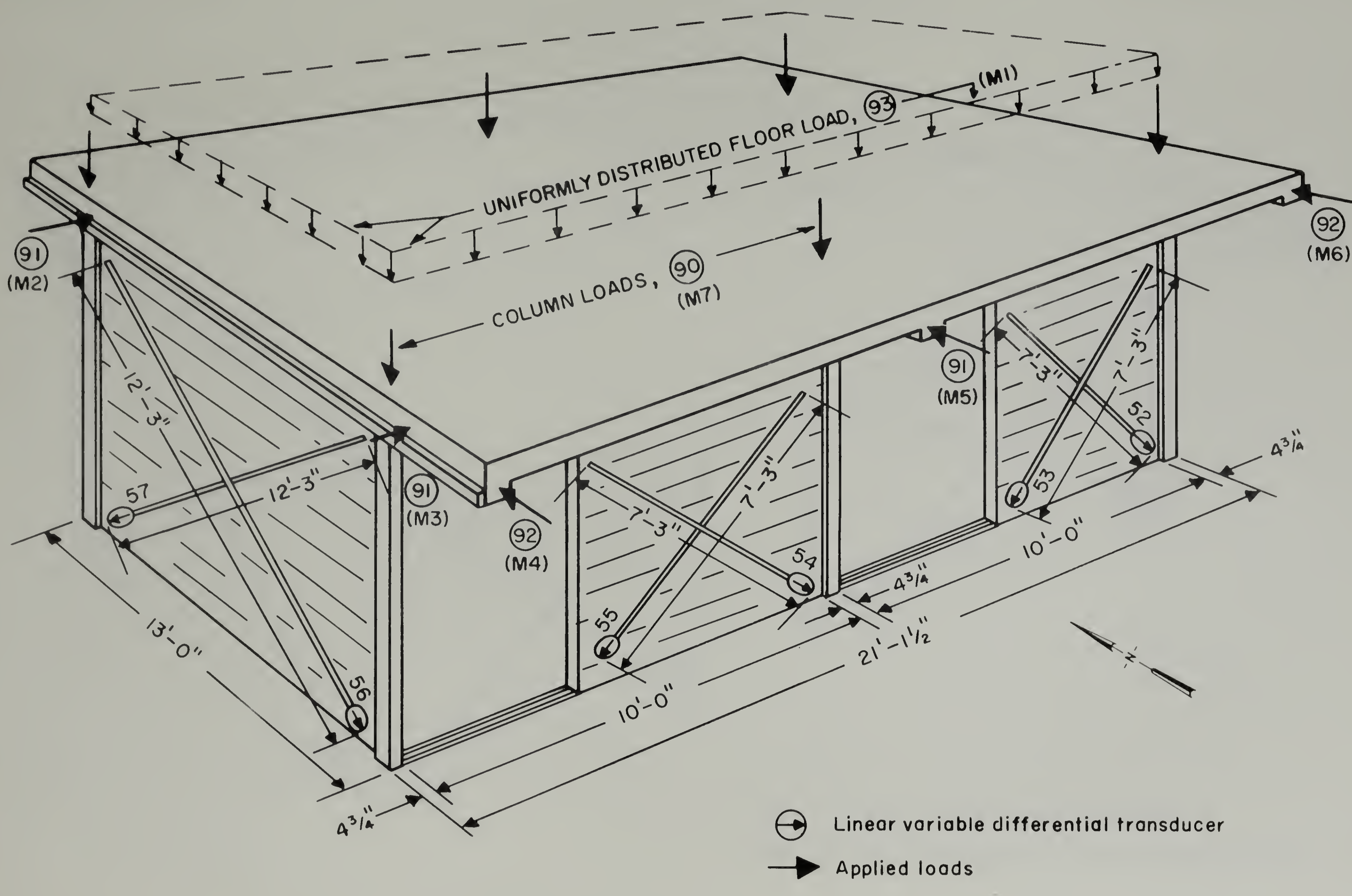


FIGURE 7.1 INSTRUMENTATION LOCATION. NORTHEAST SCHEMATIC.

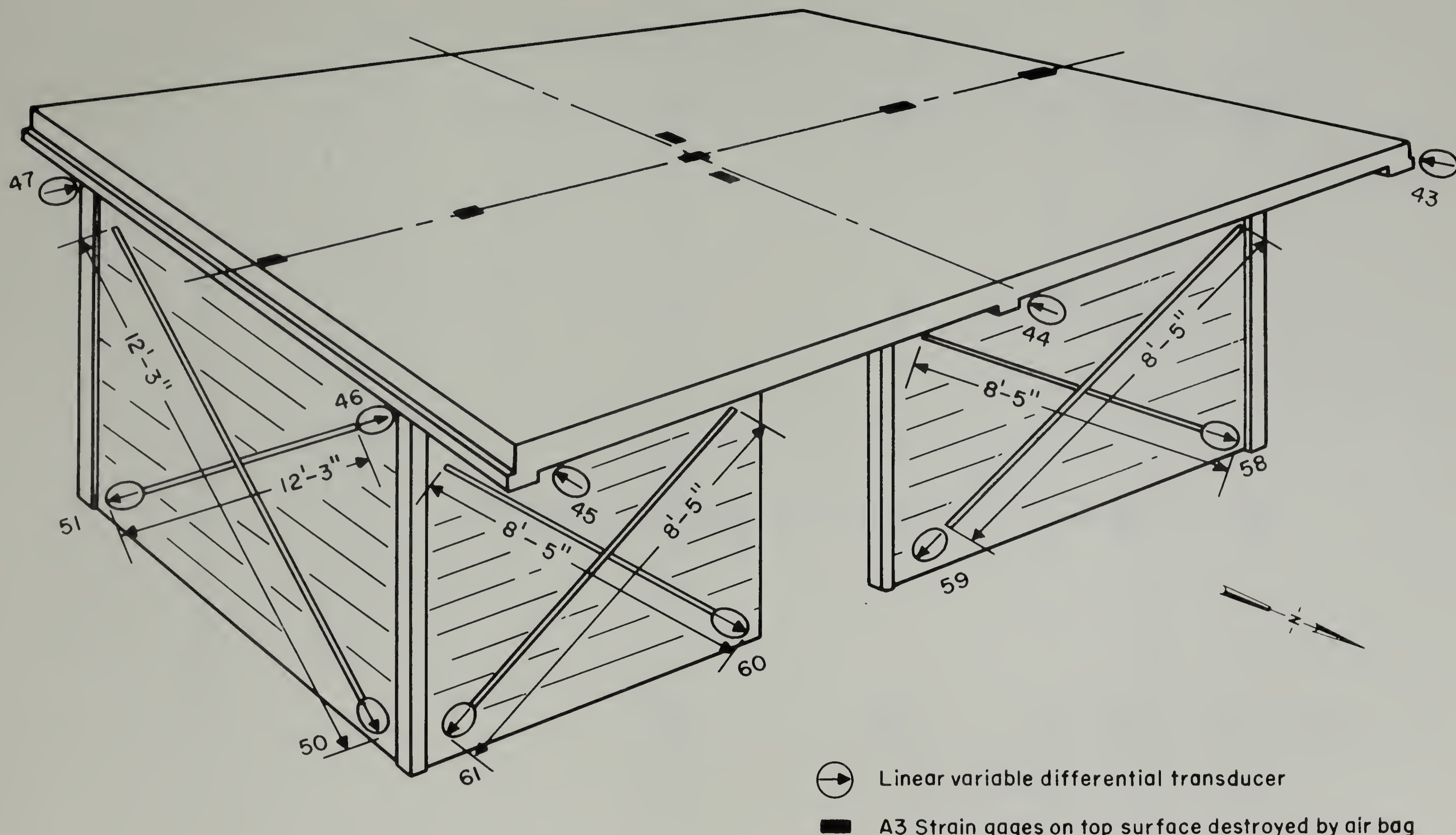
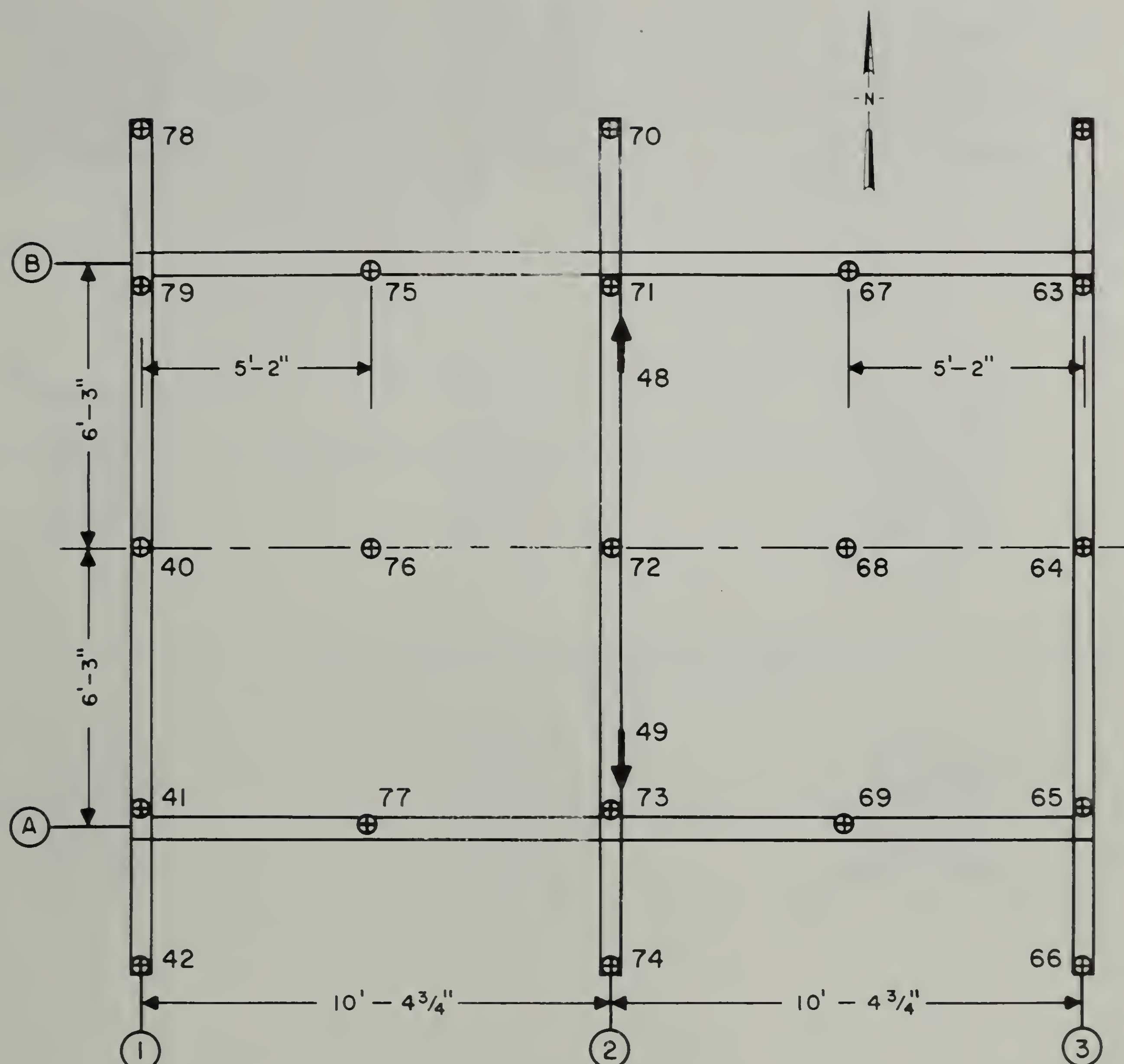


FIGURE 7.2 INSTRUMENTATION LOCATION. SOUTHWEST SCHEMATIC.



⊕ Linear variable differential transducer – vertical

➡ Linear variable differential transducer – horizontal slip

Numbers define channel location digital recorder tape

FIGURE 7.3 VERTICAL DEFLECTION GAGES

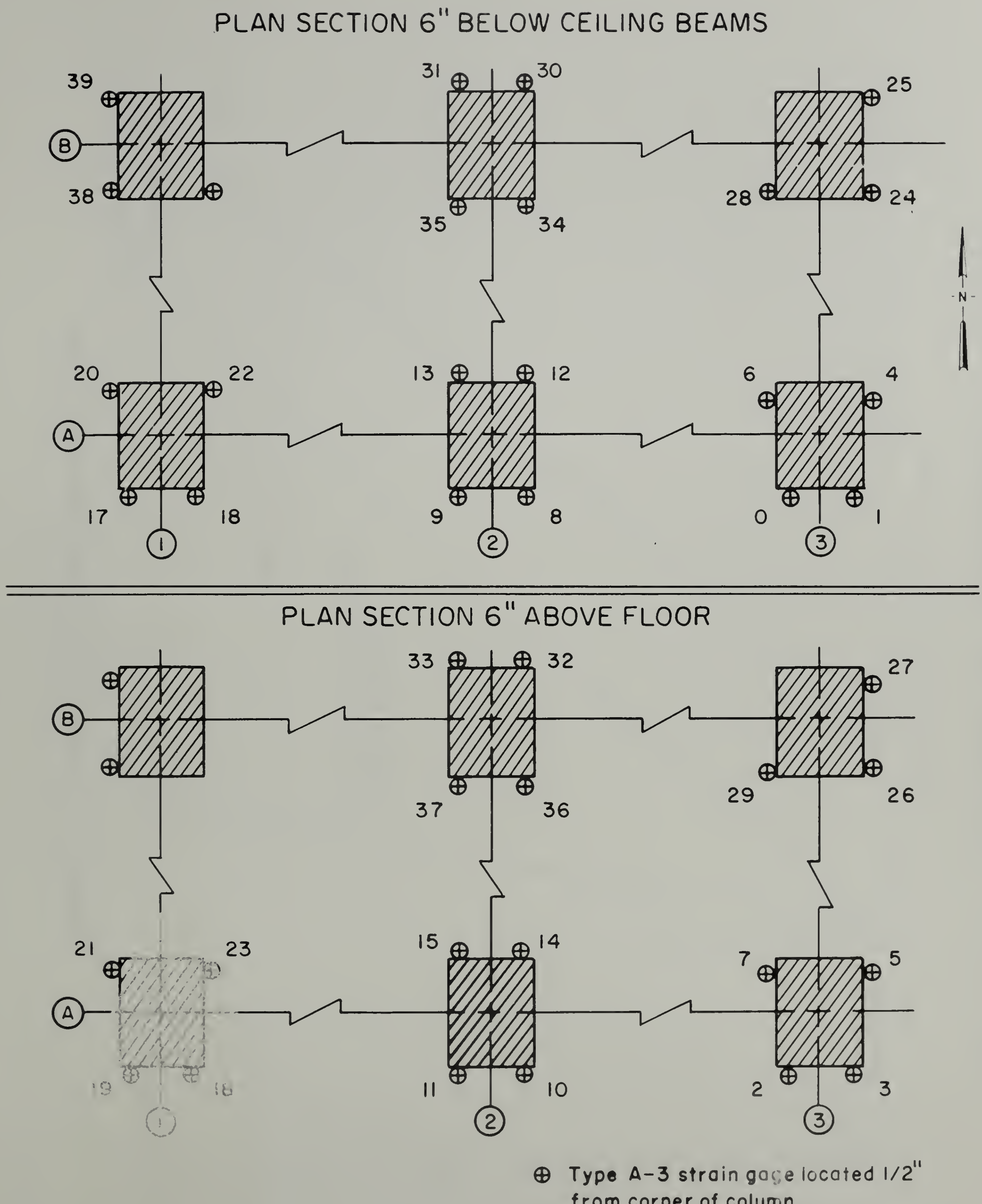


FIGURE 7.4 COLUMN STRAIN GAGE LOCATION

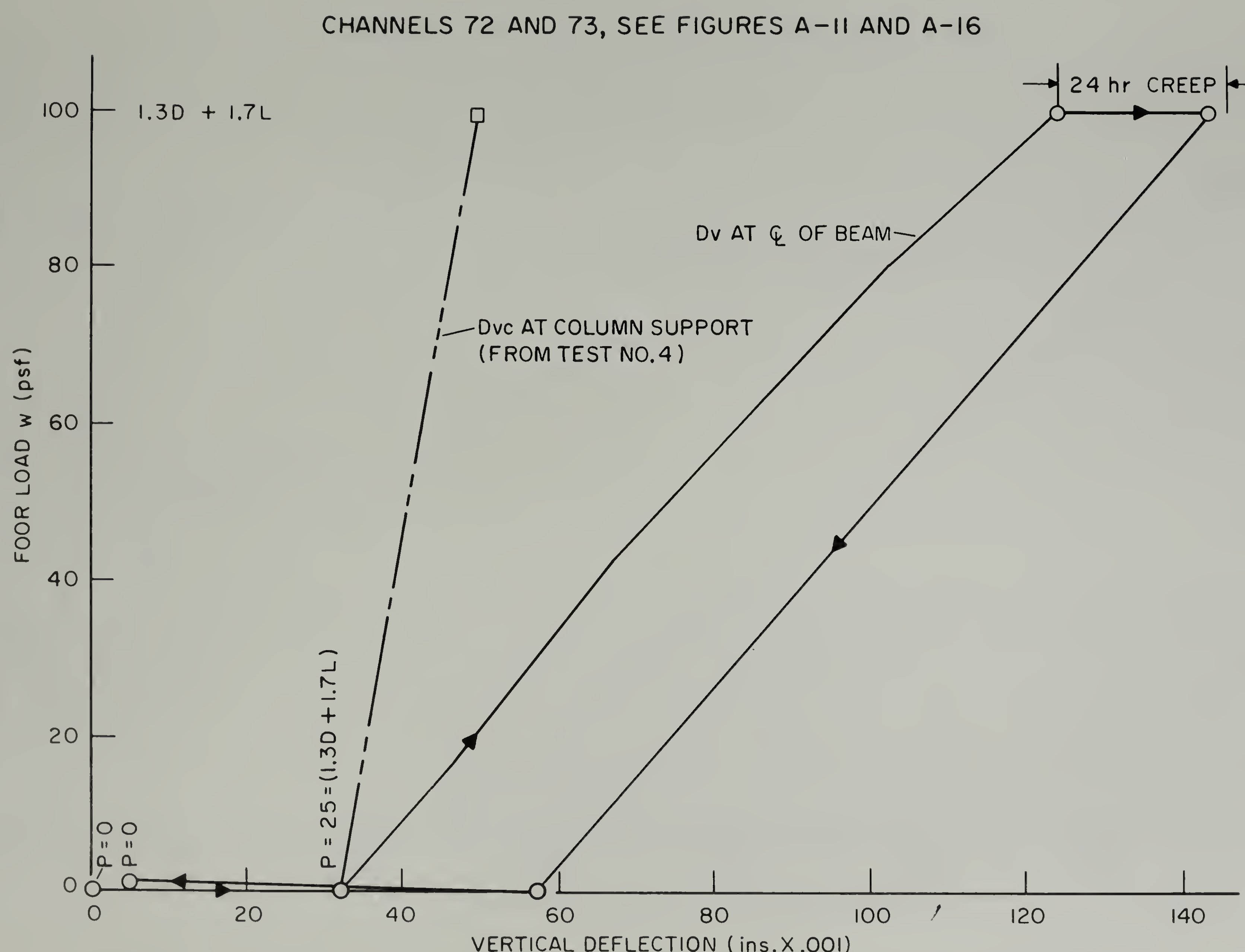


FIGURE 9.1 TEST NO. 5, SUSTAINED VERTICAL LOAD VS. MIDSPAN DEFLECTION OF CENTER MAIN BEAM.

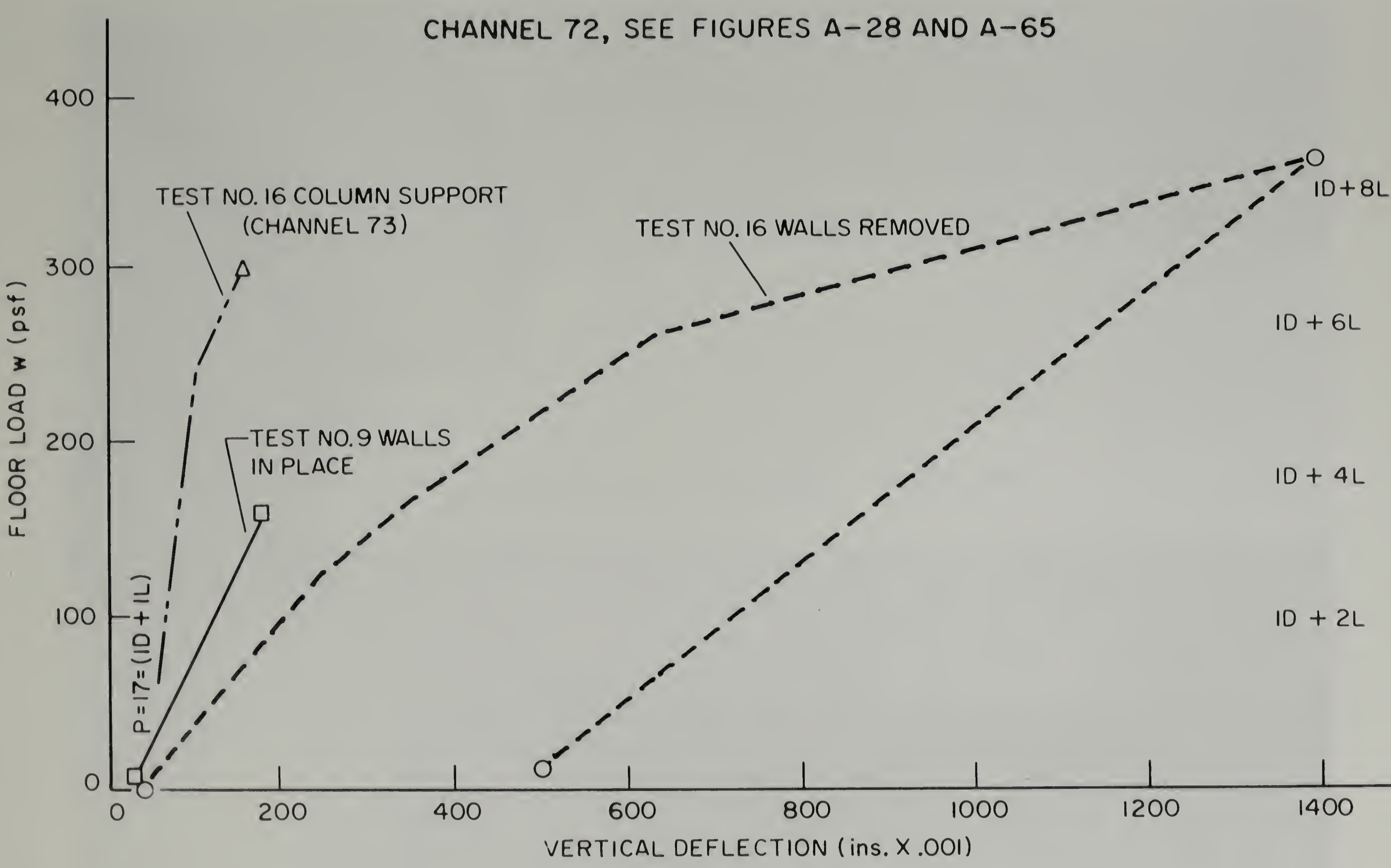


FIGURE 9.2 TEST NO. 16, MAJOR FLOOR LOAD (w) VS. MIDSPAN DEFLECTION OF CENTER MAIN BEAM.

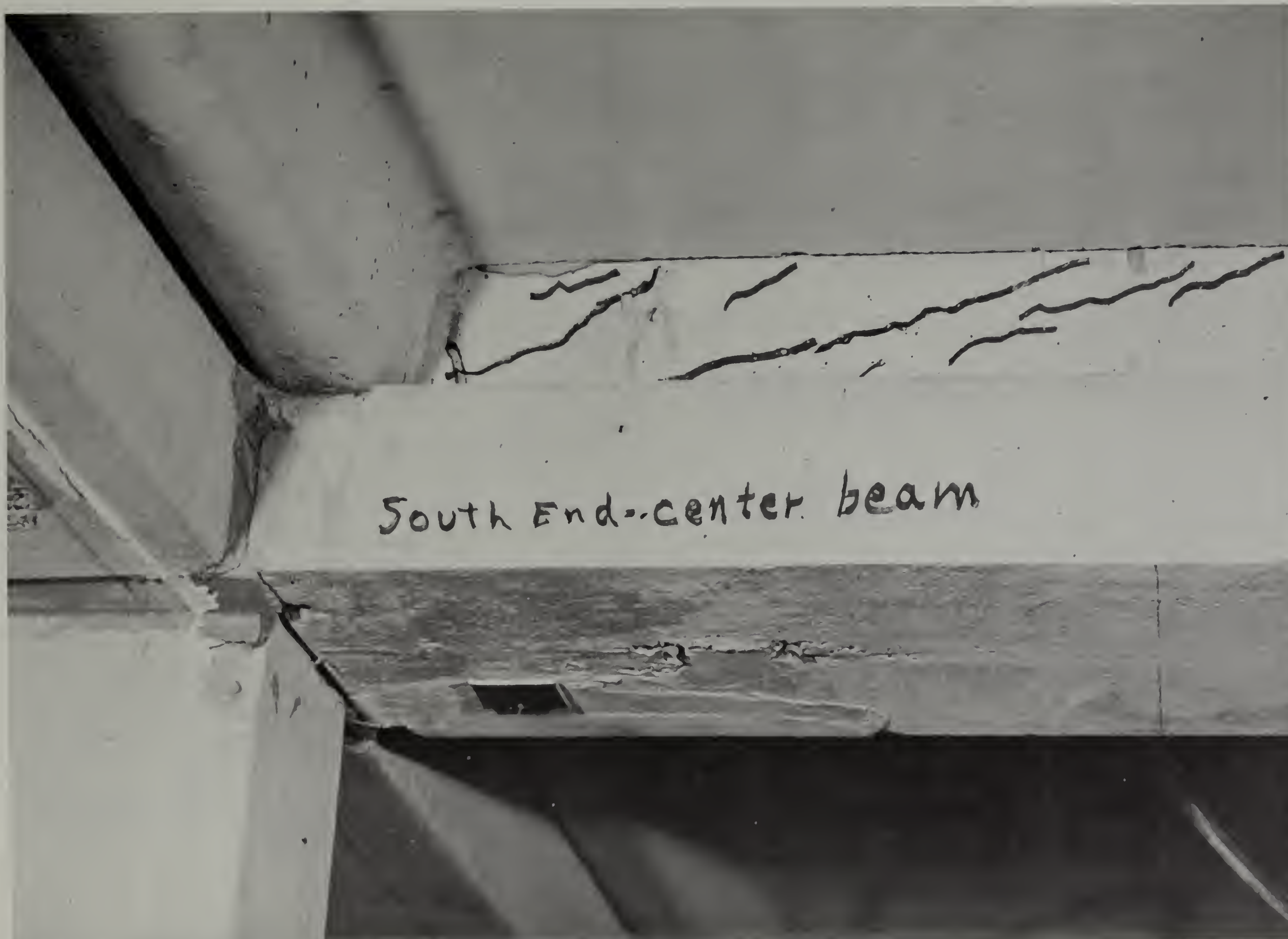


FIGURE 9.3 DIAGONAL TENSION CRACKING AT $w = 270$ PSF (TEST NO. 16)

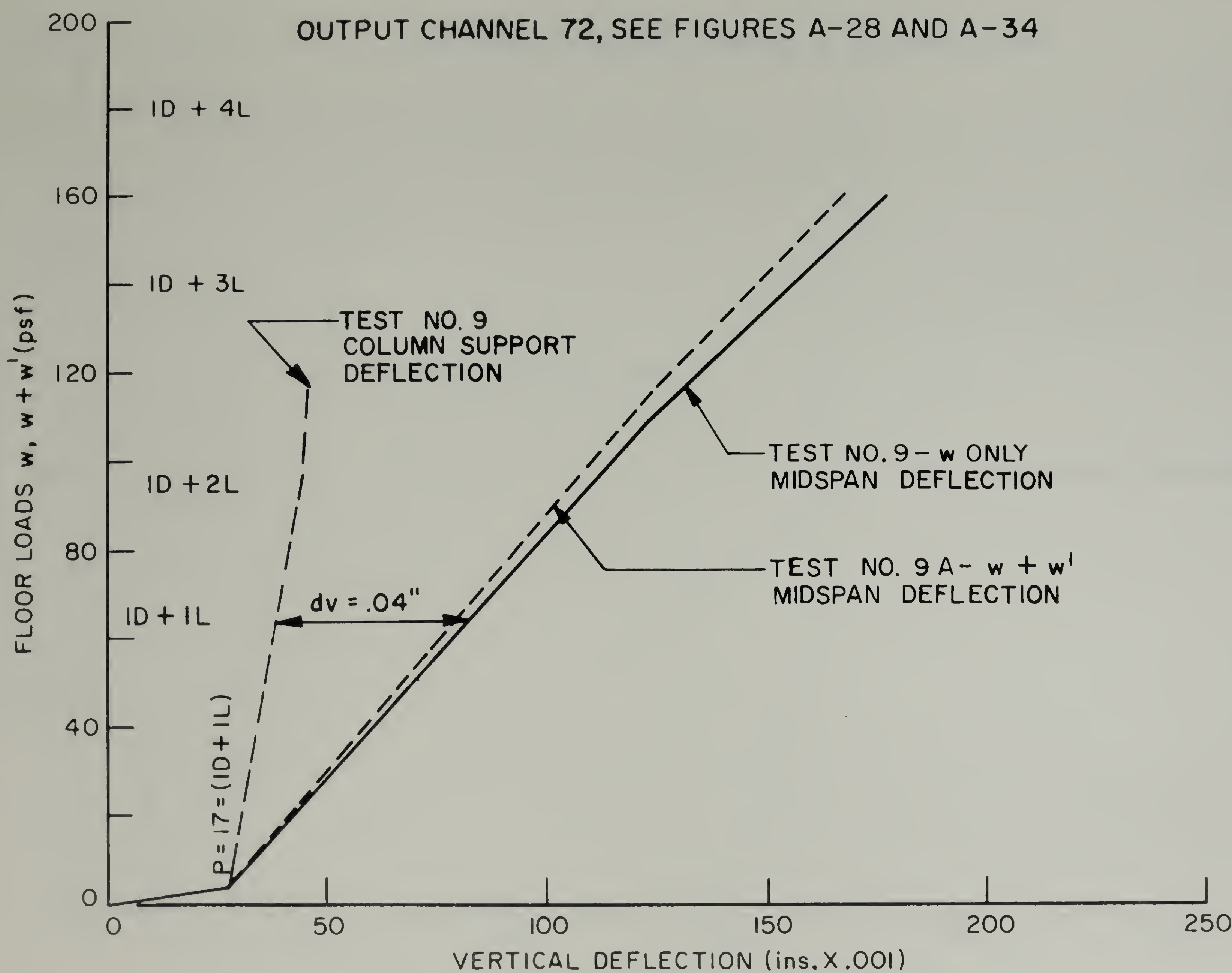


FIGURE 9.4 INTERIOR FLOOR LOAD (w) AND TOTAL FLOOR LOAD (w + w' VS. MIDSPAN AND COLUMN SUPPORT DEFLECTION OF CENTER MAIN BEAM.

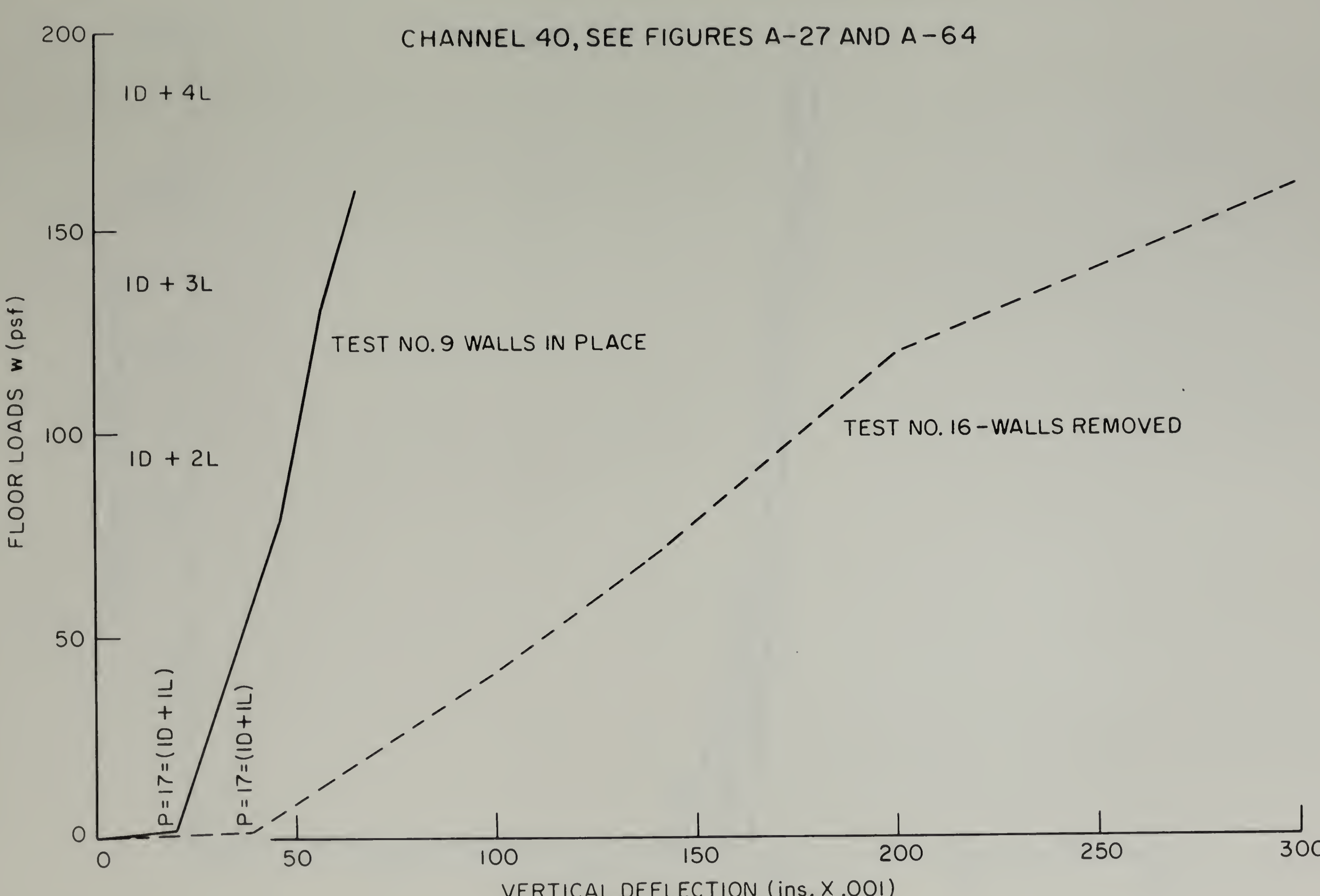


FIGURE 9.5 MIDSPAN DEFLECTION OF WEST MAIN BEAM WITH AND WITHOUT WALLS.

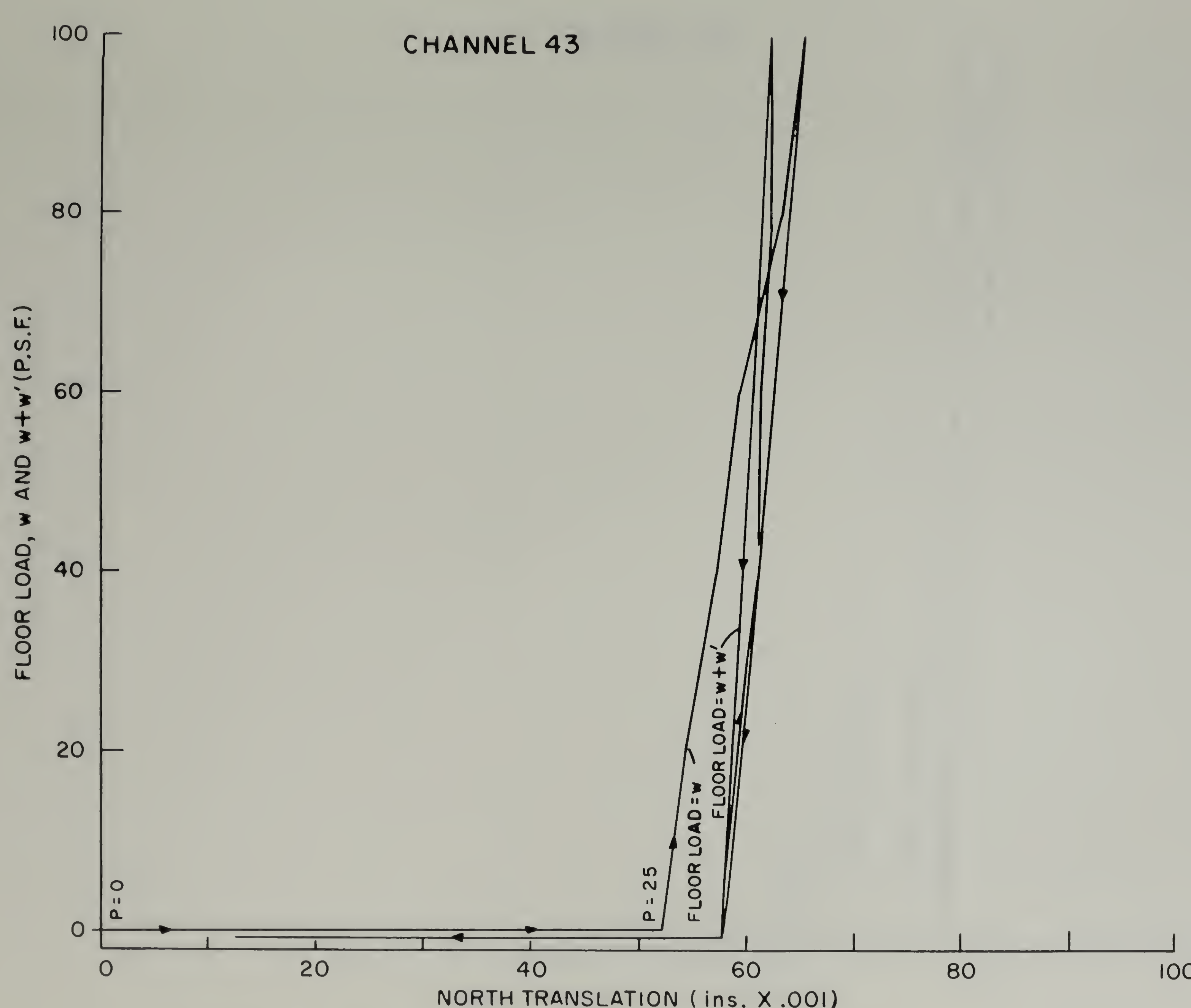


FIGURE 9.6 TEST NO. 12A AND 13A, VERTICAL LOAD VS. NORTH TRANSLATION.

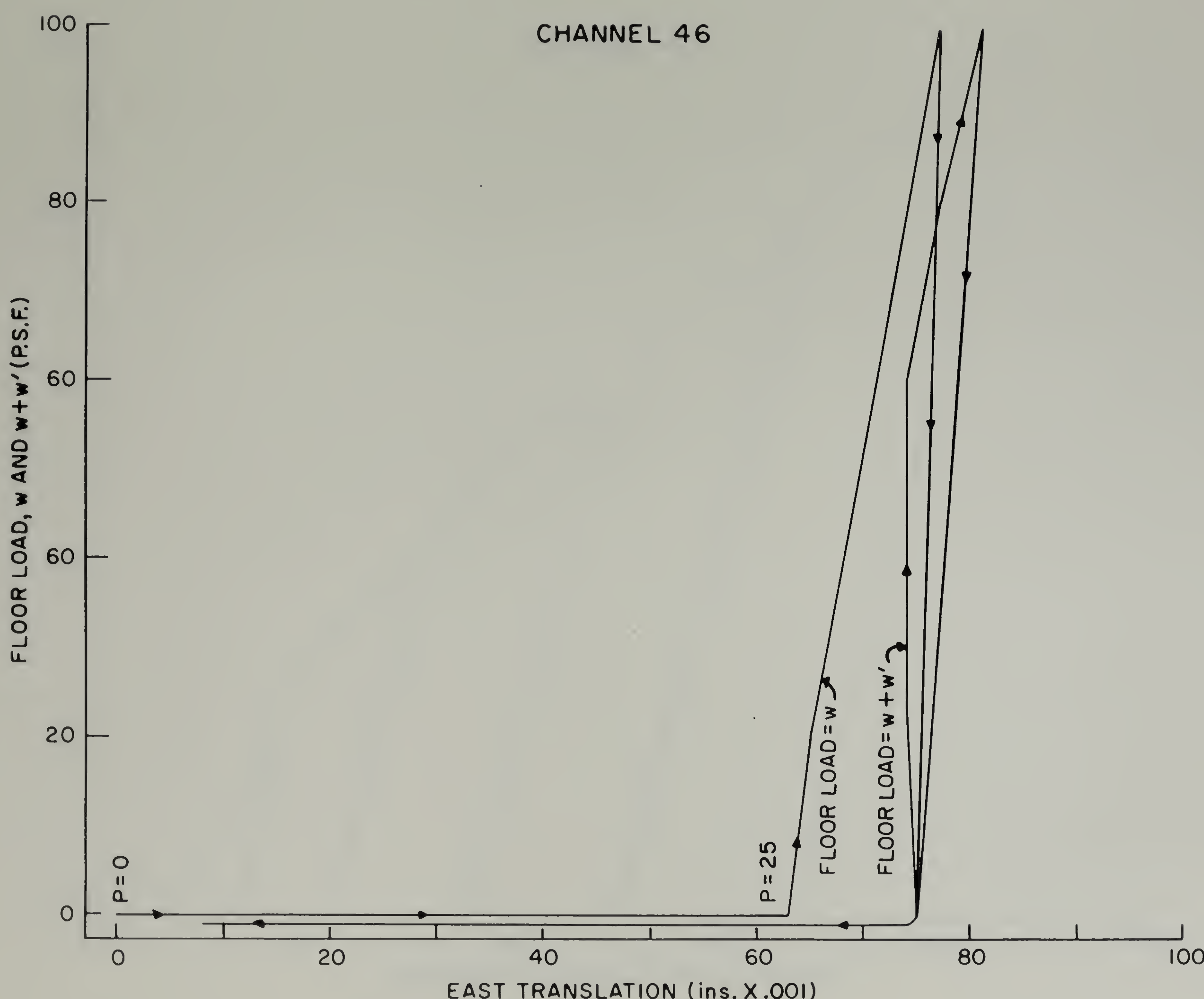


FIGURE 9.7 TEST NO. 12 & 13, VERTICAL LOAD VS. EAST TRANSLATION.

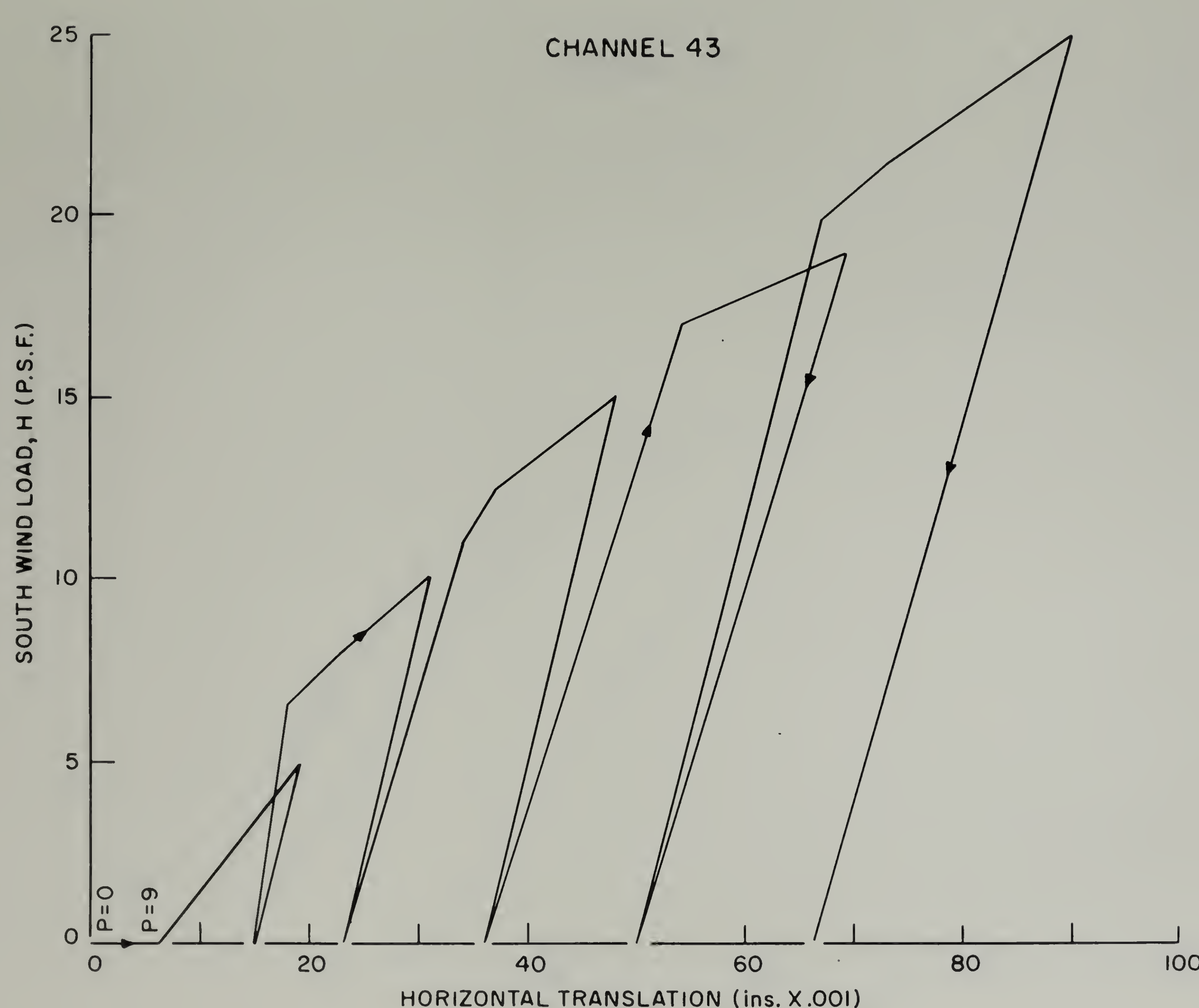


FIGURE 9.8 TEST NO. 2, SOUTH WIND LOAD VS. TRANSLATION.

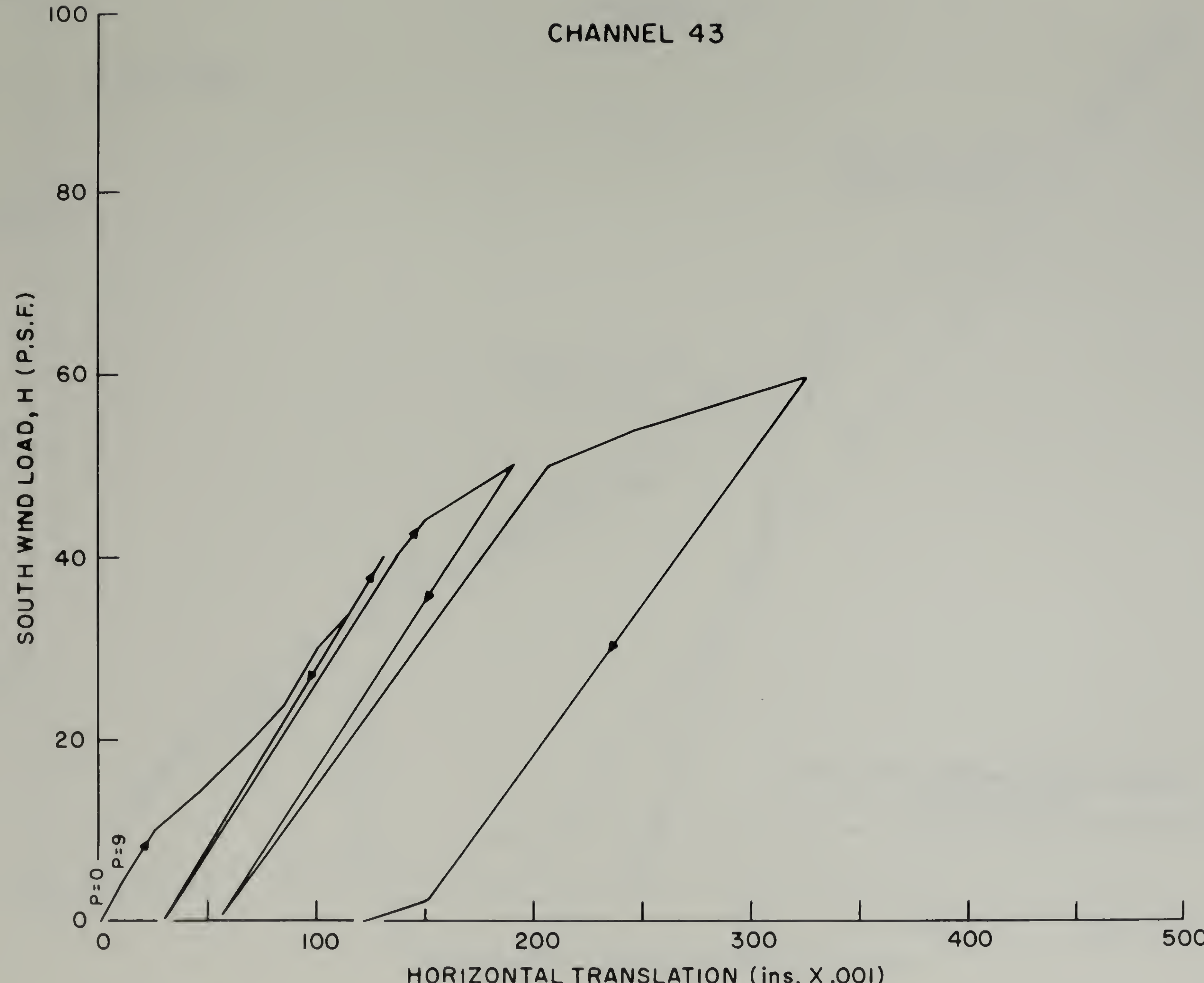


FIGURE 9.9 TEST NO. 10, SOUTH WIND LOAD VS. TRANSLATION.

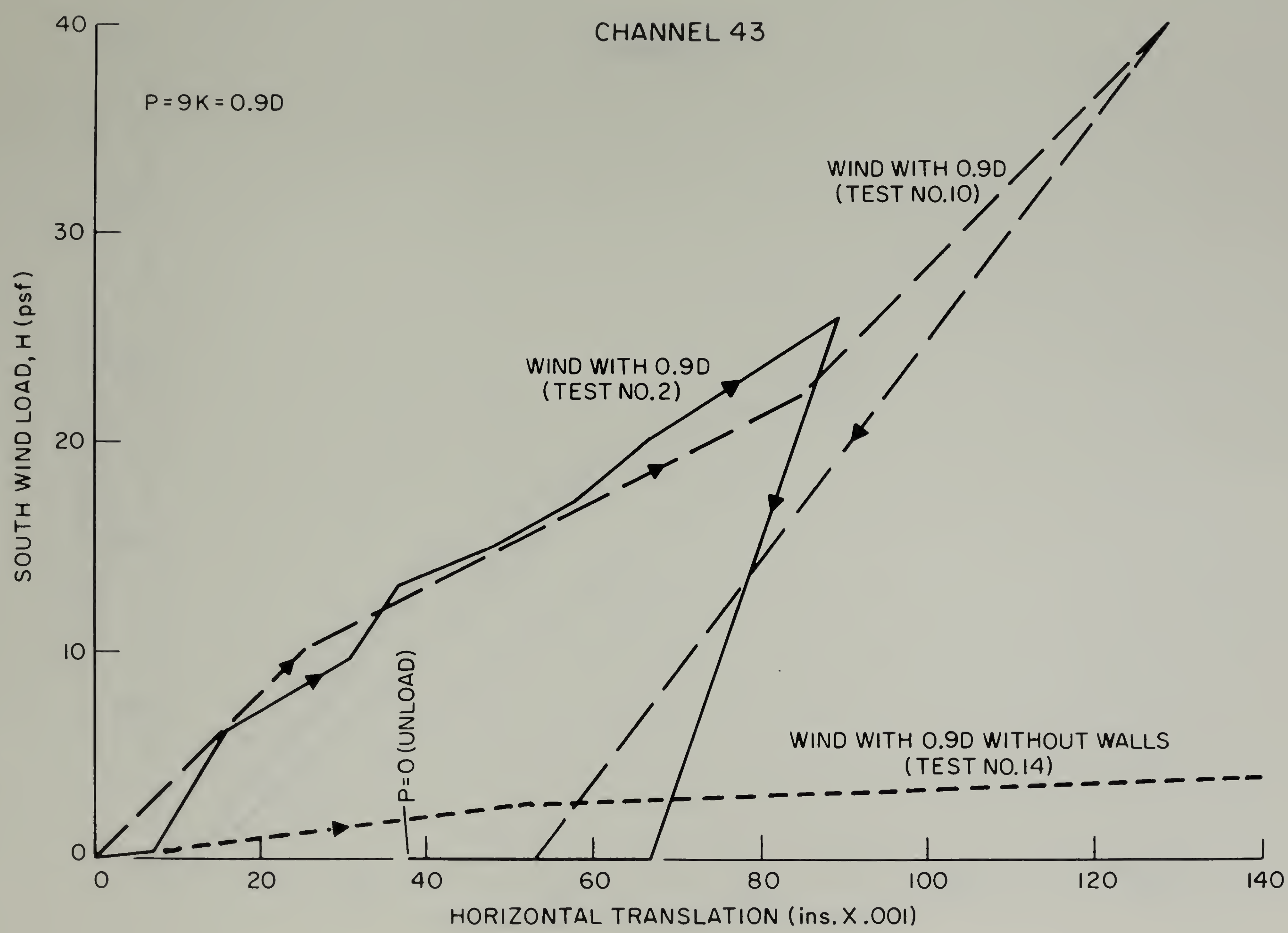


FIGURE 9.10 N–S HORIZONTAL TRANSLATION OF STRUCTURE WITH AND WITHOUT WALLS

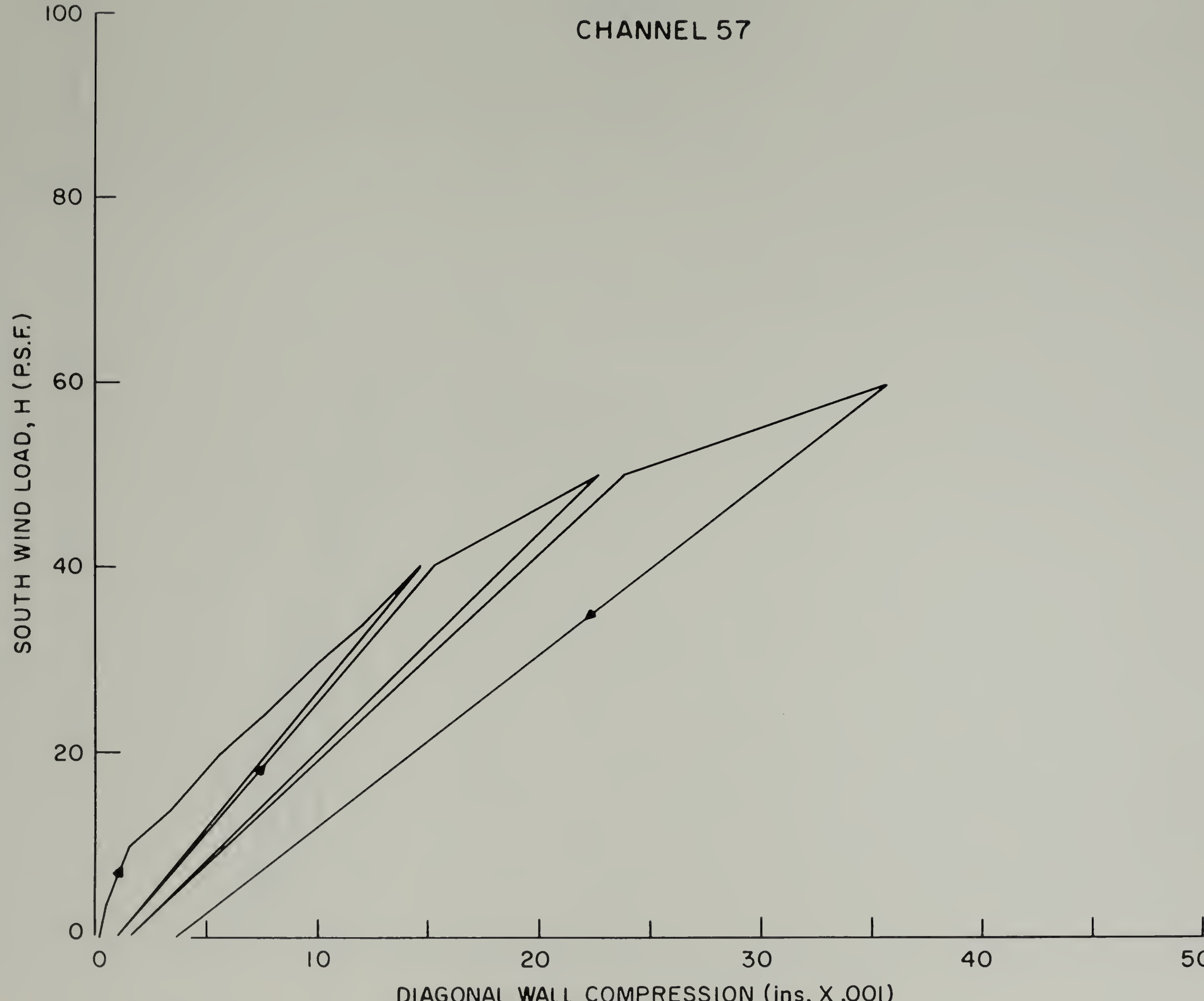


FIGURE 9.11 TEST NO. 10, SOUTH WIND LOAD VS. WALL COMPRESSION.

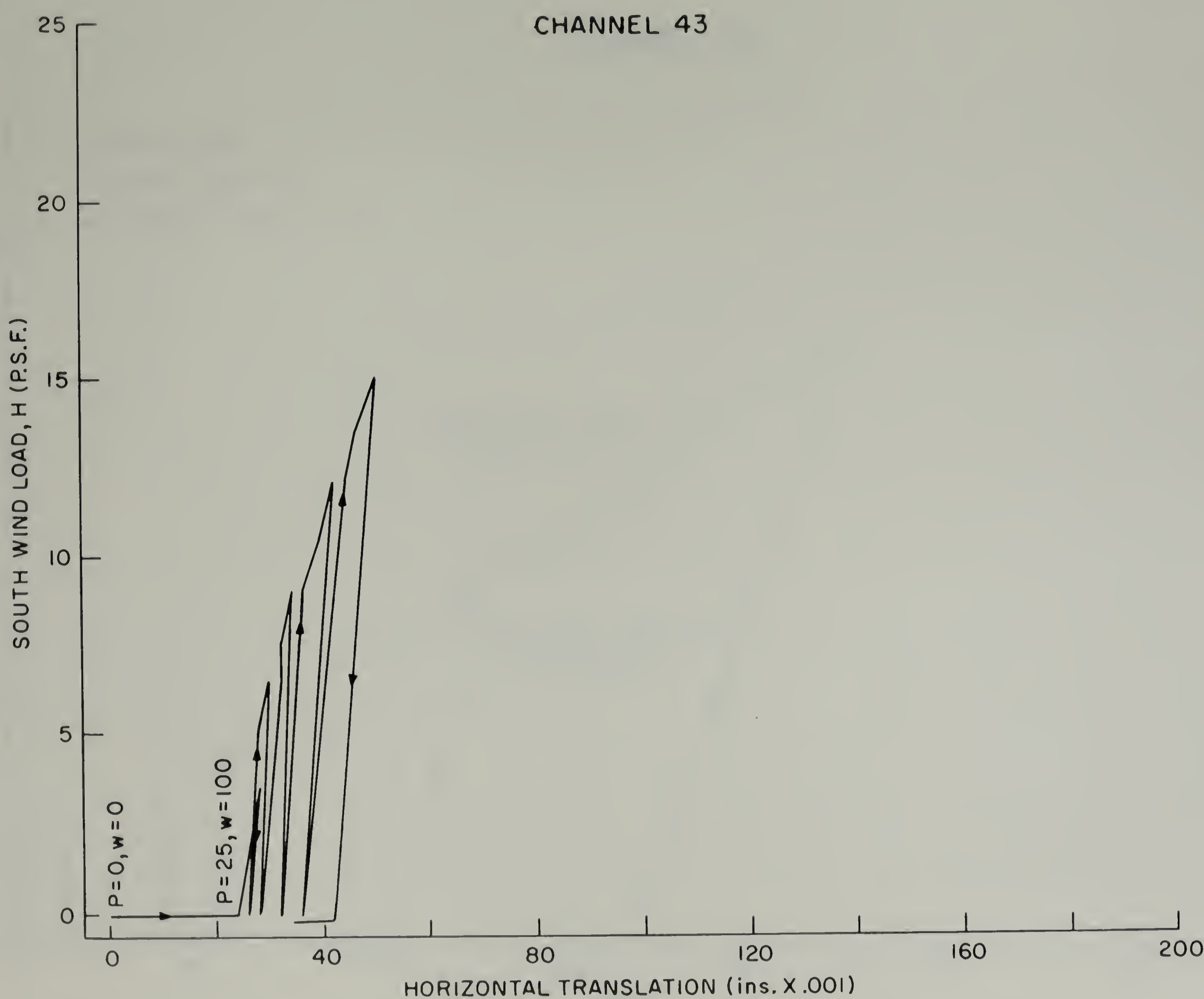


FIGURE 9.12 TEST NO. 7, SOUTH WIND LOAD VS. TRANSLATION.

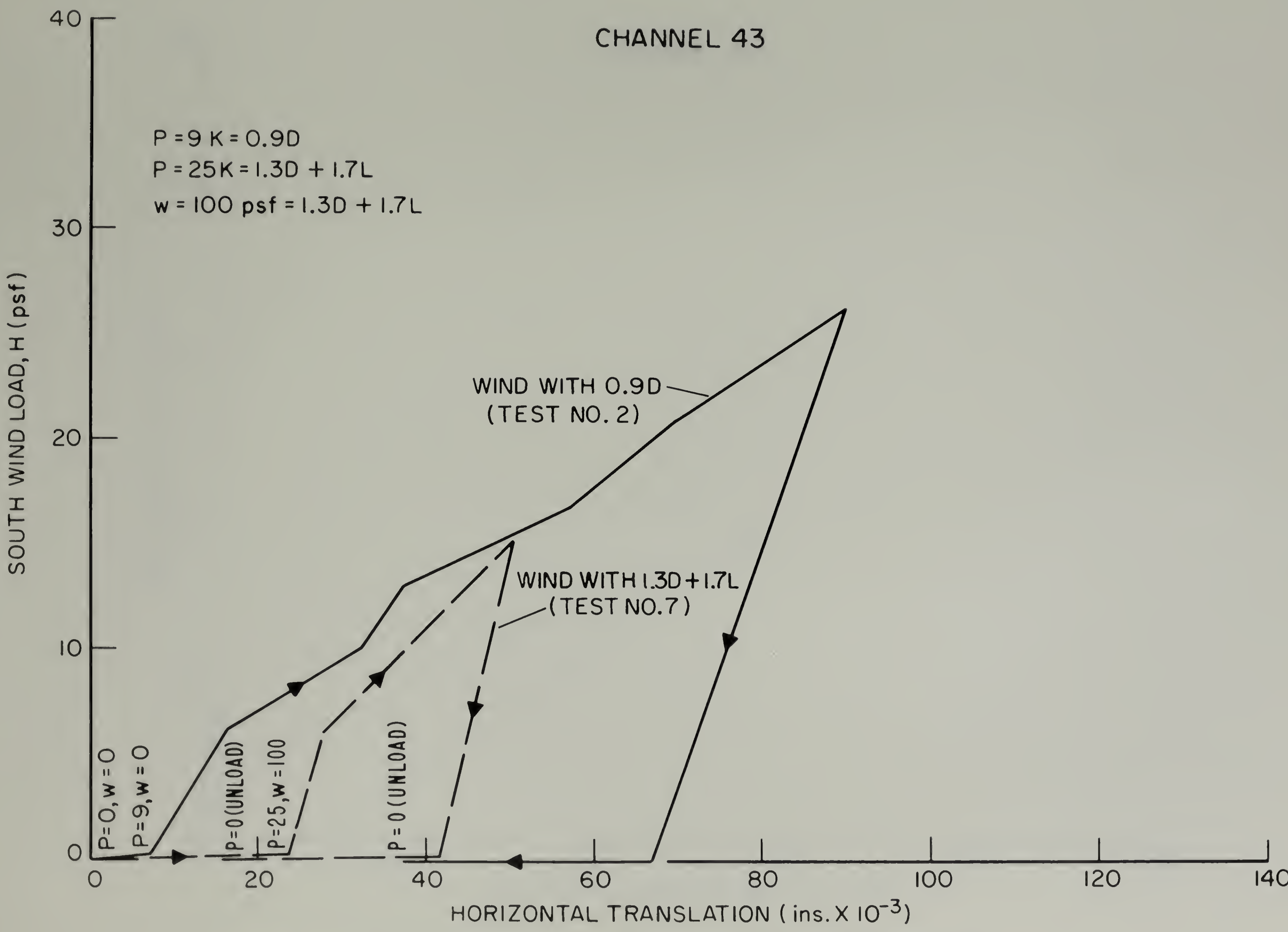


FIGURE 9.13 EFFECT OF VERTICAL LOADS ON N–S HORIZONTAL TRANSLATION.

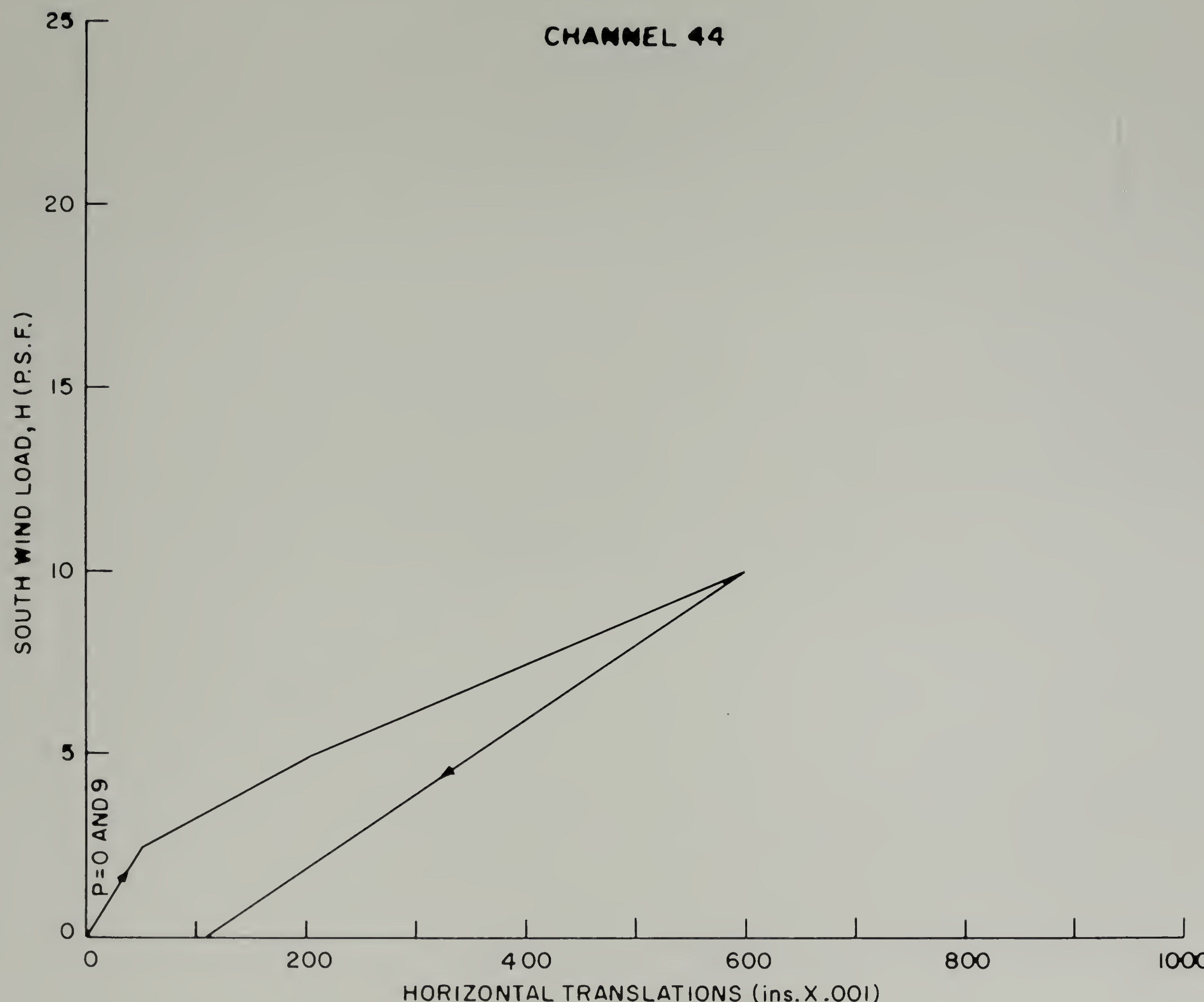


FIGURE 9.14 TEST NO. 14, SOUTH WIND LOAD VS. TRANSLATION.

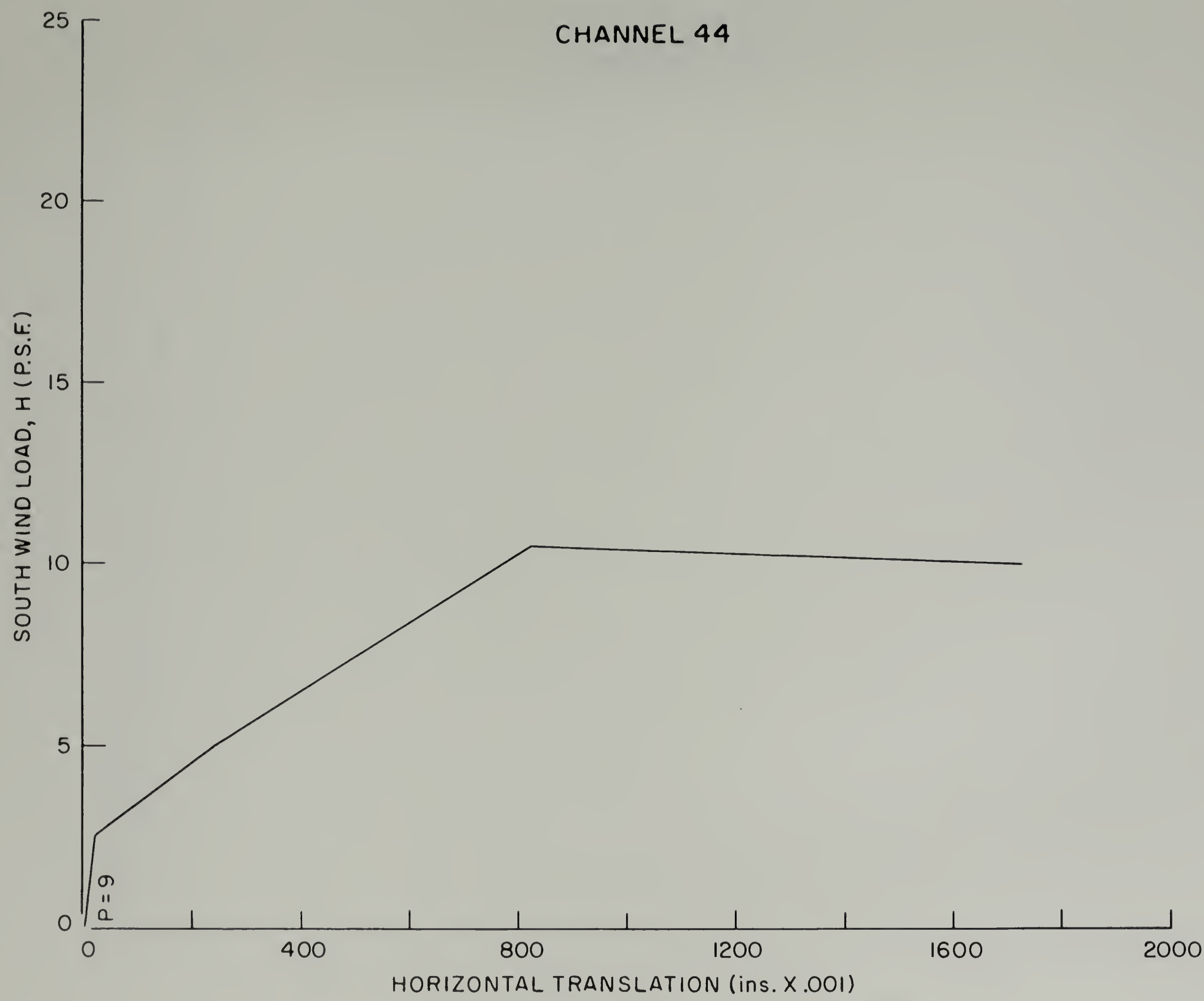


FIGURE 9.15 TEST NO. 18, SOUTH WIND LOAD VS. TRANSLATION.

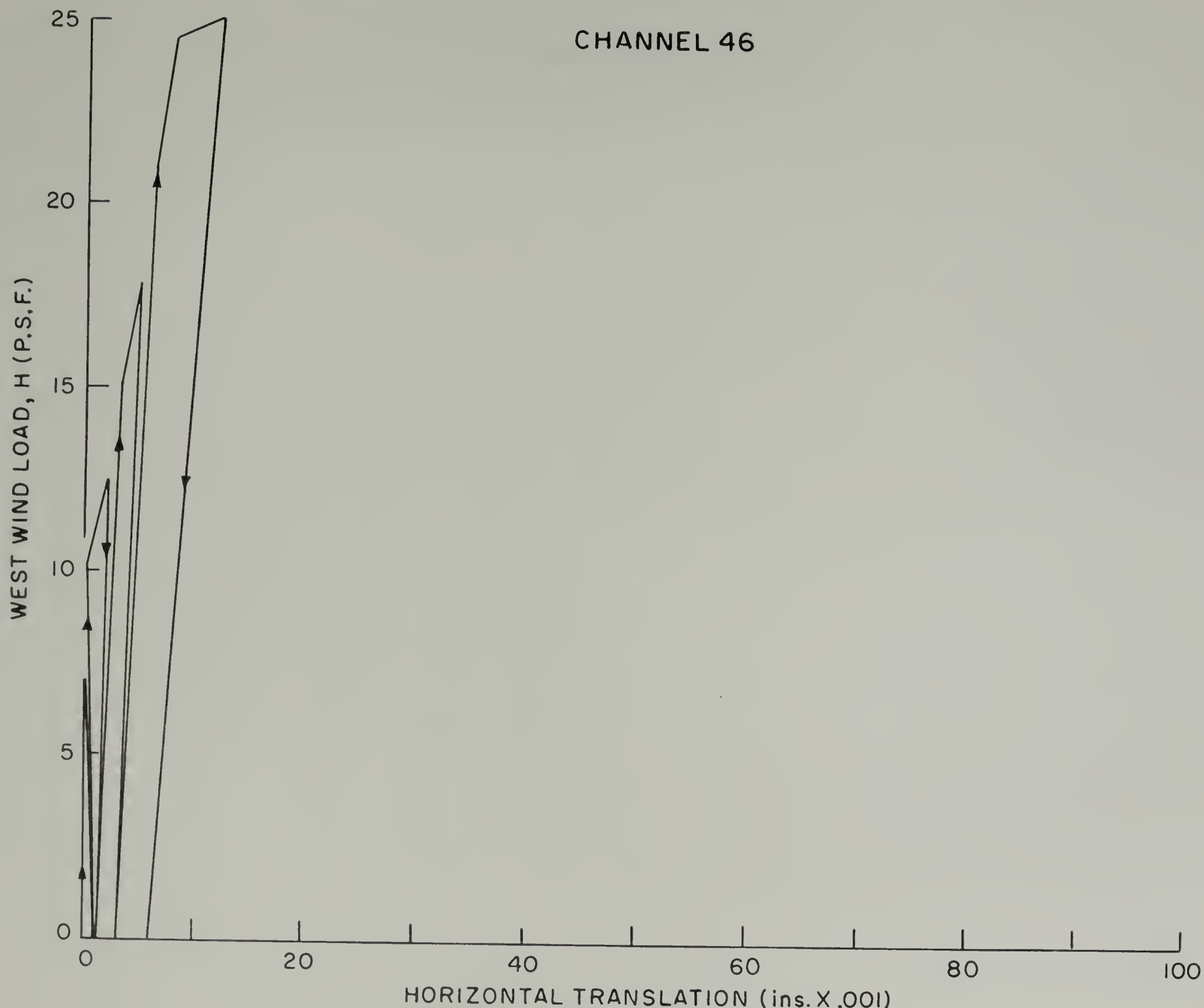


FIGURE 9.16 TEST NO. 3, WEST WIND LOAD VS. TRANSLATION.

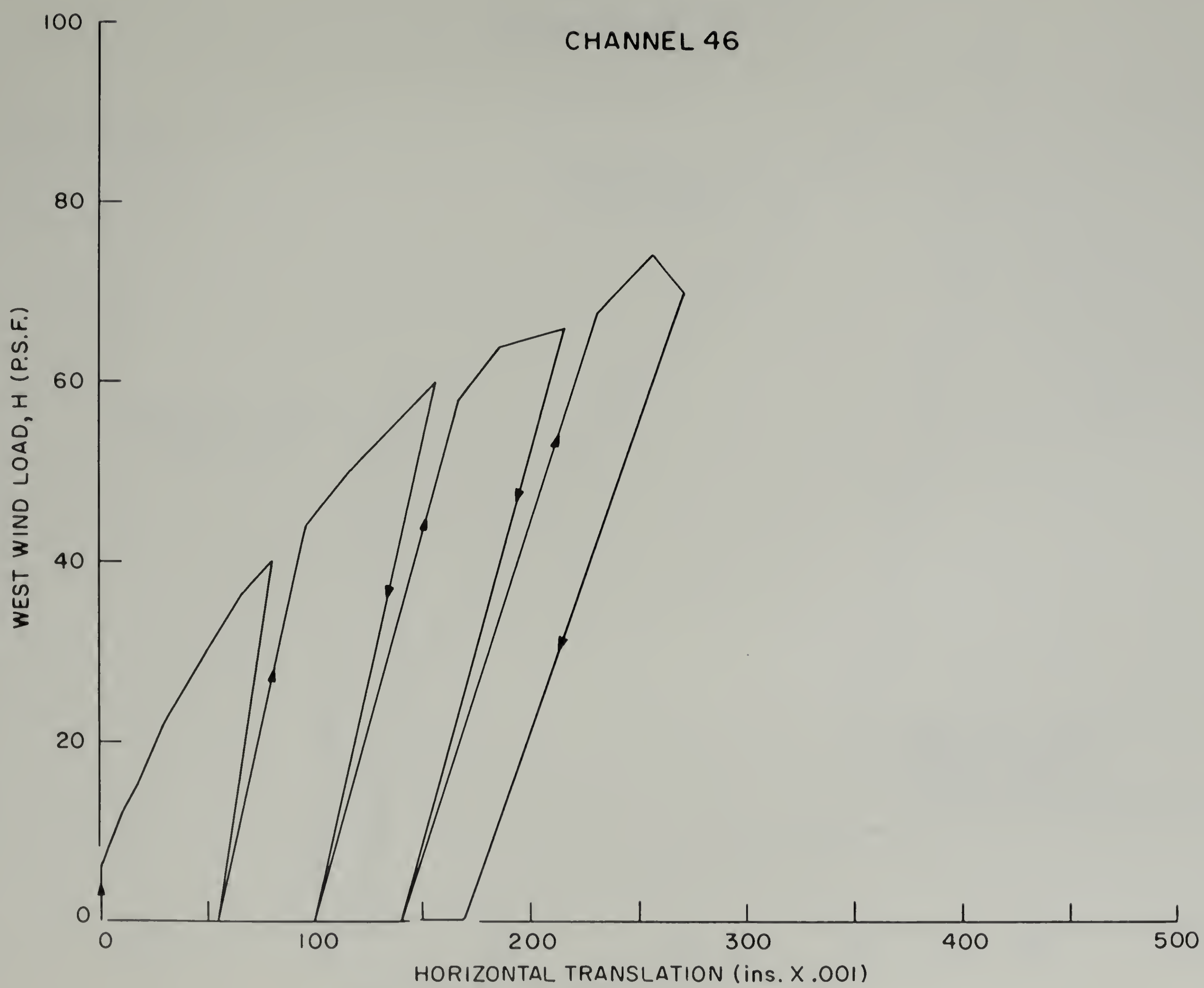


FIGURE 9.17 TEST NO. 11, WEST WIND LOAD VS. TRANSLATION.

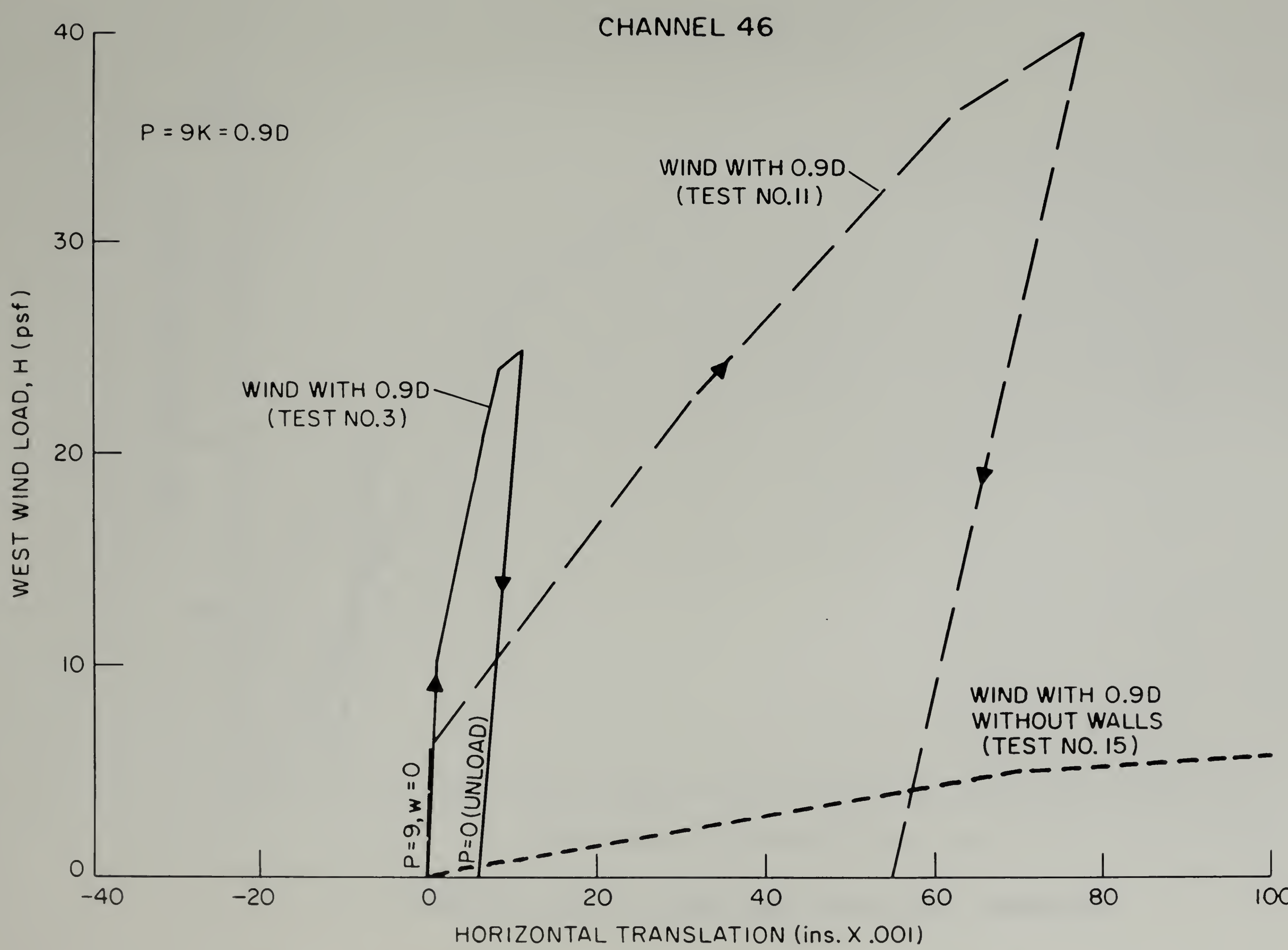


FIGURE 9.18 E–W HORIZONTAL TRANSLATION OF STRUCTURE WITH AND WITHOUT WALLS

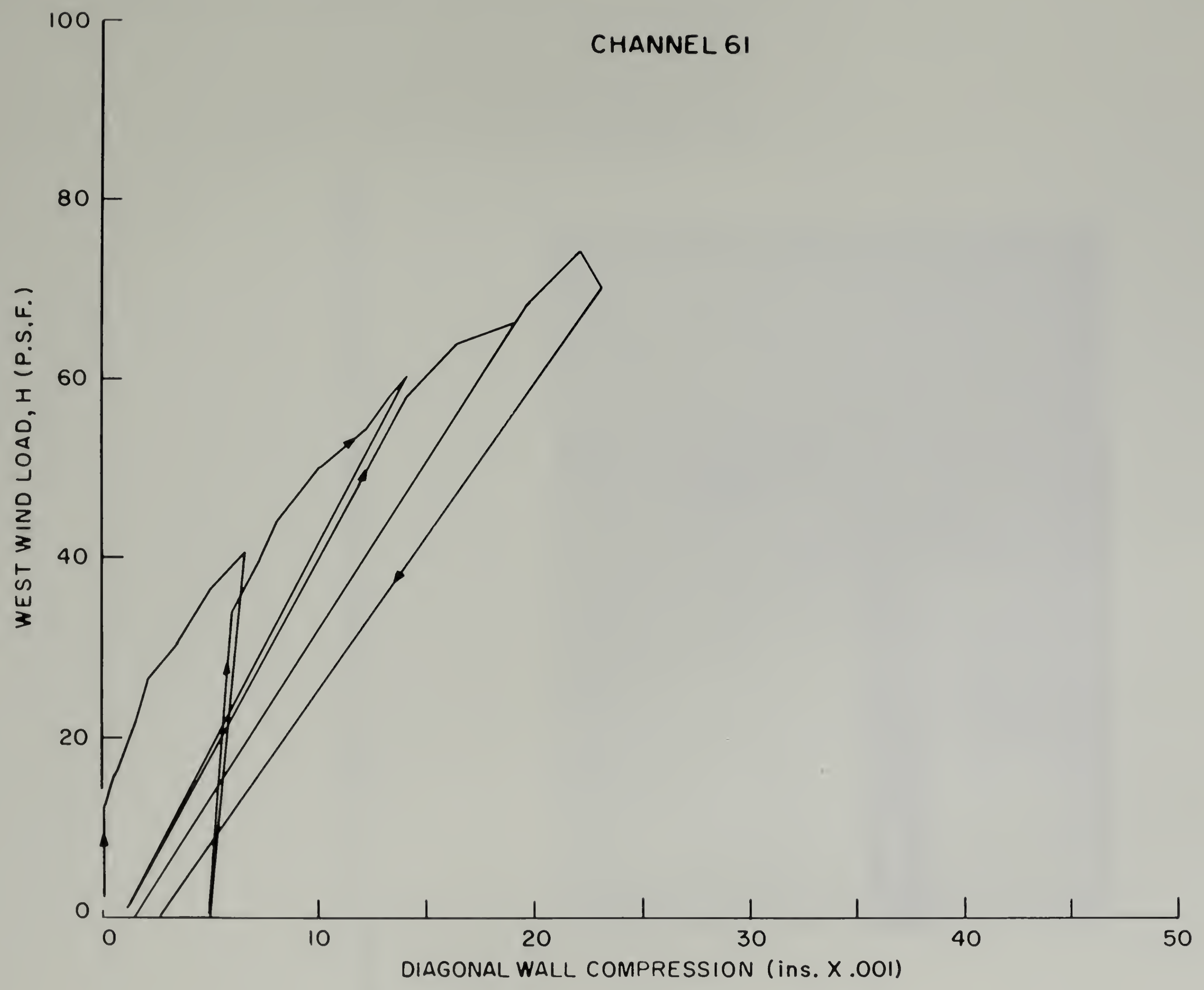


FIGURE 9.19 TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

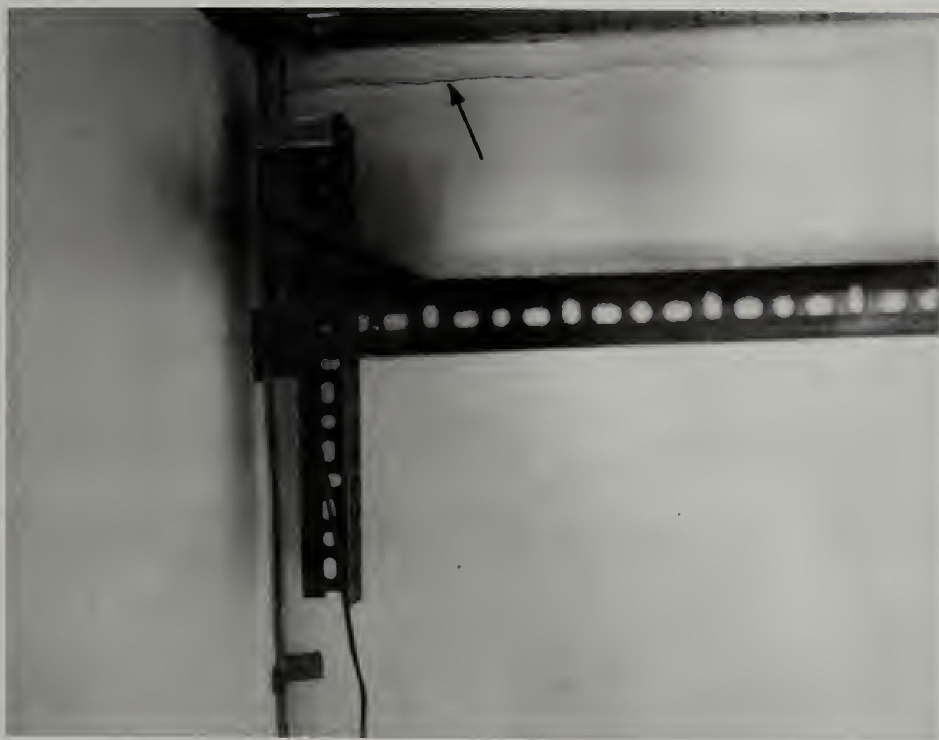


FIGURE 9.20 DRYWALL CRACK NEAR CEILING ON INTERIOR SIDE OF EAST WALL

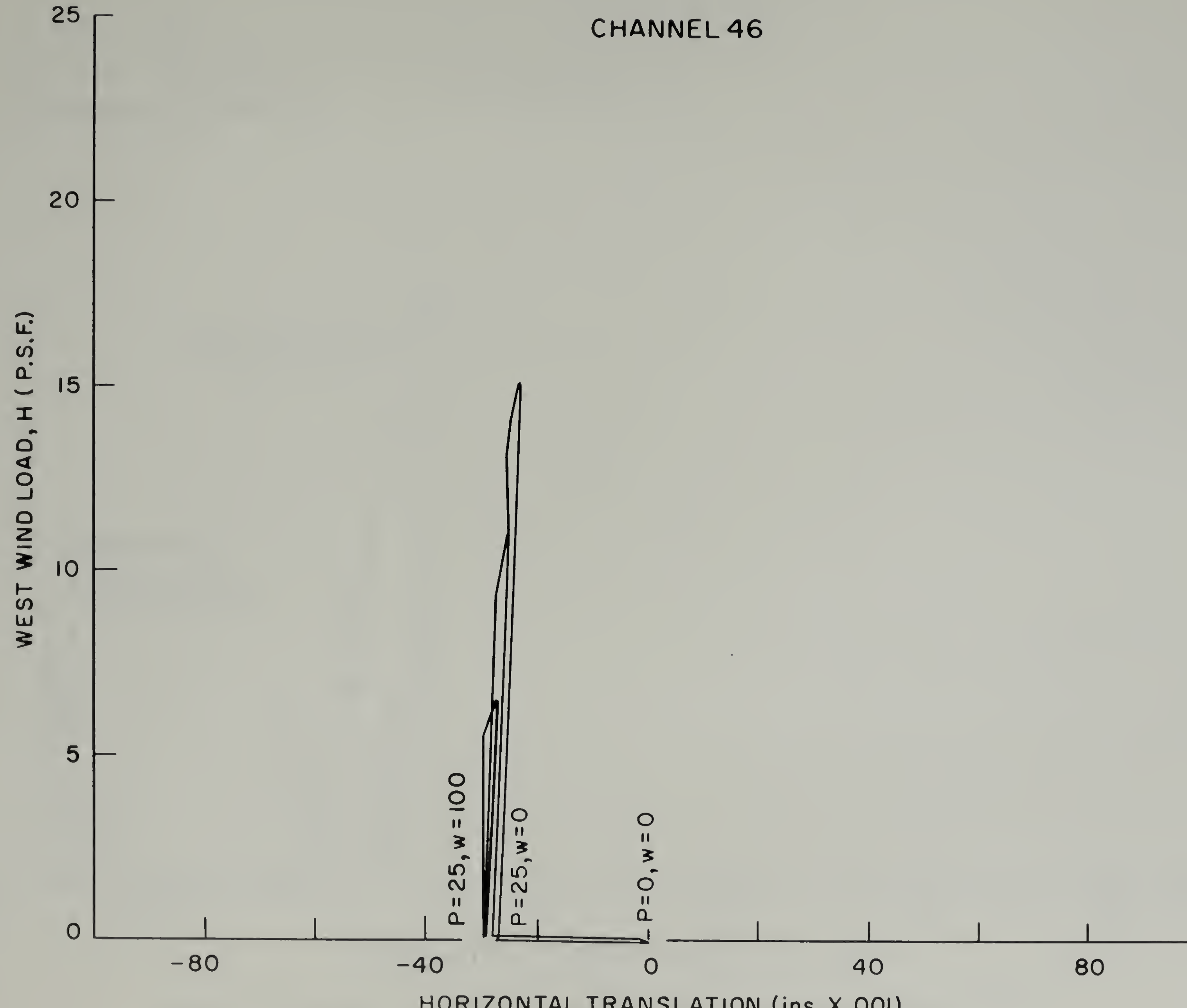


FIGURE 9.21 TEST NO. 8, WEST WIND LOAD VS. TRANSLATION.

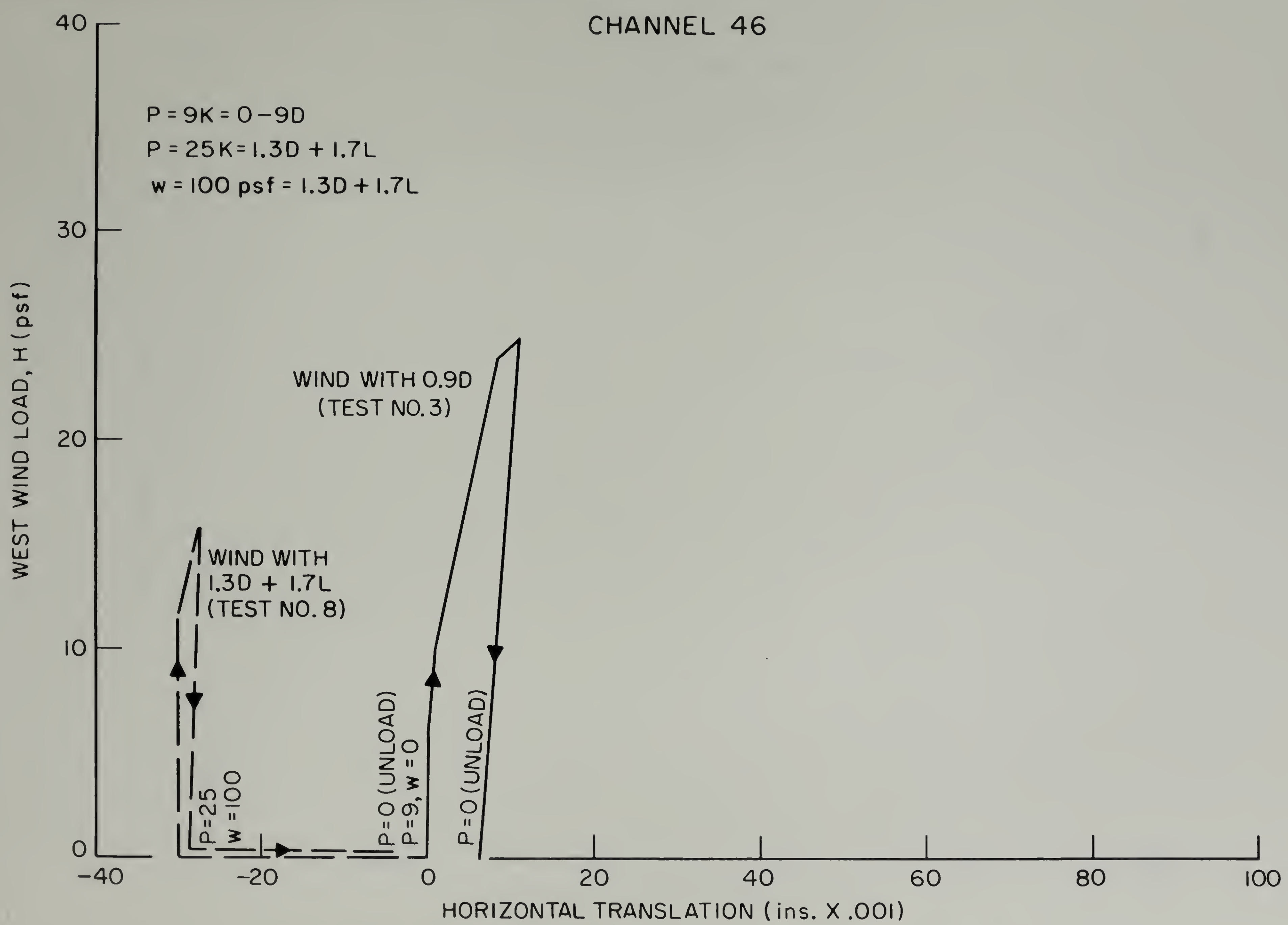


FIGURE 9.22 EFFECT OF VERTICAL LOADS ON E–W HORIZONTAL TRANSLATION.

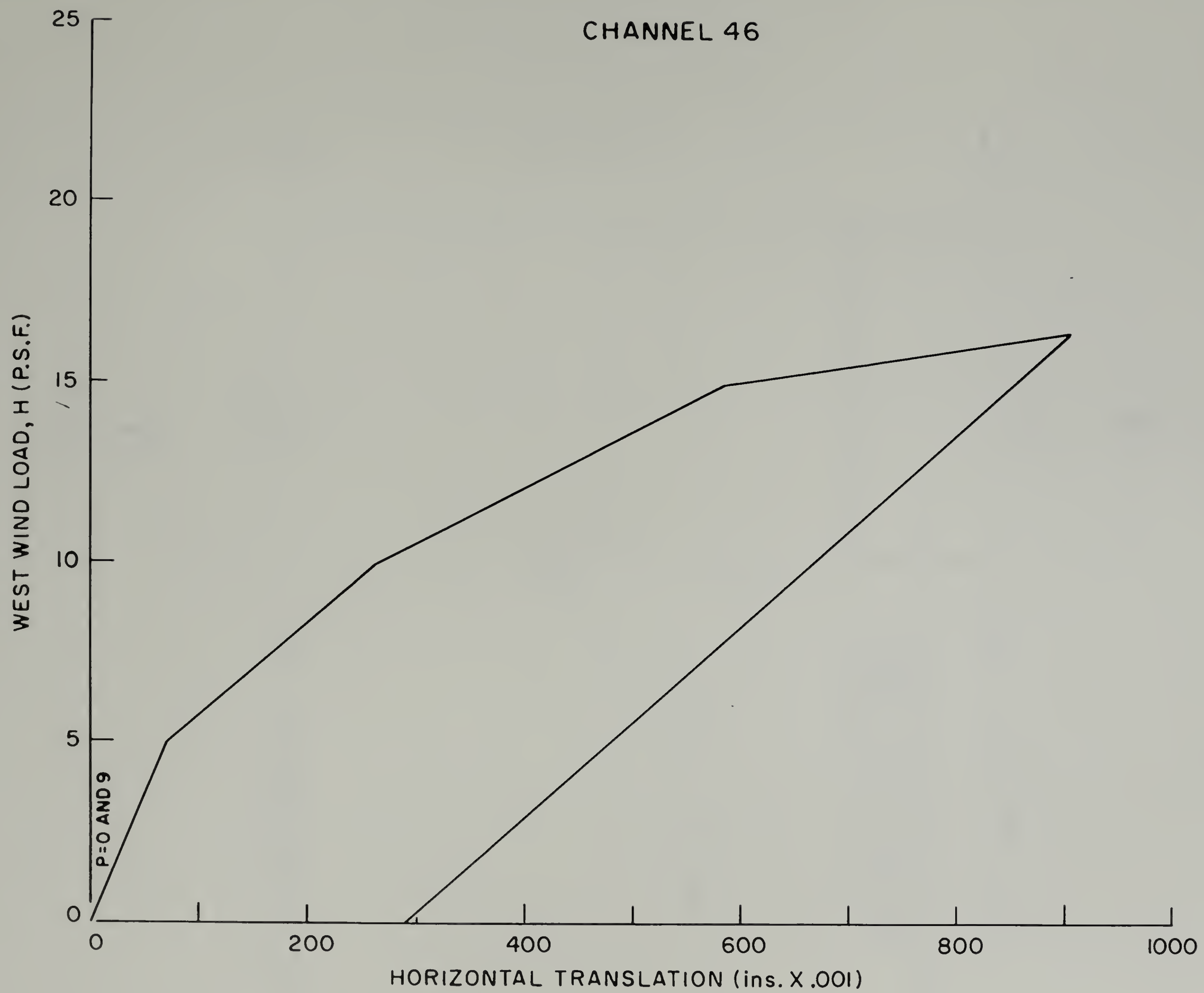


FIGURE 9.23 TEST NO. 15, WEST WIND LOAD VS. TRANSLATION.

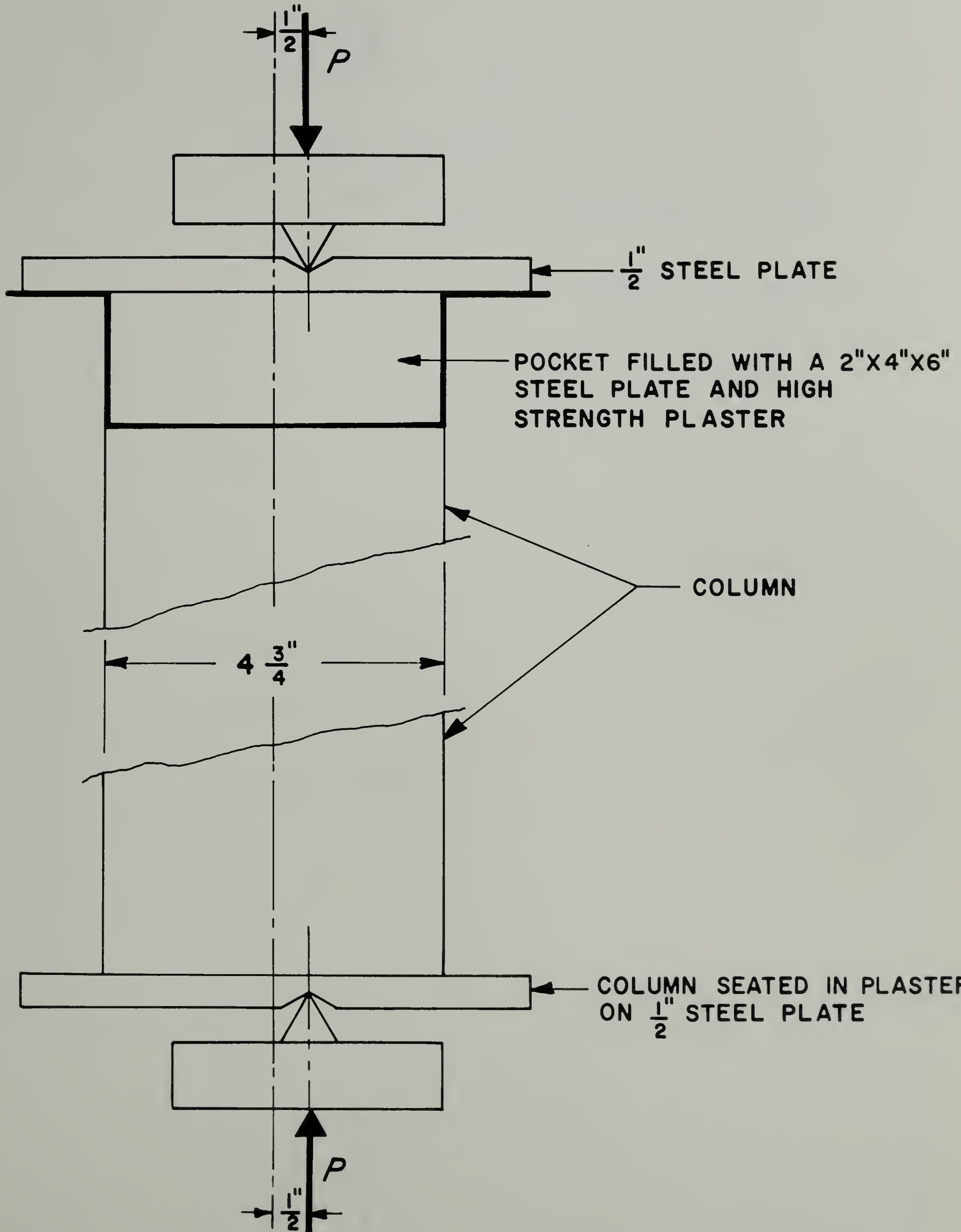


FIG. 10.1 TEST METHOD FOR COLUMN TESTS (Minor Axis)

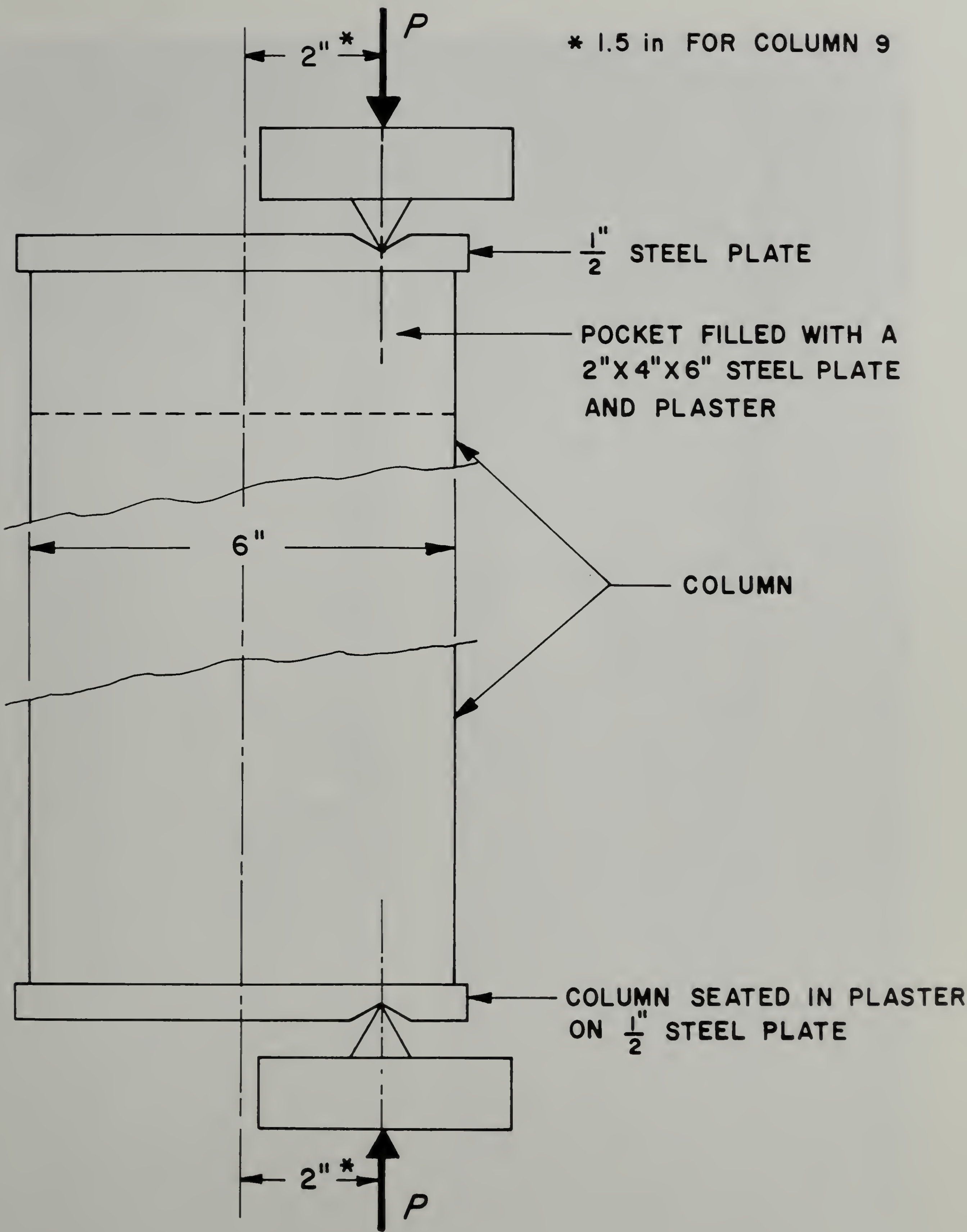


FIG. 10.2 TEST METHOD FOR COLUMN TESTS (Major Axis)

FIG. 10.3 COLUMN UNDER TEST

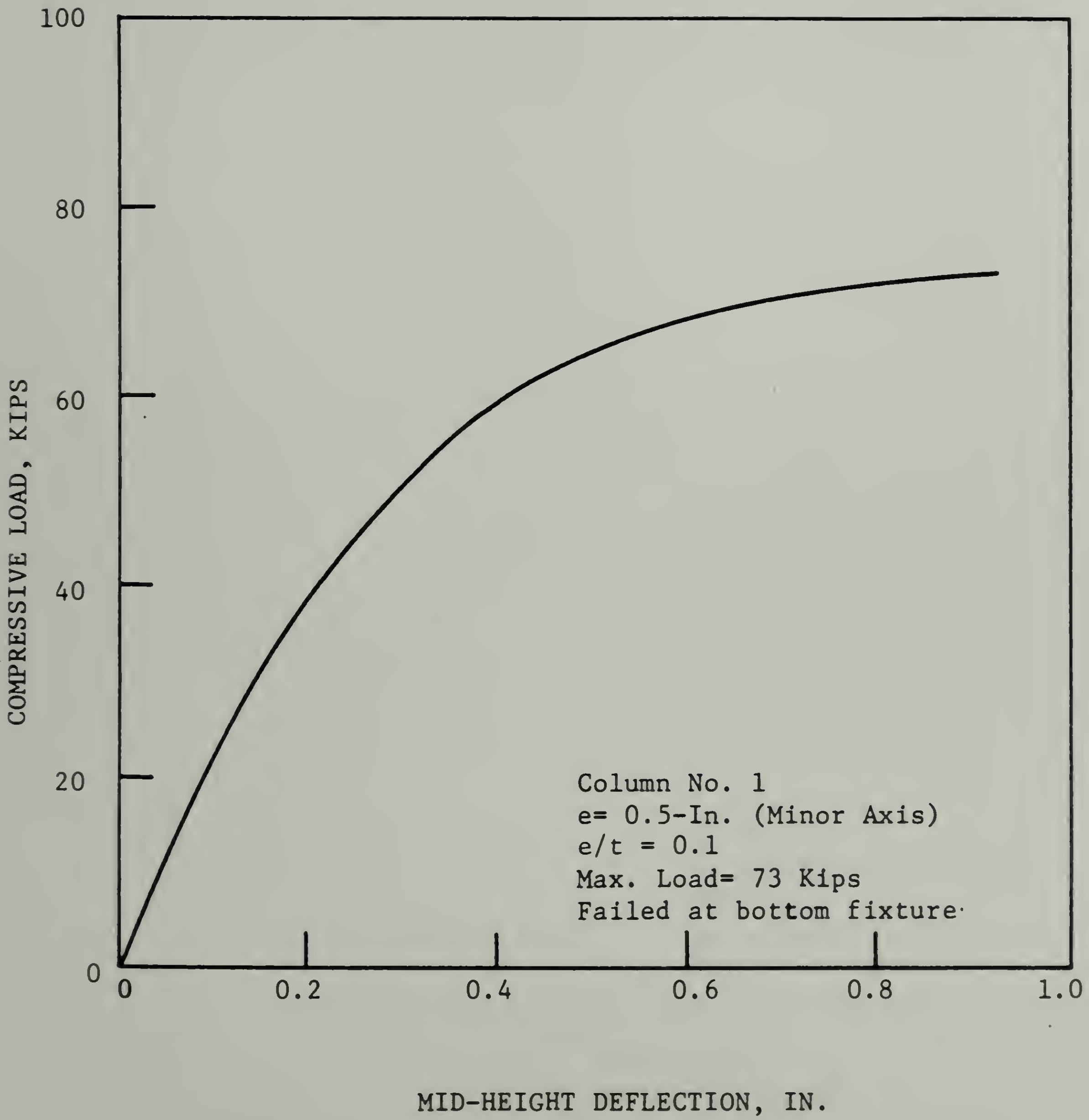


Figure 10.4 SHORT-TERM TEST ON COLUMN NO. 1

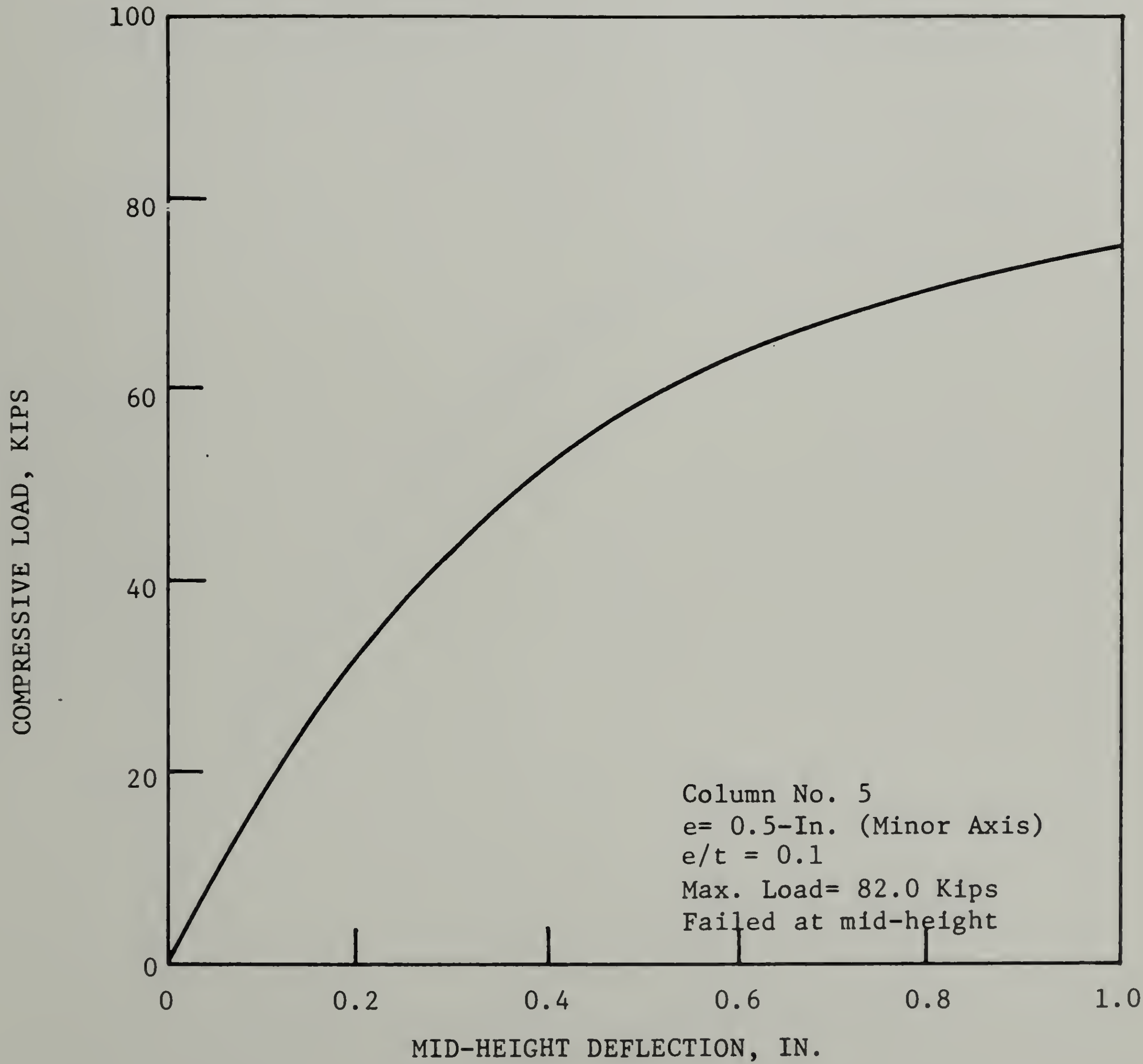


FIGURE 10.5 SHORT-TERM TEST ON COLUMN NO. 5

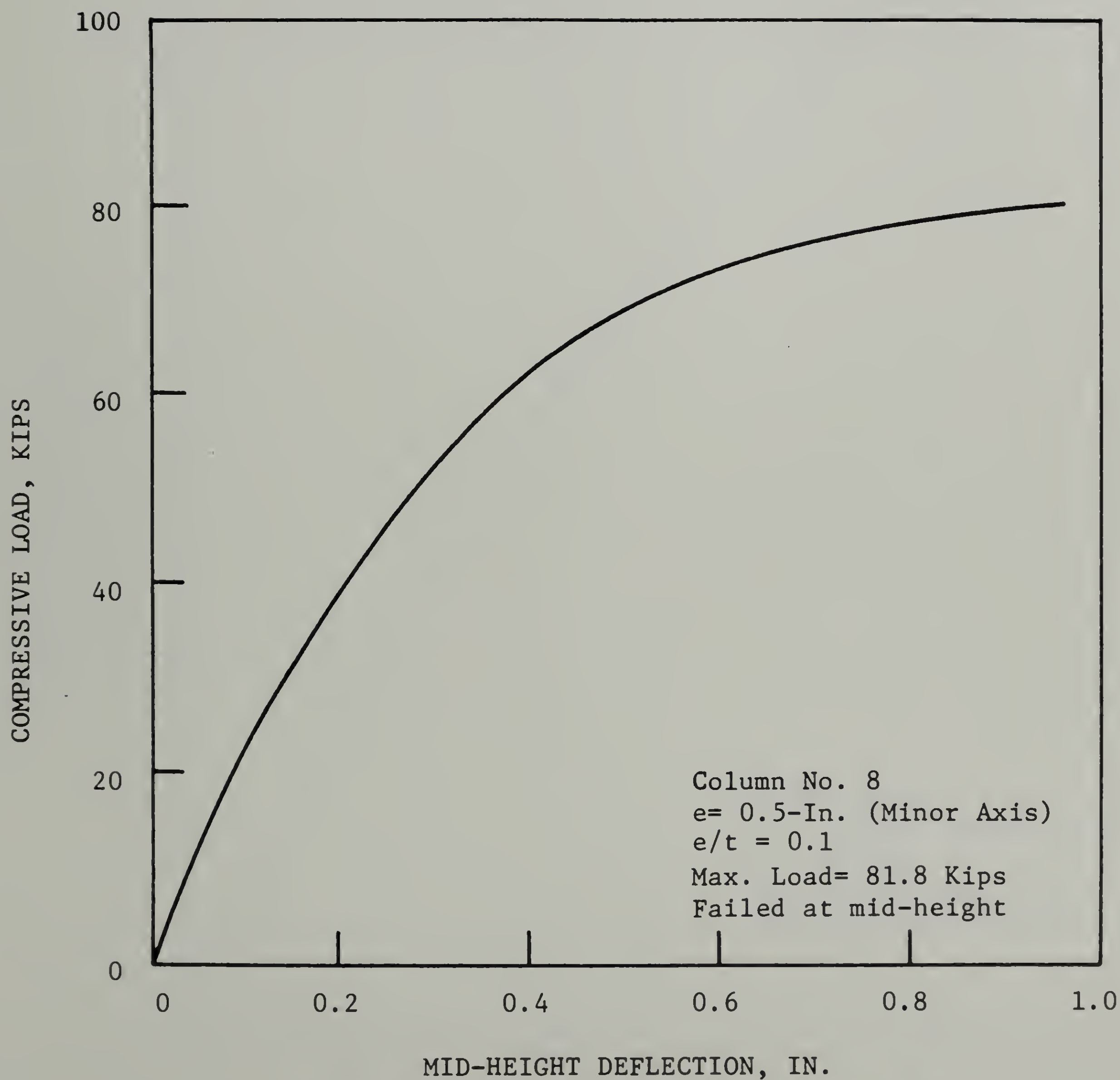


FIGURE 10.6 SHORT-TERM TEST ON COLUMN NO. 8

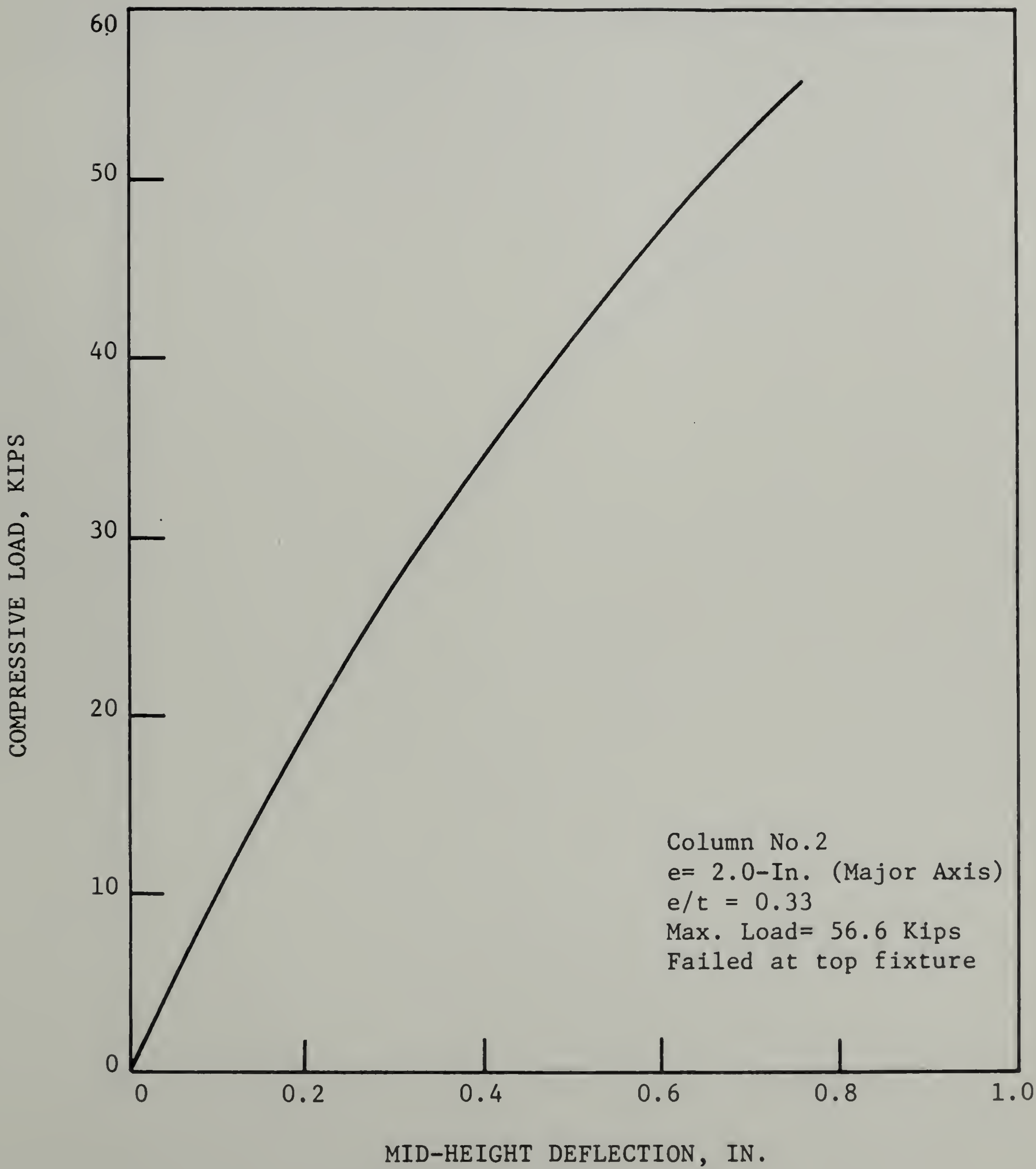


FIGURE 10.7 SHORT-TERM TEST ON COLUMN NO. 2

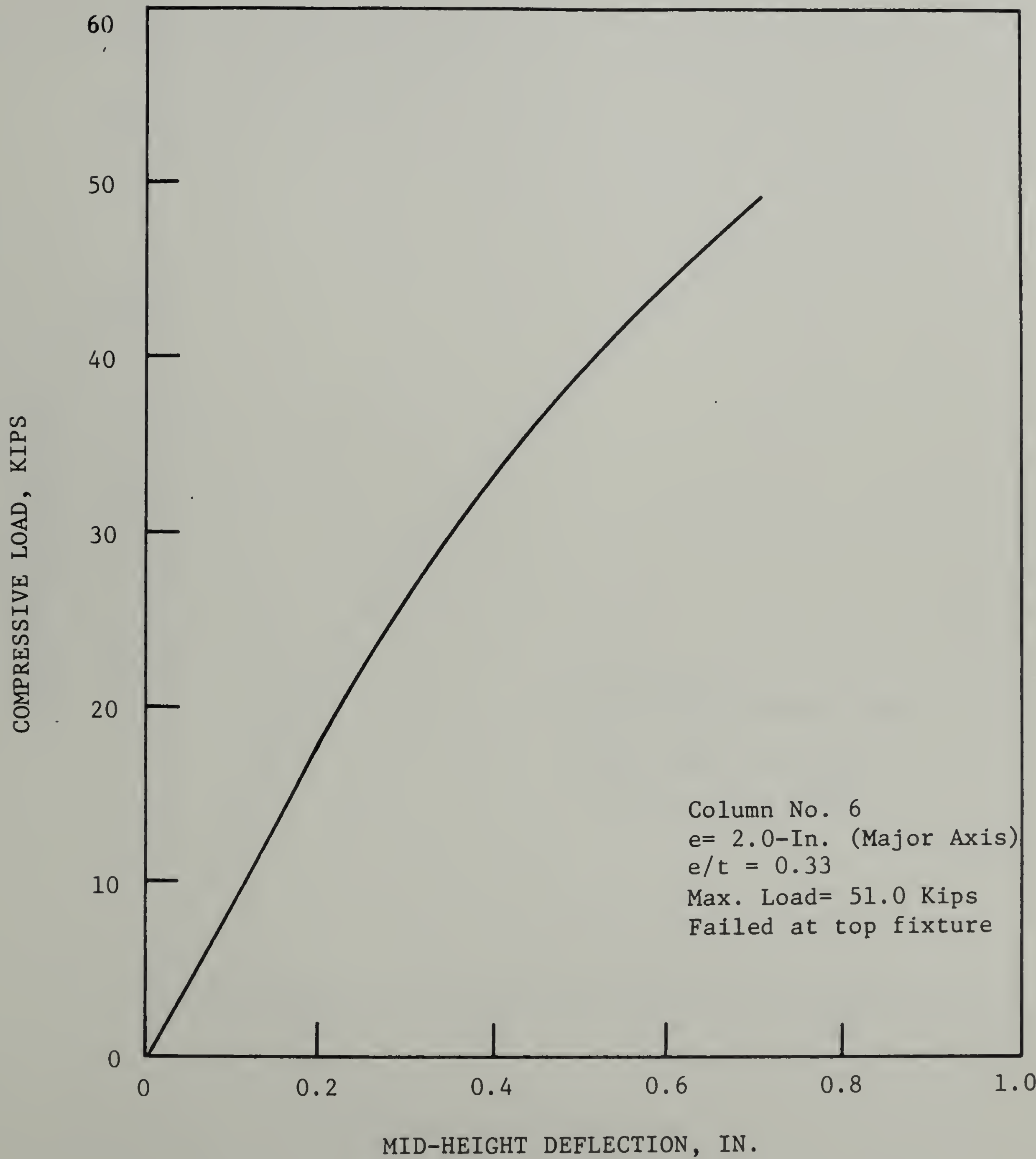


FIGURE 10.8 SHORT-TERM TEST ON COLUMN NO. 6

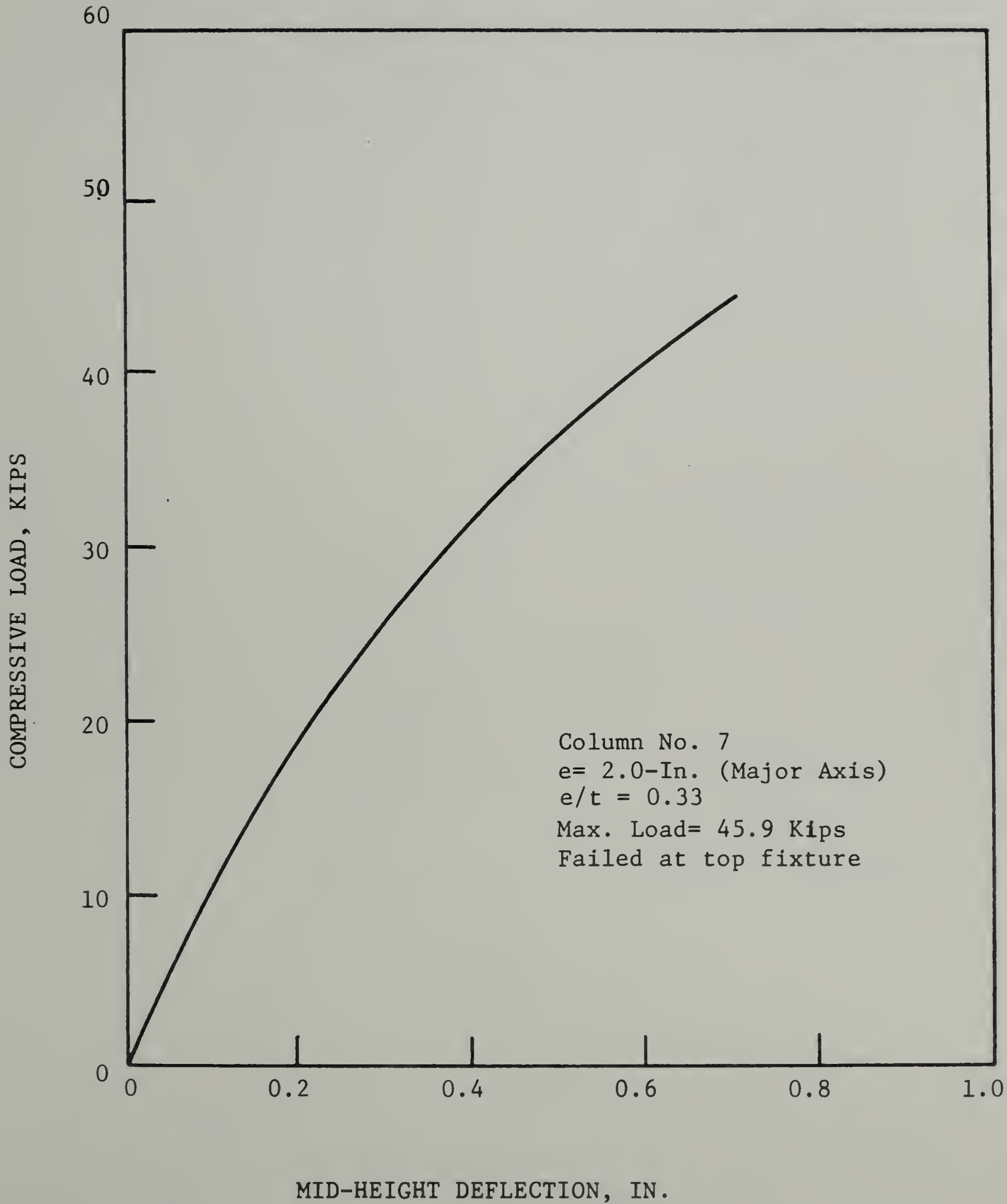


FIGURE 10.9 SHORT-TERM TEST ON COLUMN NO. 7

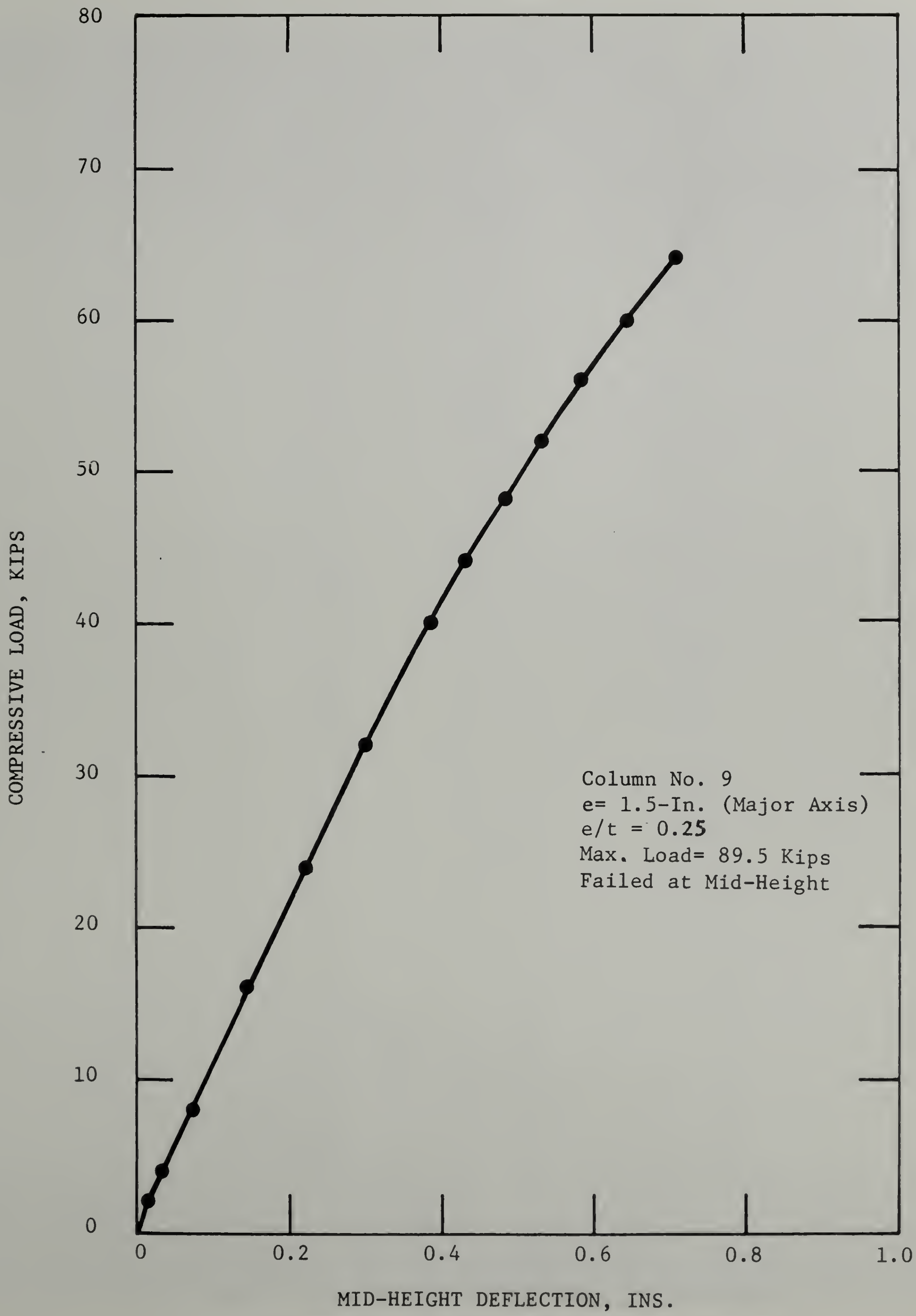


FIGURE 10.10 SHORT TERM TEST ON COLUMN NO. 9

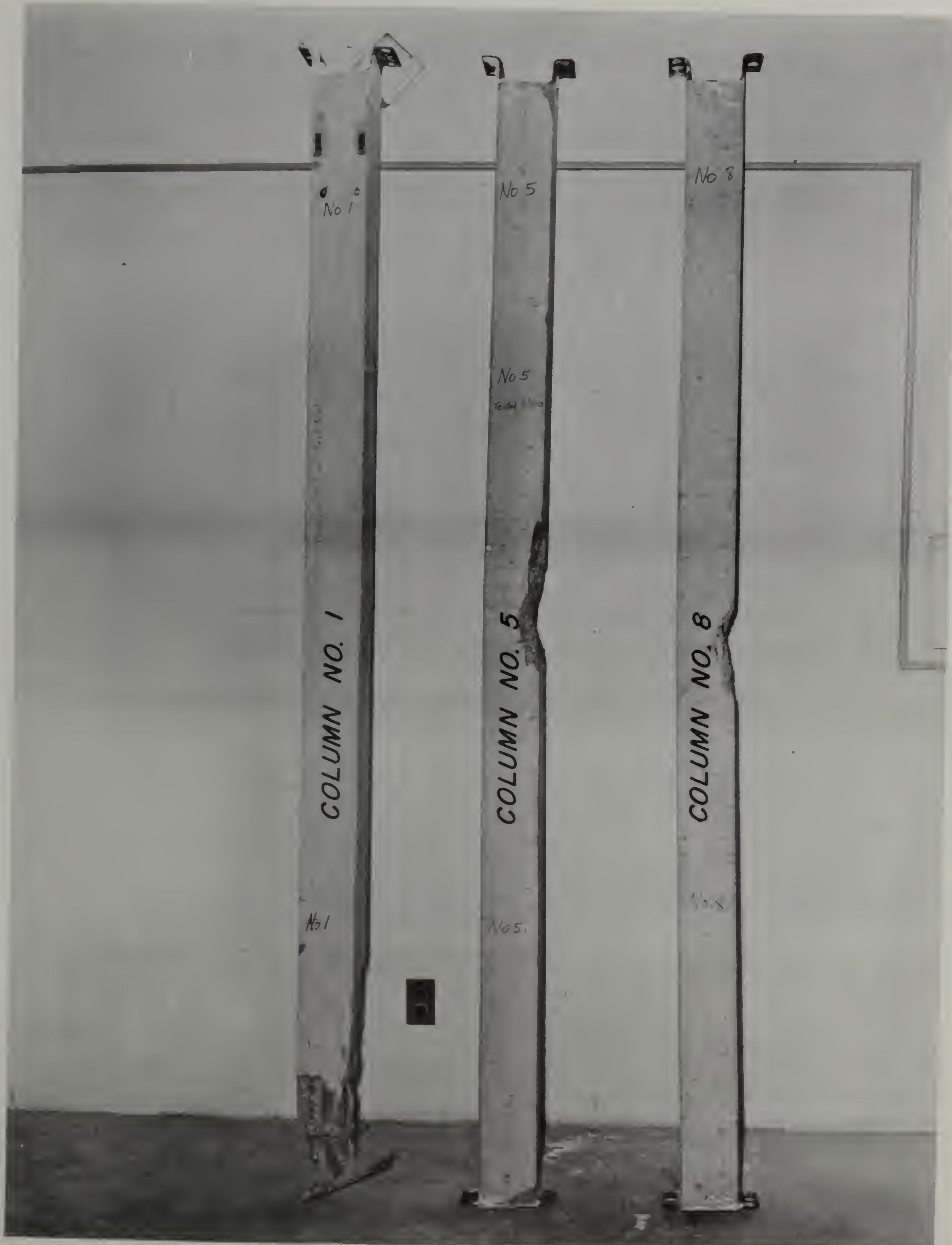


FIG. 10.11 COLUMNS AFTER TESTING

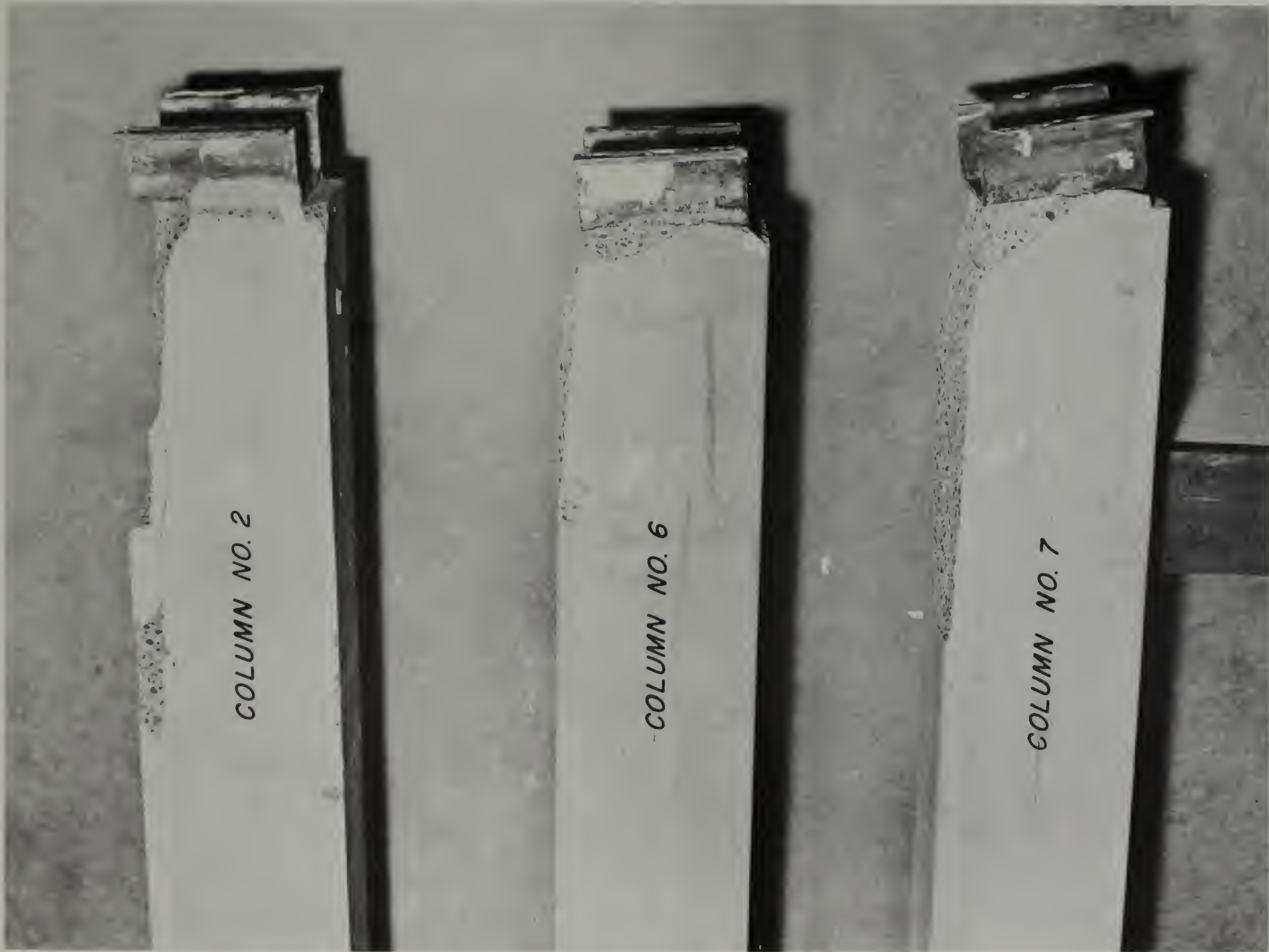


FIG. 10.12 COLUMN ENDS AFTER TESTING

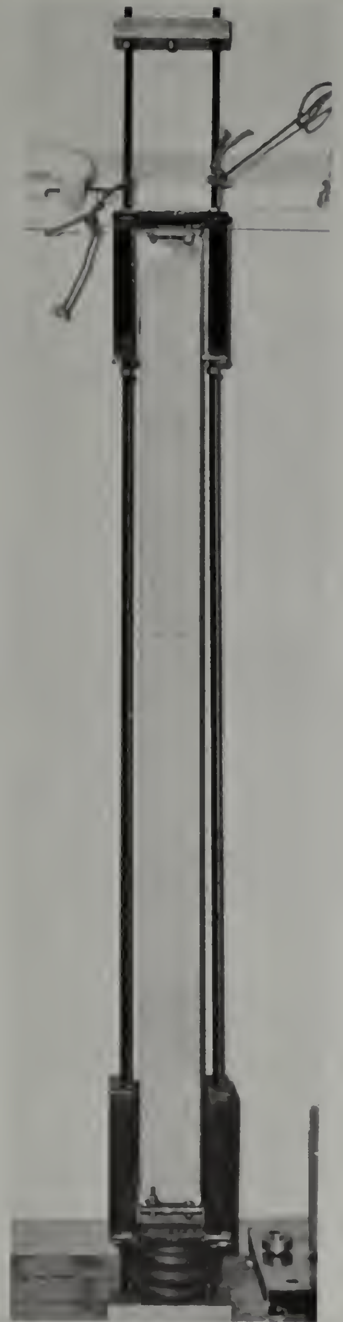


Fig. 10.13 Creep loading frame for column No. 3 minor axis bending

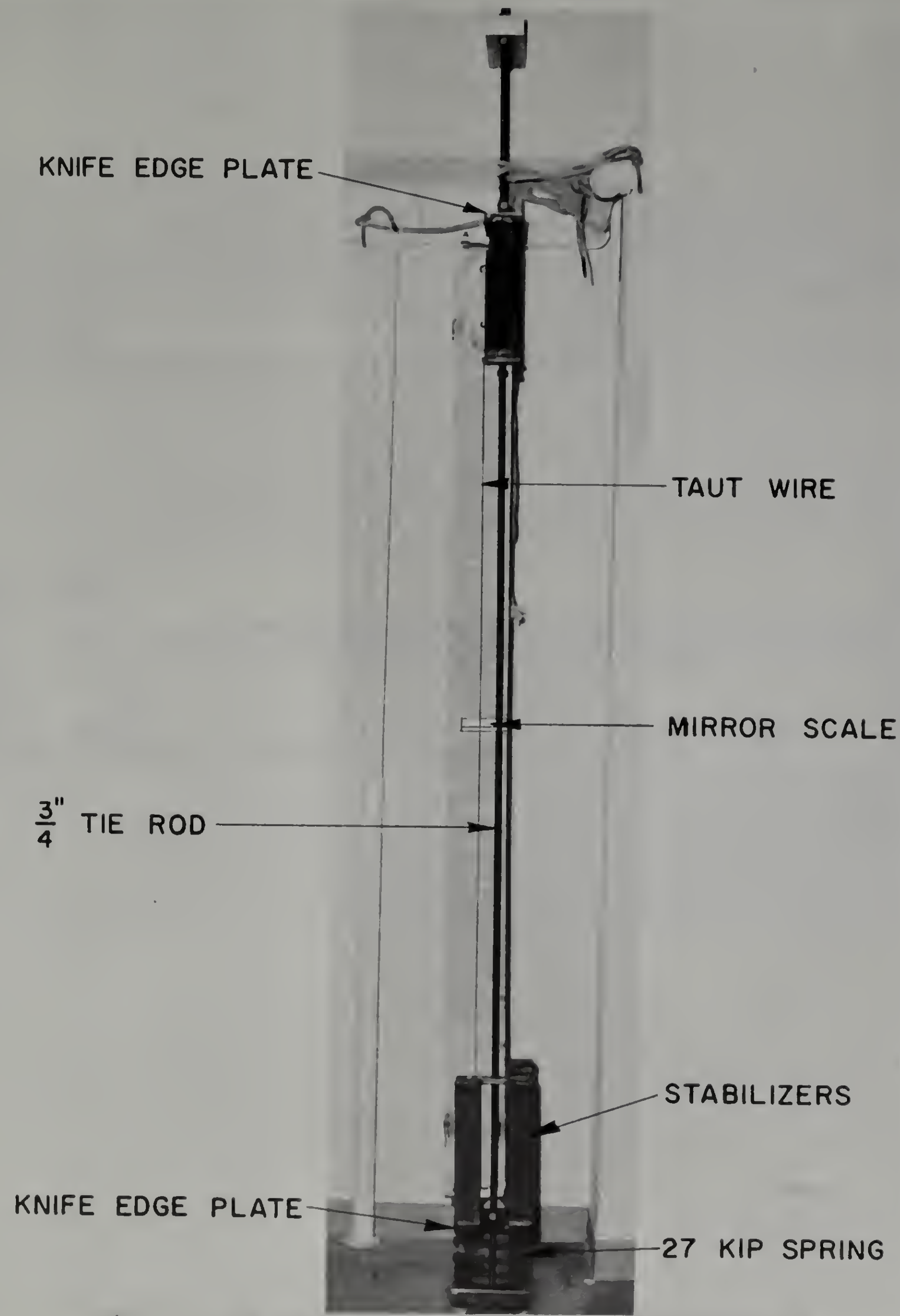


Fig. 10.14 Creep loading frame for column No. 4 major axis bending

Fig. 10.15 Detail of base for column No. 4 creep frame

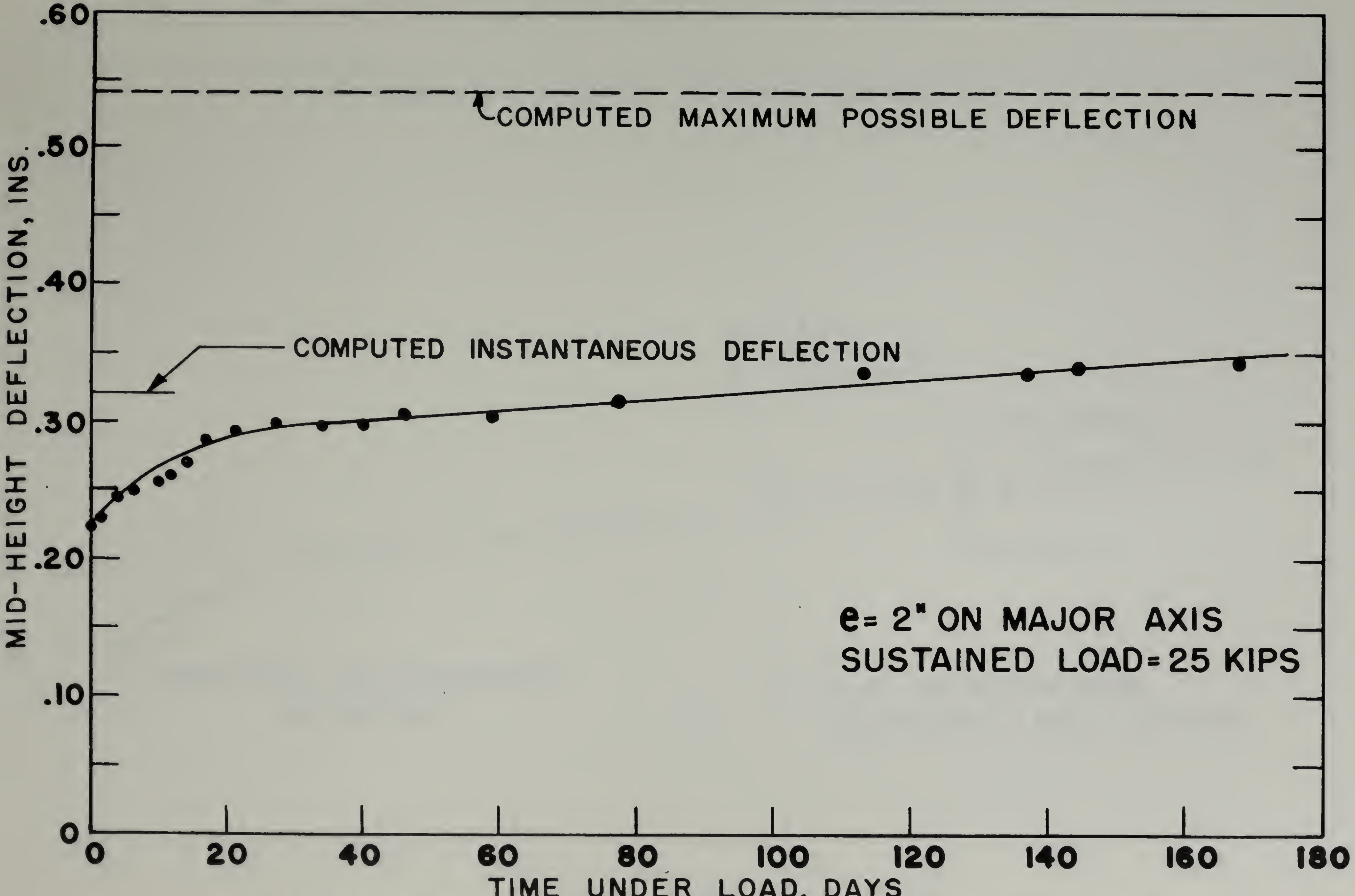


FIGURE 10.16 - SUSTAINED LOAD TEST, COLUMN NO. 4

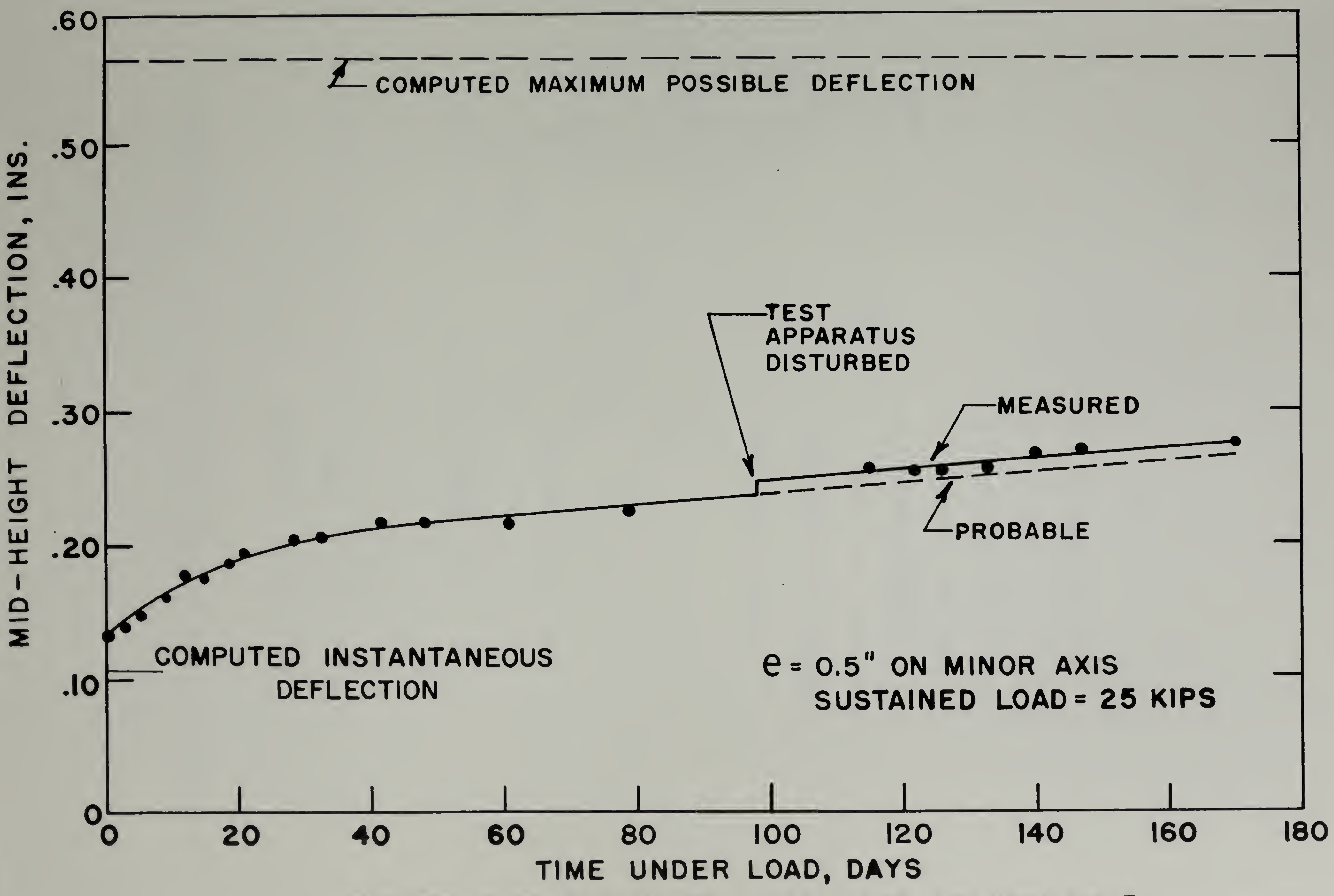


FIGURE 10.17 - SUSTAINED LOAD TEST, COLUMN NO. 3

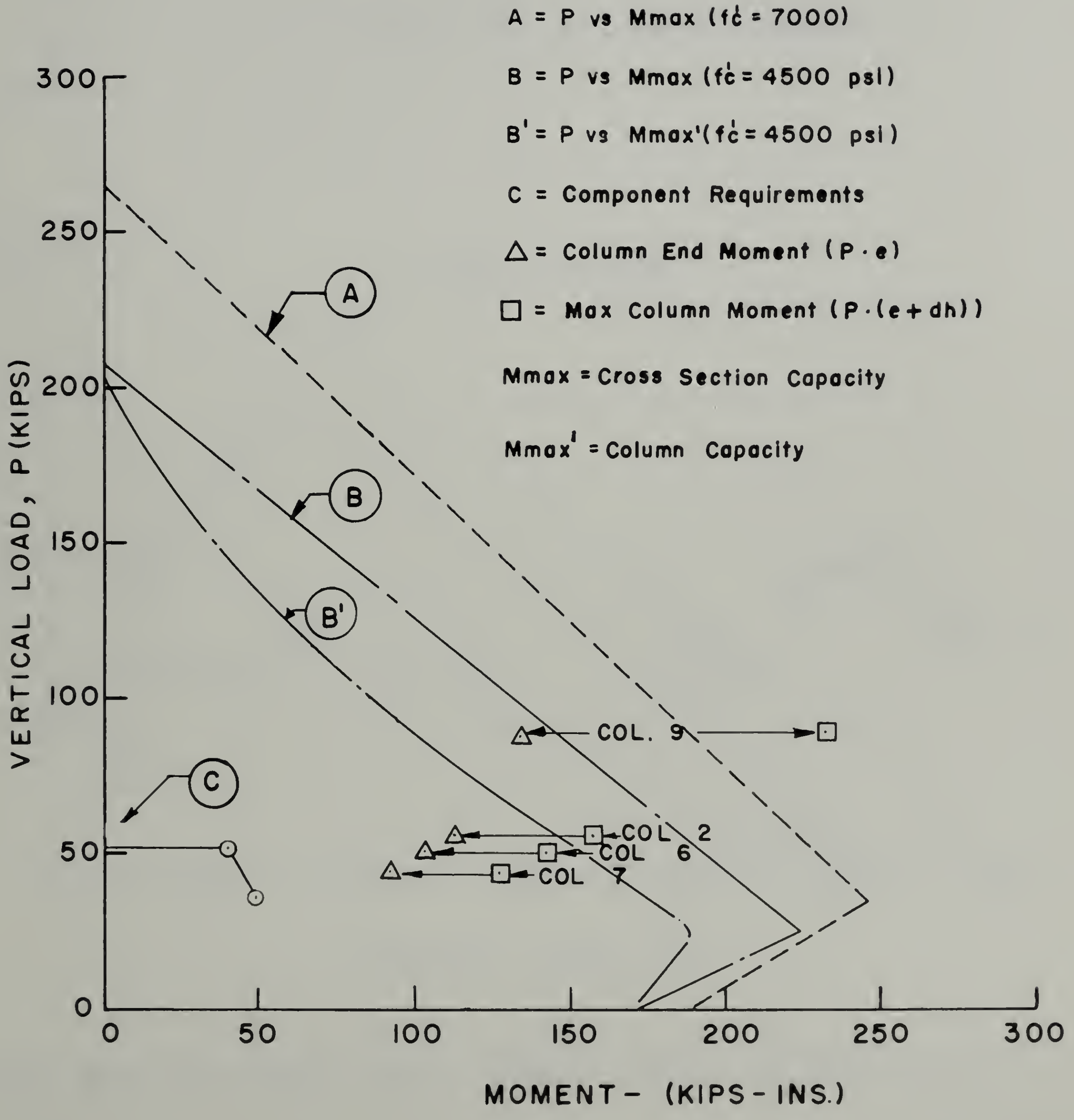


FIGURE 10.18 COLUMNS WITH MAJOR AXIS ECCENTRICITY

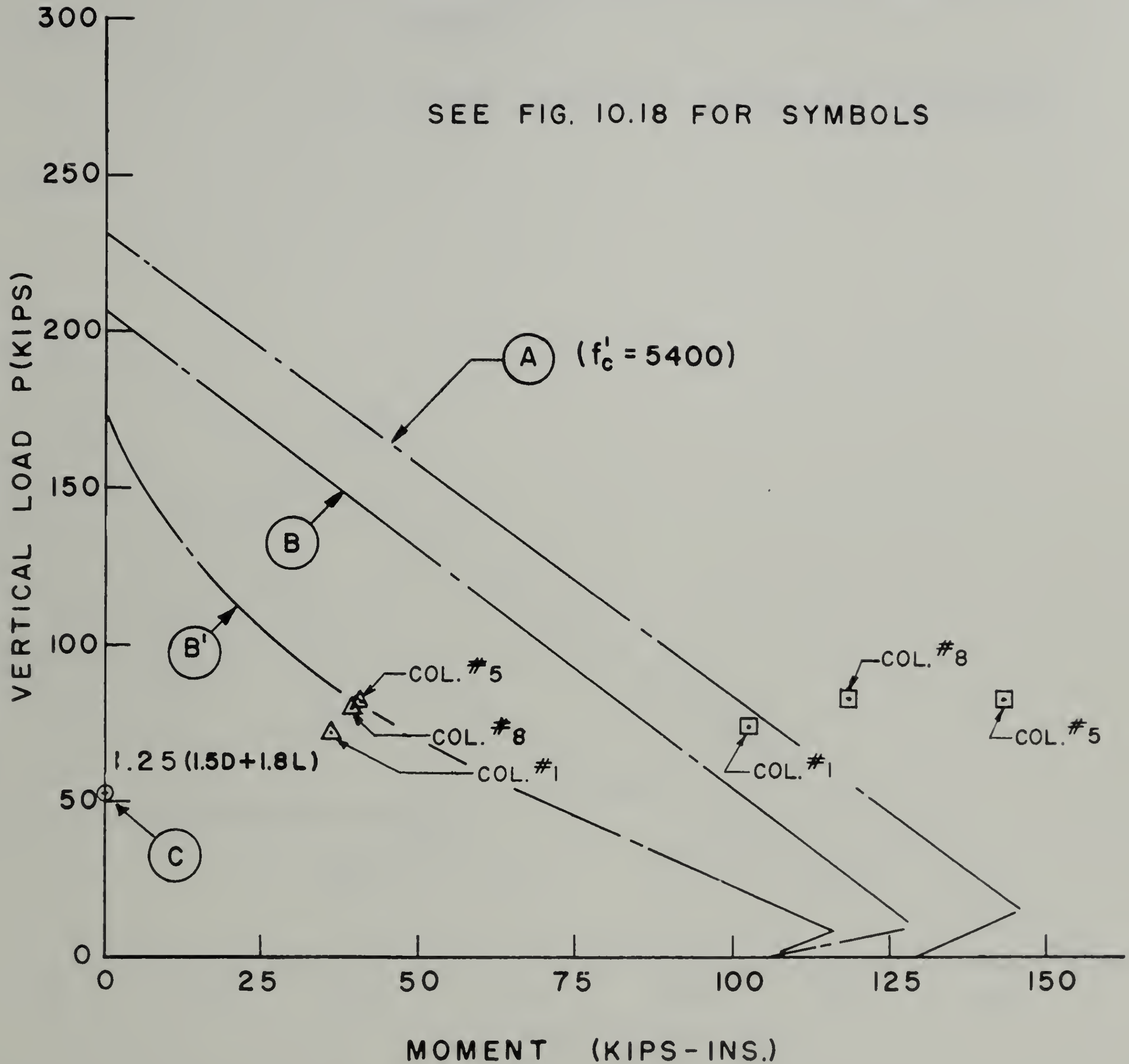


FIGURE 10.19 COLUMNS WITH MINOR AXIS ECCENTRICITY

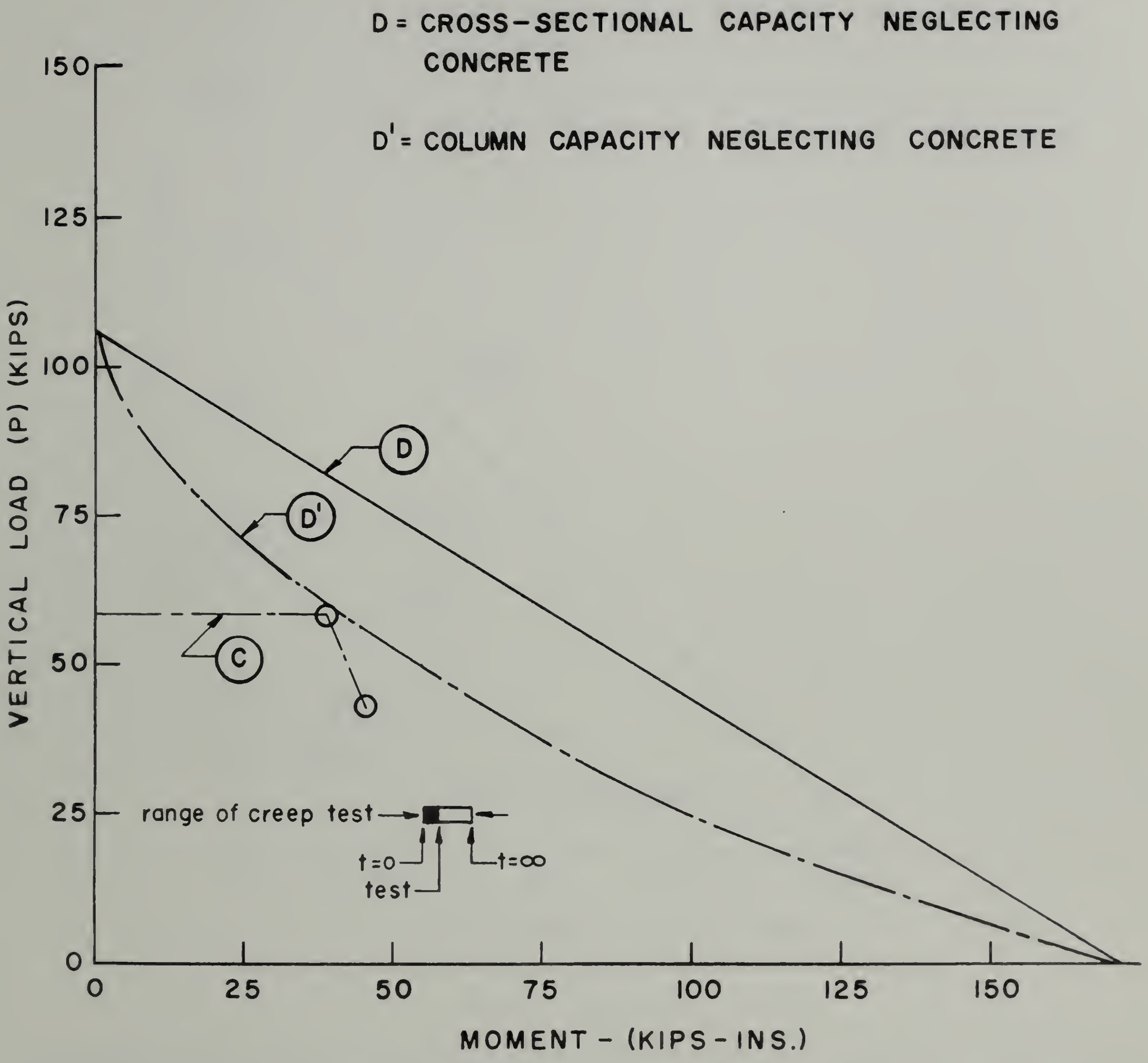


FIGURE 10.20 COLUMN CREEP WITH MAJOR AXIS ECCENTRICITY

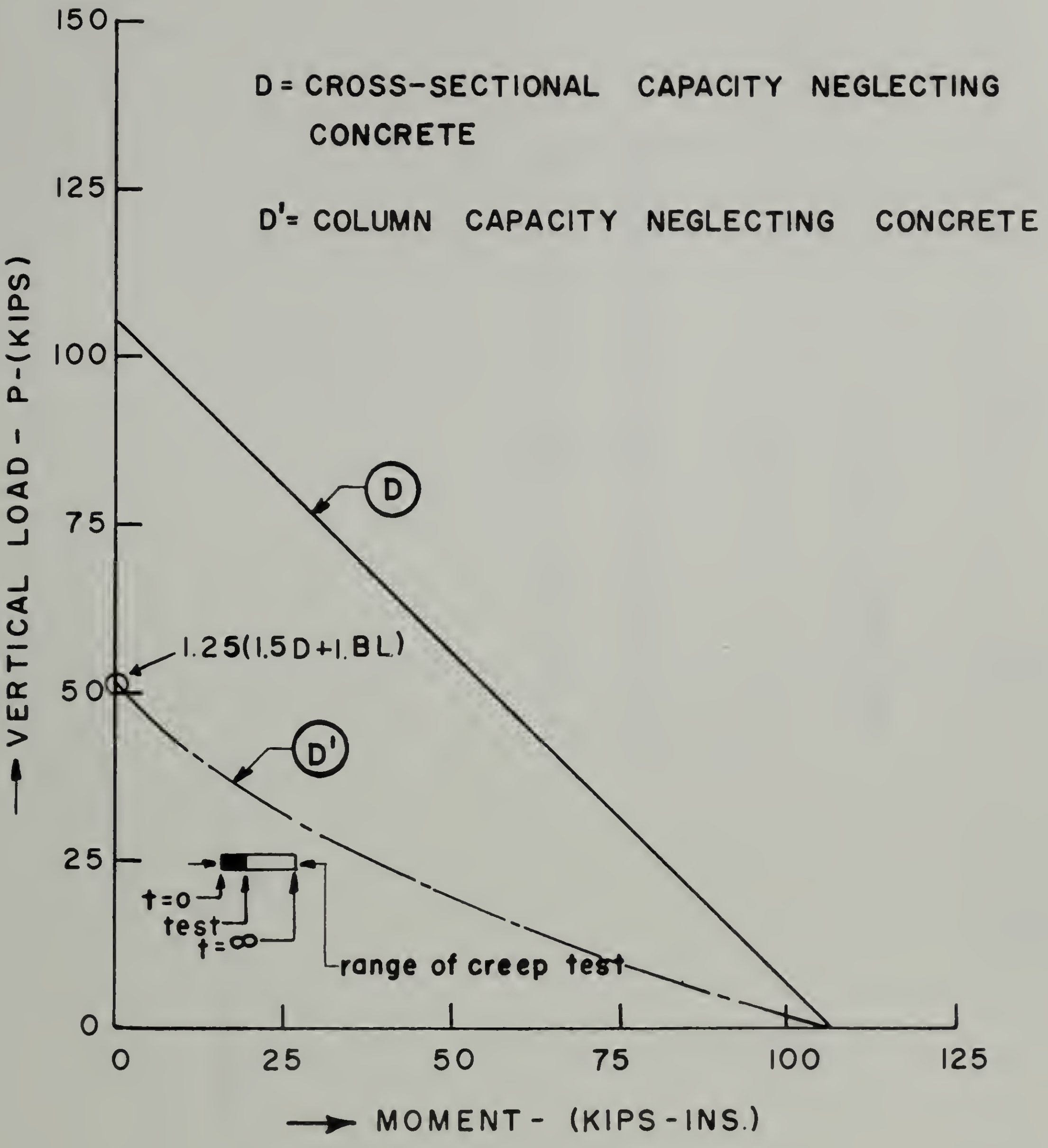


FIGURE 10.21 COLUMN CREEP WITH MINOR AXIS ECCENTRICITY

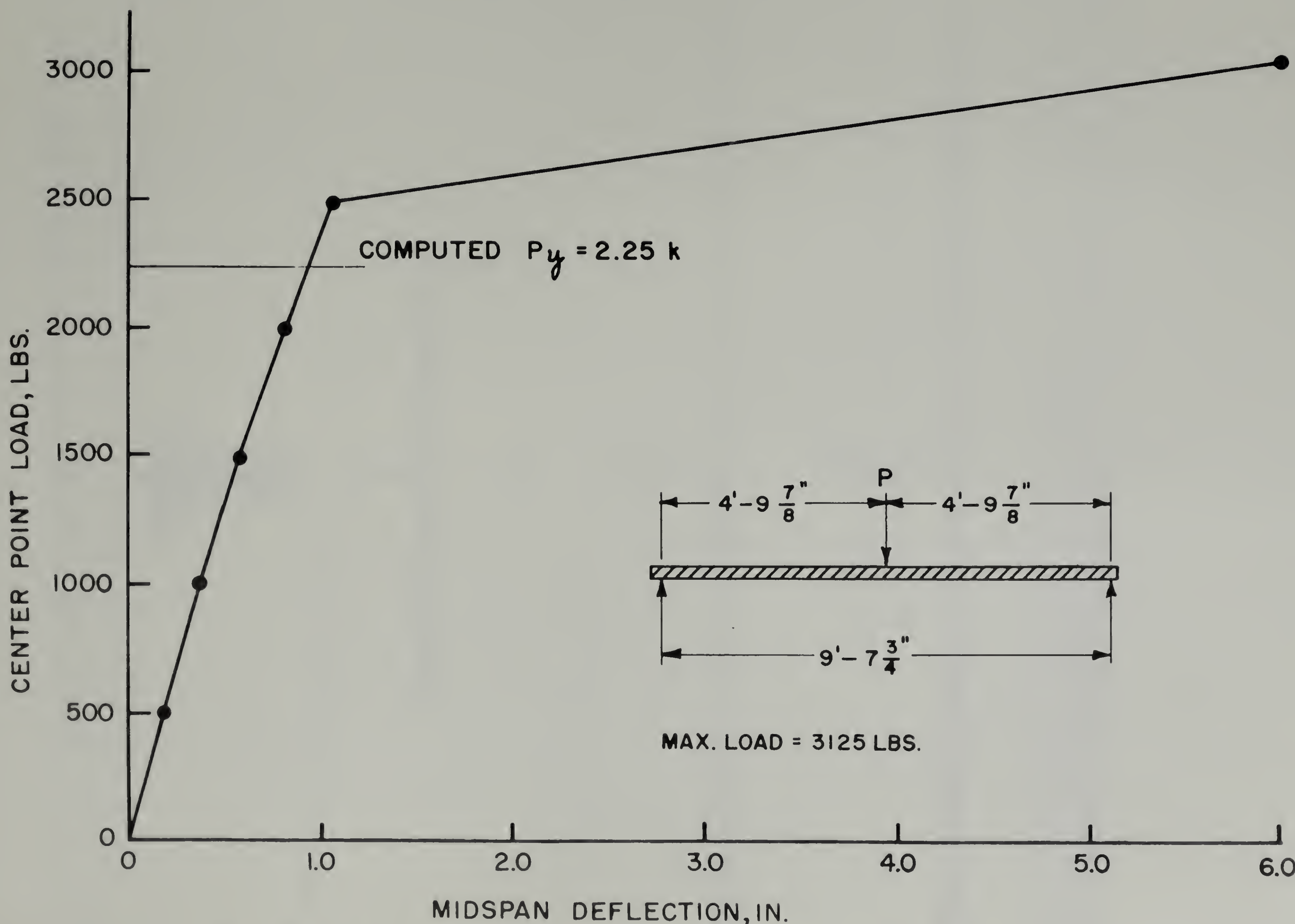


FIGURE 10.22 CHANNEL ROOF SLAB TEST

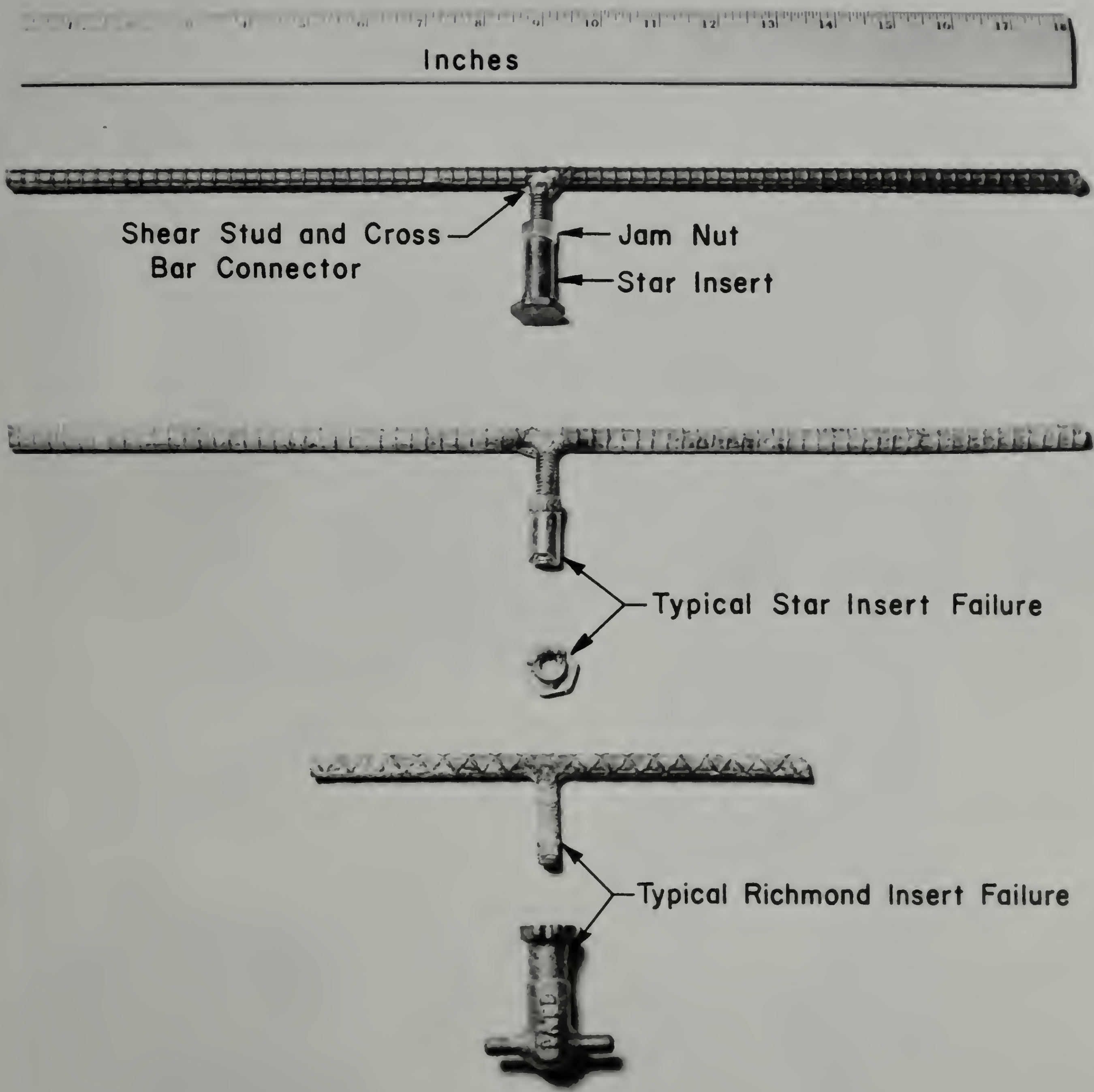


FIG. 10.23 BEAM TO SLAB SHEAR CONNECTORS

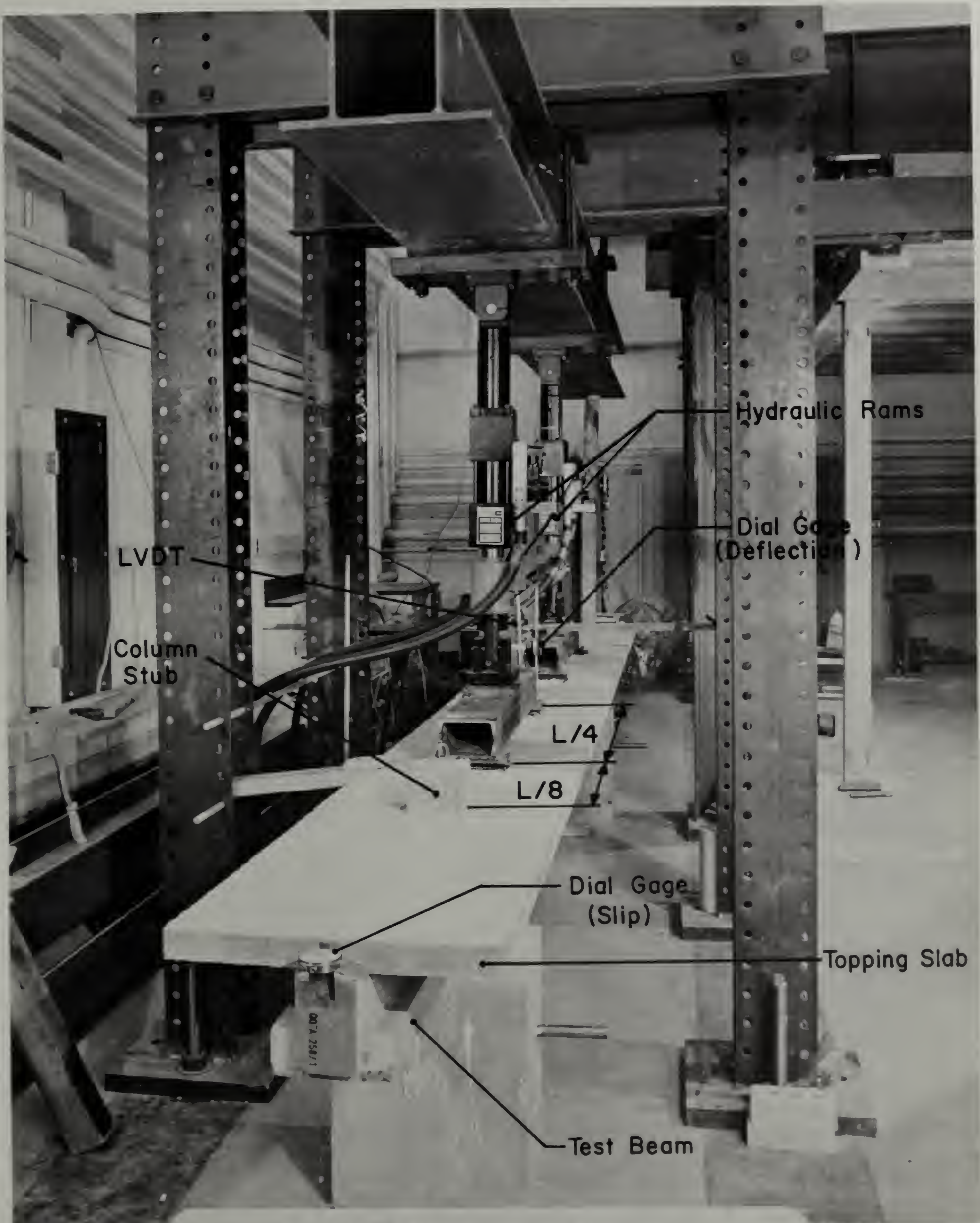


Fig. 10.24 Fatigue test on main beam

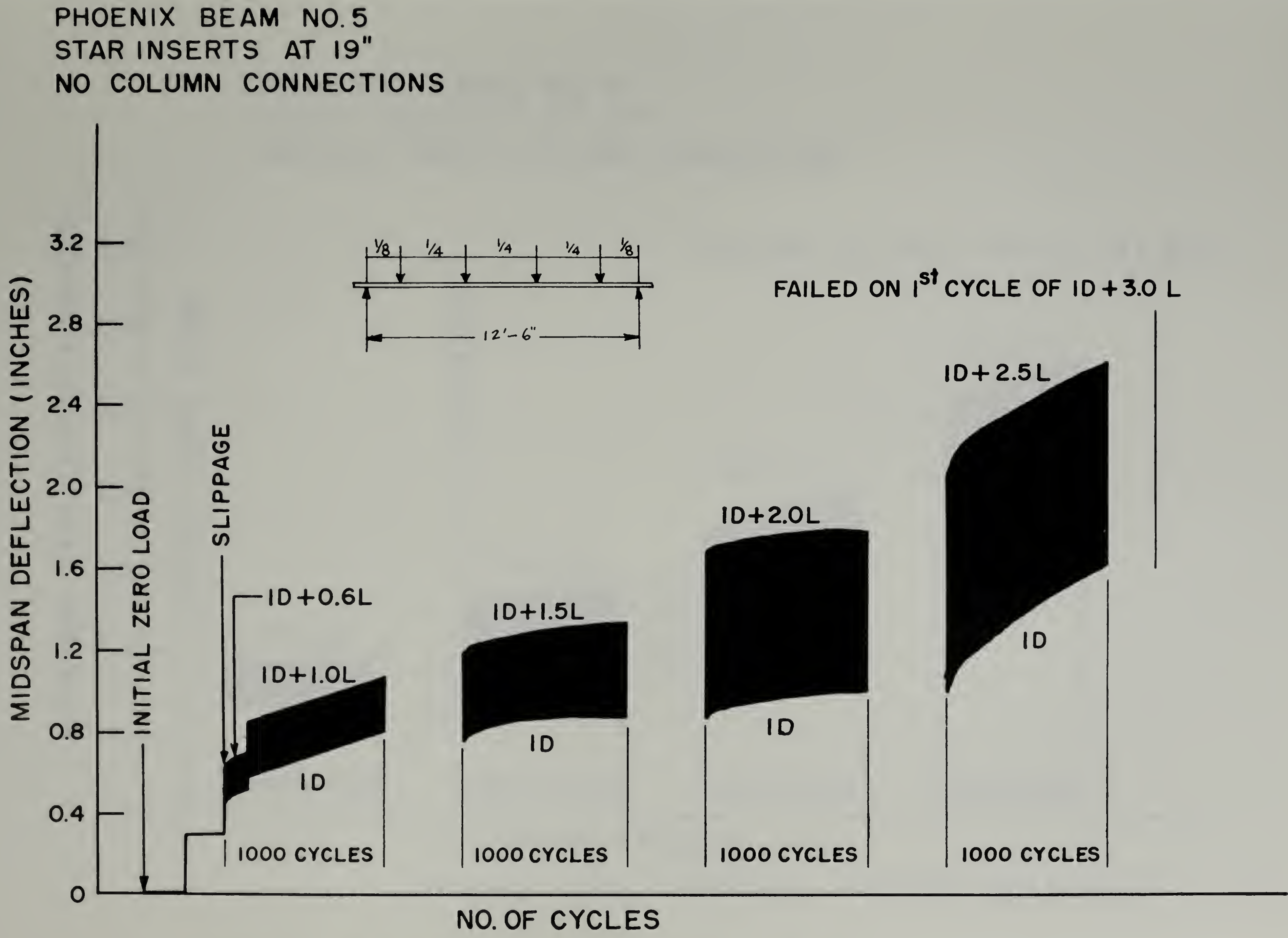


FIGURE 10.25 FATIGUE LOADING, BEAM NO. 5

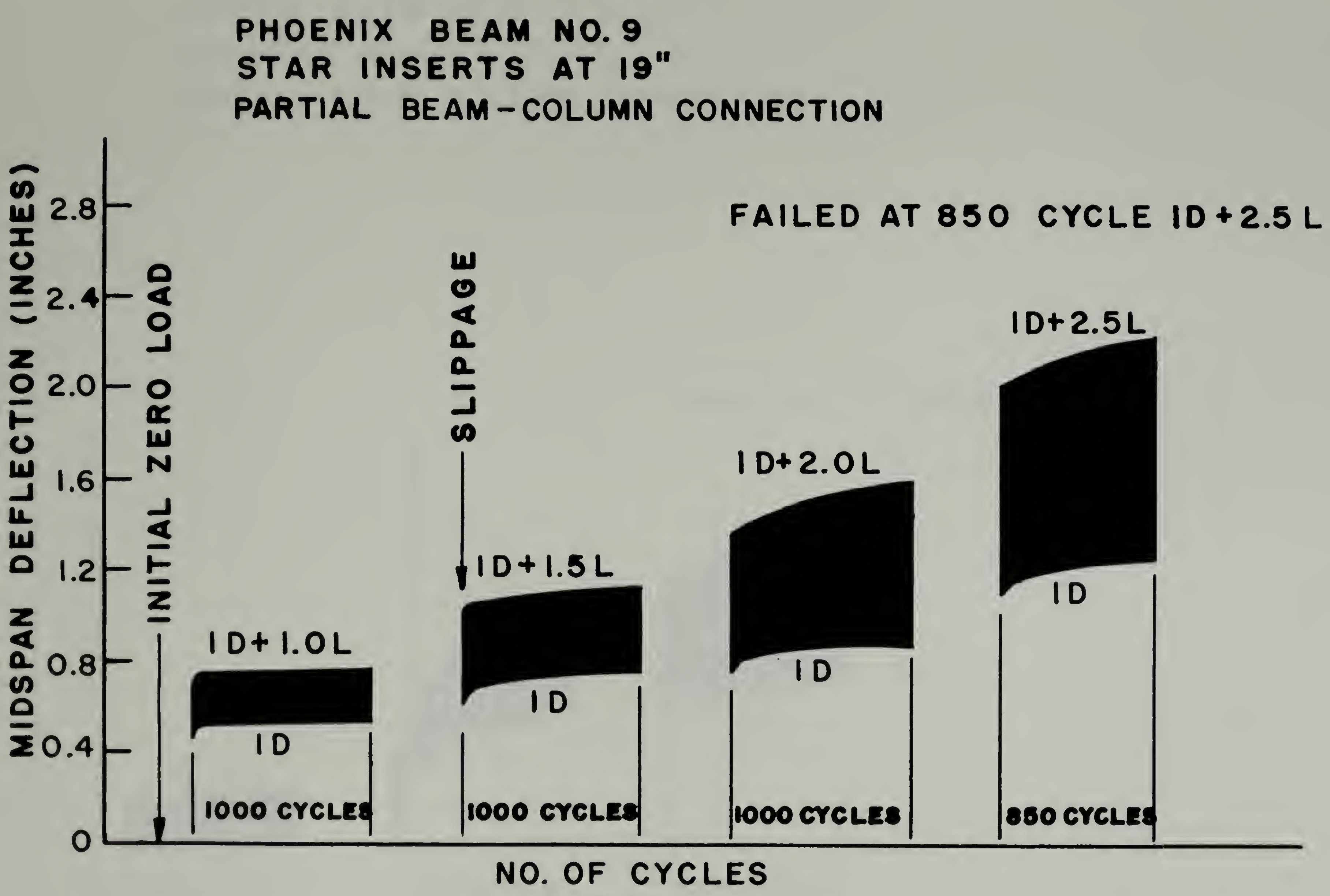


FIGURE 10.26 FATIGUE LOADING, BEAM NO. 9

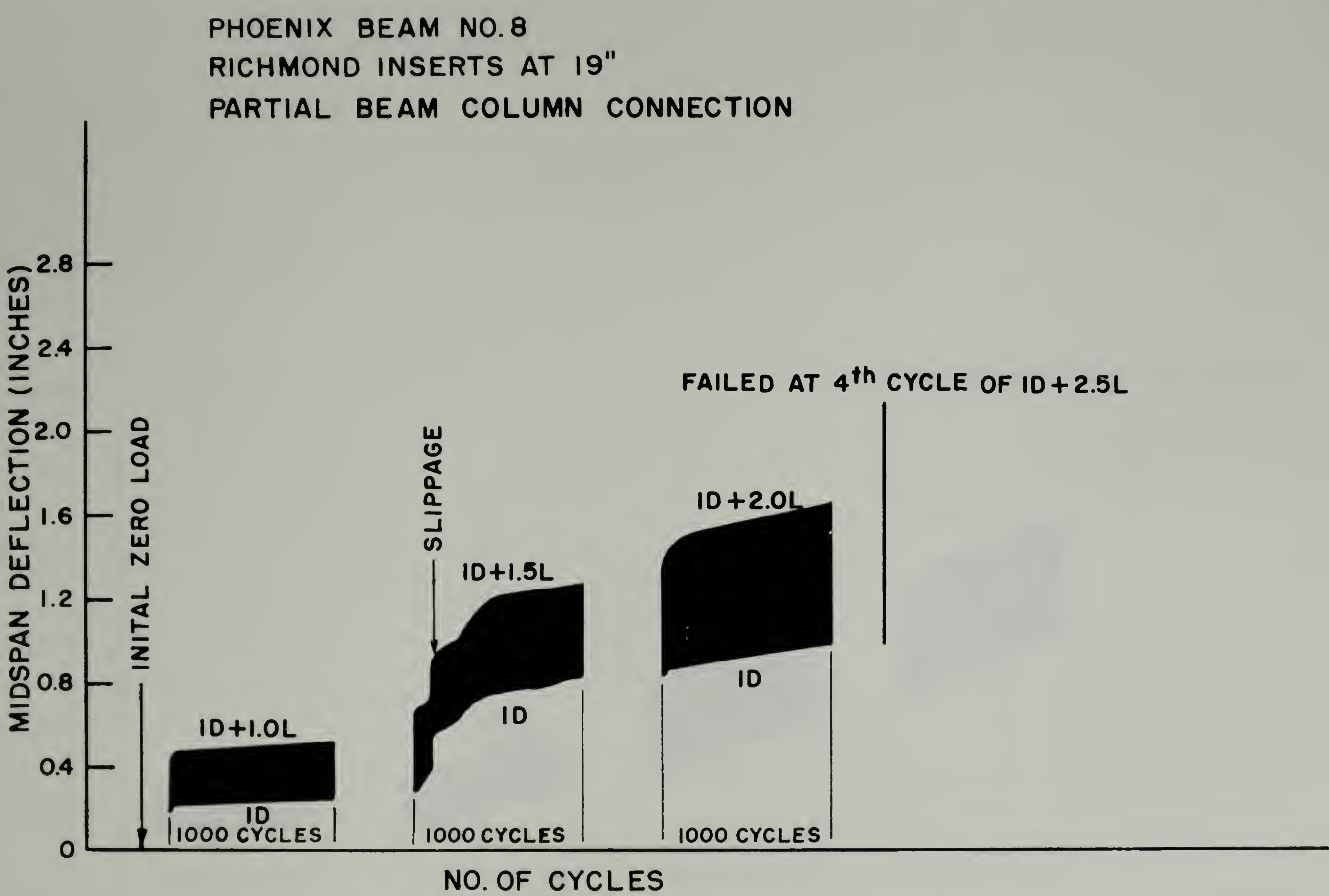


FIGURE 10.27 FATIGUE LOADING, BEAM NO. 8

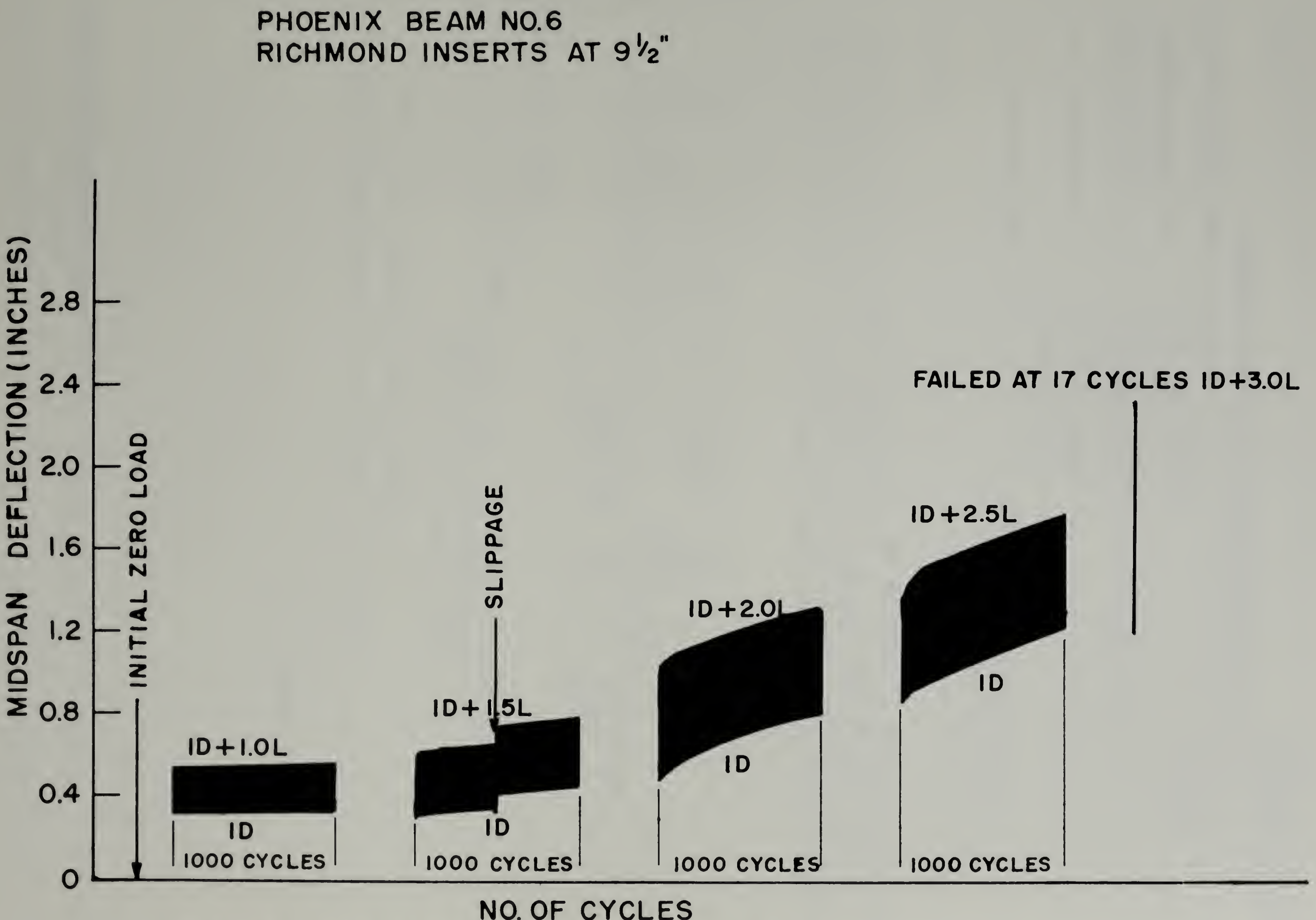


FIGURE 10.28 FATIGUE LOADING, BEAM NO. 6

FIGURE 11.1

PAGE OF PAGES

FORM PR-221
(7-67)

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

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

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

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

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

QUANTITY REPRESENTED	SOURCE OF MATERIAL
--	National Bureau of Standards

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

TEST RESULTS

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

The data recorded were:

Number of voids smaller than .02 inches = 2114
Number of voids larger than .02 inches = 151
Total number of voids (N) ----------- = 2265
Total length of traverse (T) -------- = 124.76 inches
Traverse in inches over voids smaller than .02 inches = 7.83 inches
" " " " " larger than .02 " = 6.22 "
" " " " all voids ----------------- = 14.05 "

Calculated values:

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

Voids in porous aggregate were not counted.

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

M. C. Gleason

~~CHIEF, DIVISION OF PHYSICAL RESEARCH~~	PER
Chief, Materials Division	M C Gleason

EDITION OF 6-59 OF THIS FORM MAY BE USED

APPENDIX A

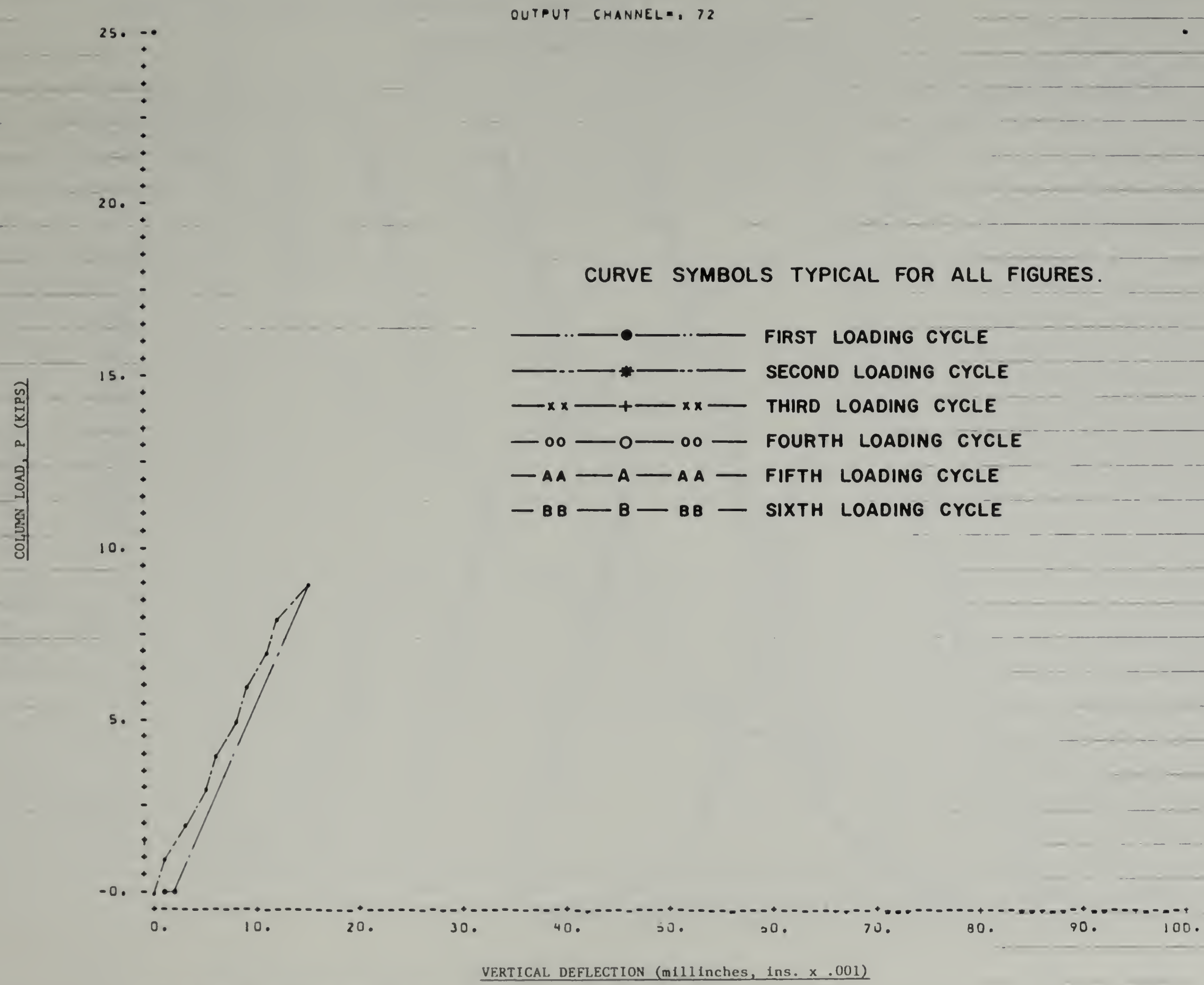


FIGURE NO. A.1 - TEST NO. 1, COLUMN LOAD VS. BEAM DEFLECTION

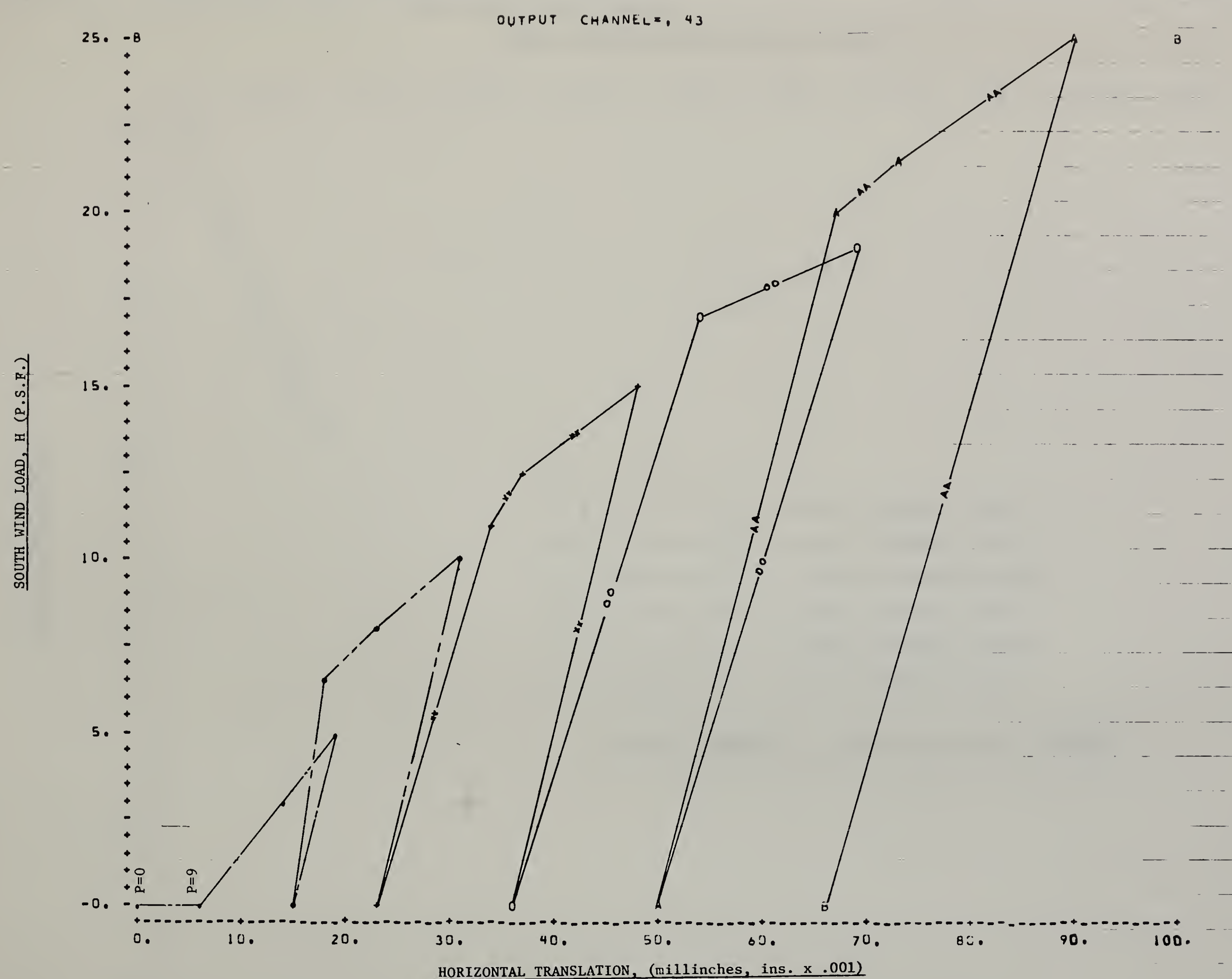


FIGURE NO. A.2 - TEST NO. 2, SOUTH WIND LOAD VS. TRANSLATION

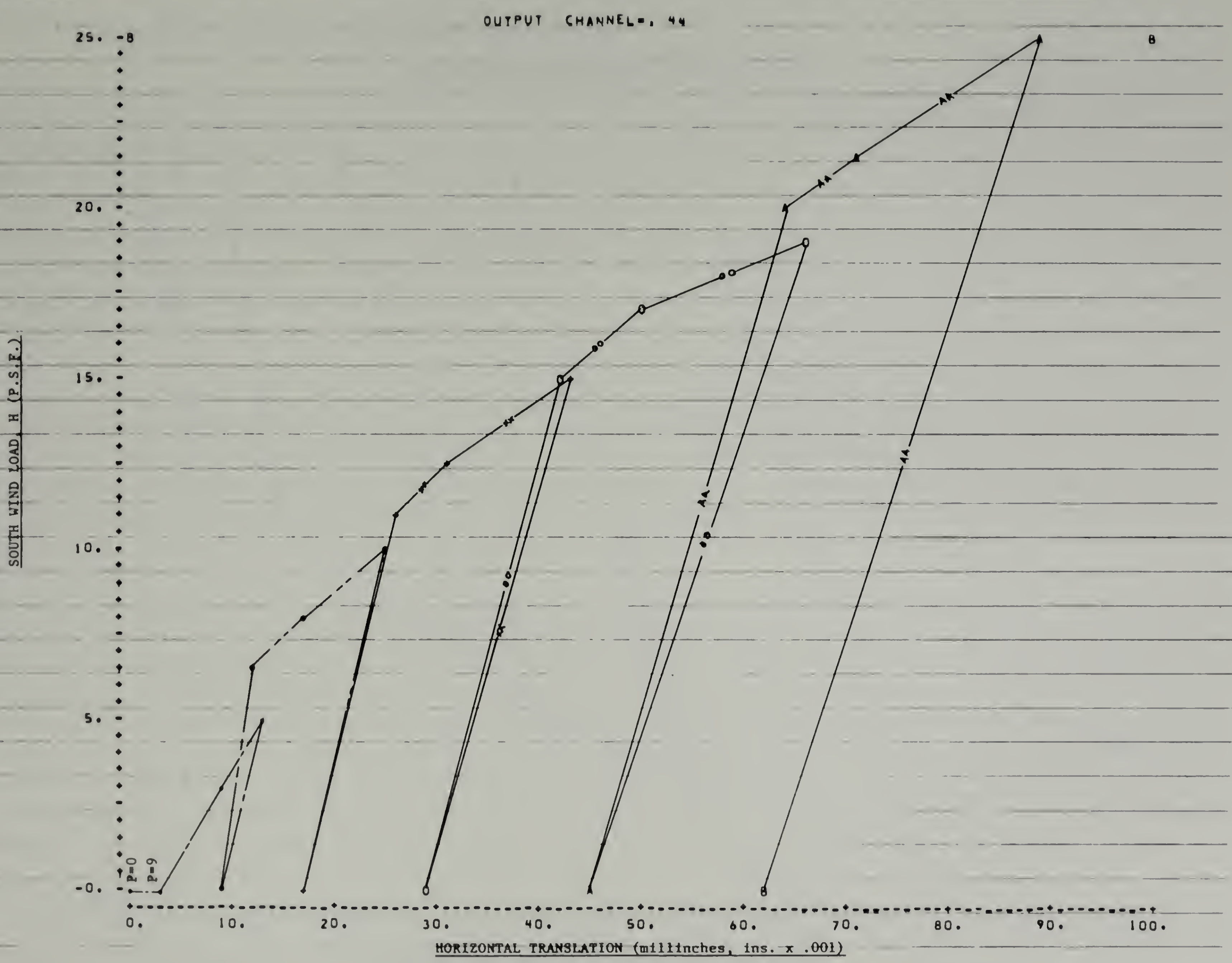


FIGURE NO. 8.3 - TEST NO. 2, SOUTH WIND LOAD VS. TRANSLATION

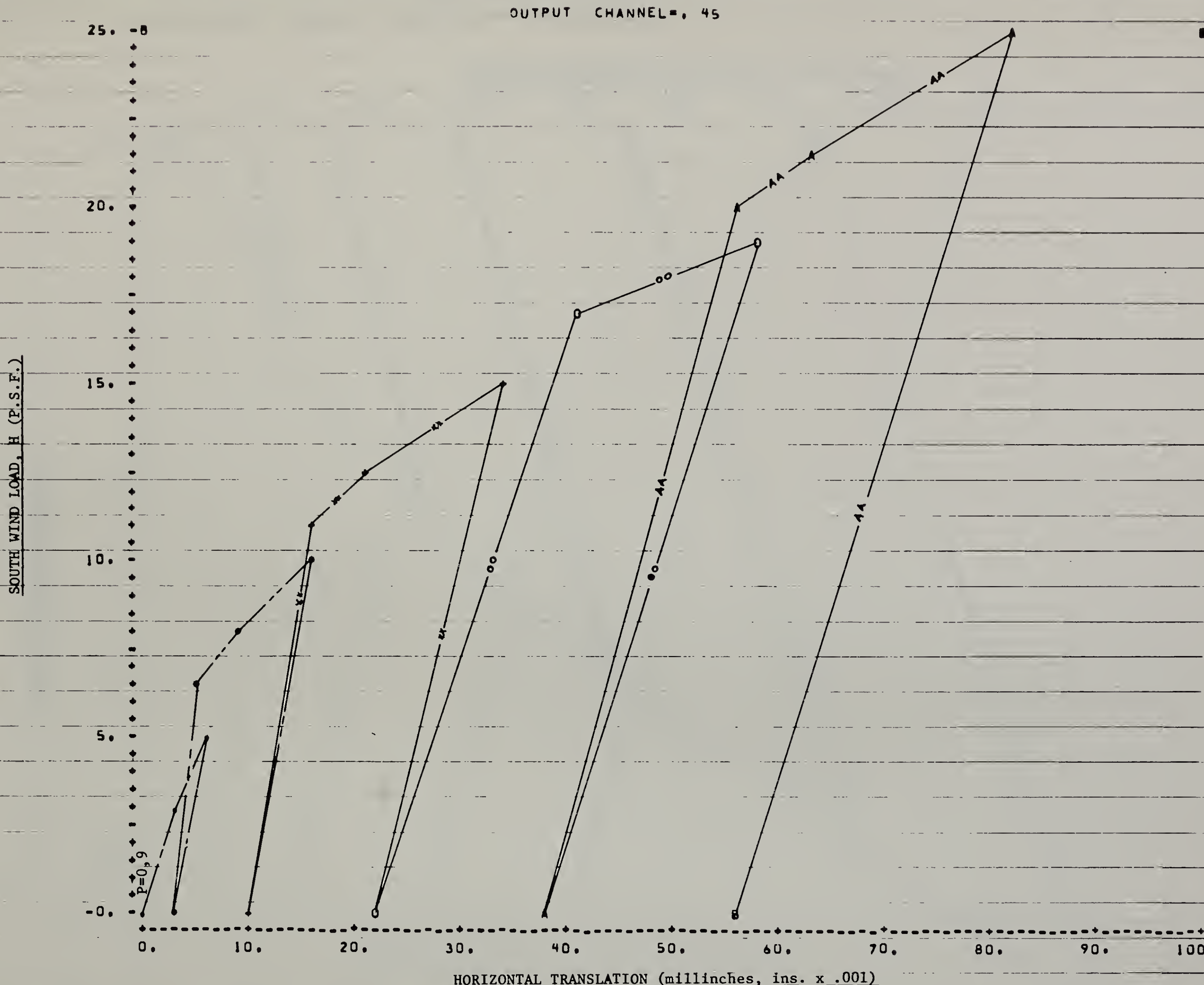


FIGURE No. A.4 - TEST NO. 2, SOUTH WIND LOAD VS. TRANSLATION

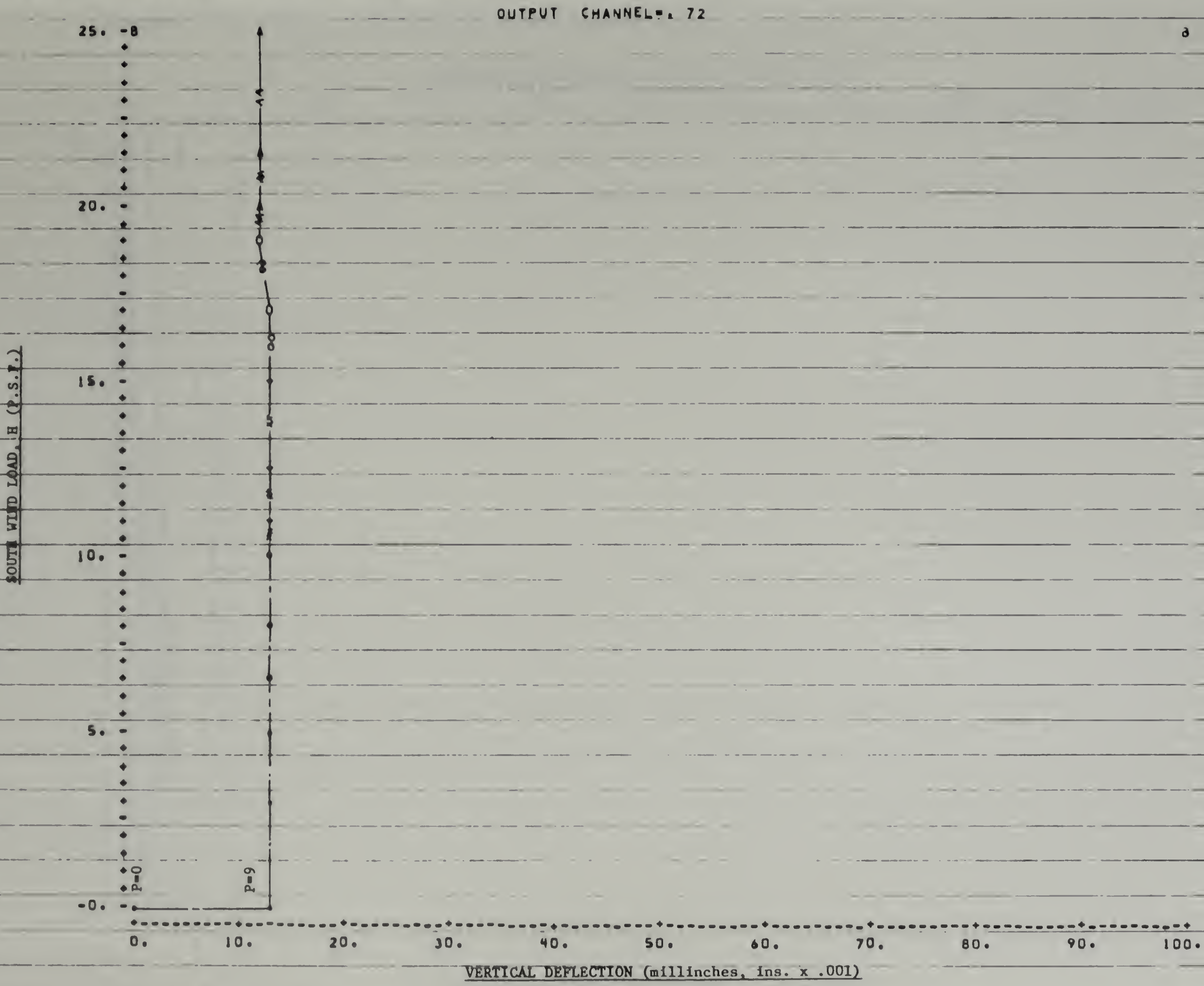


FIGURE NO. A.5 - TEST NO. 2, SOUTH WIND LOAD VS. BEAM DEFLECTION

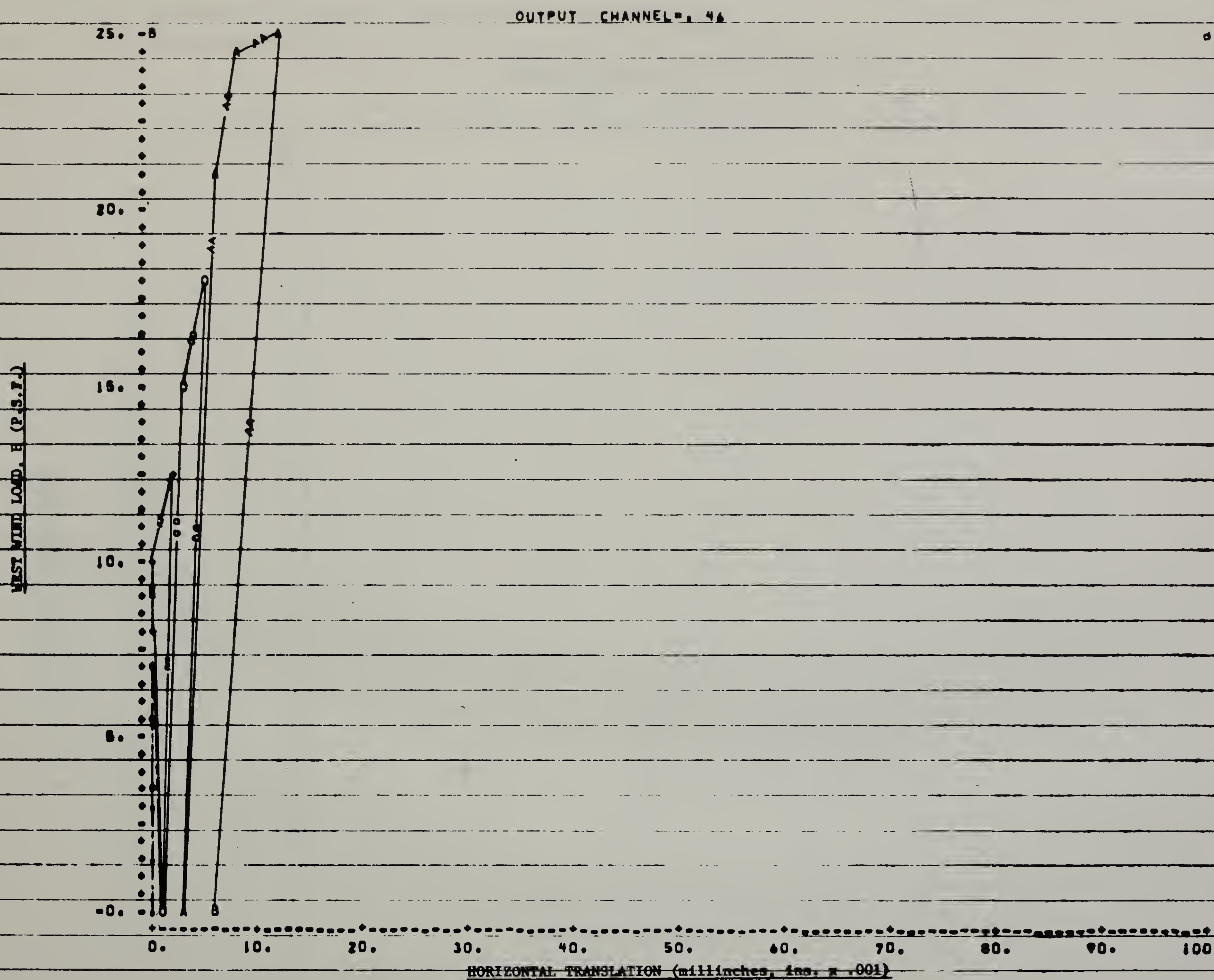


FIGURE NO. A.6 - TEST NO. 3, WEST WIND LOAD VS. TRANSLATION

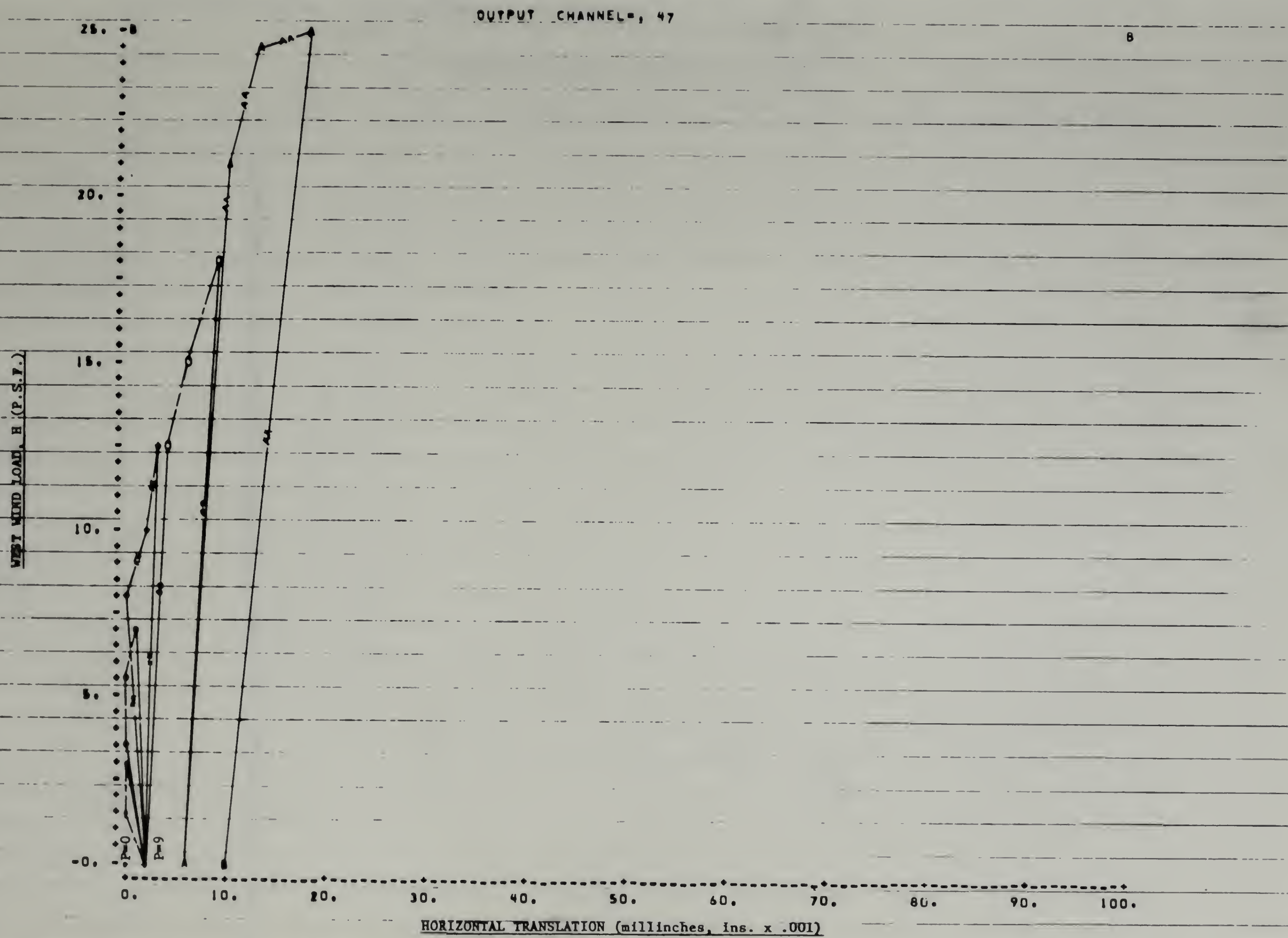


FIGURE NO. A.7 - TEST NO. 3, WEST WIND LOAD VS. TRANSLATION

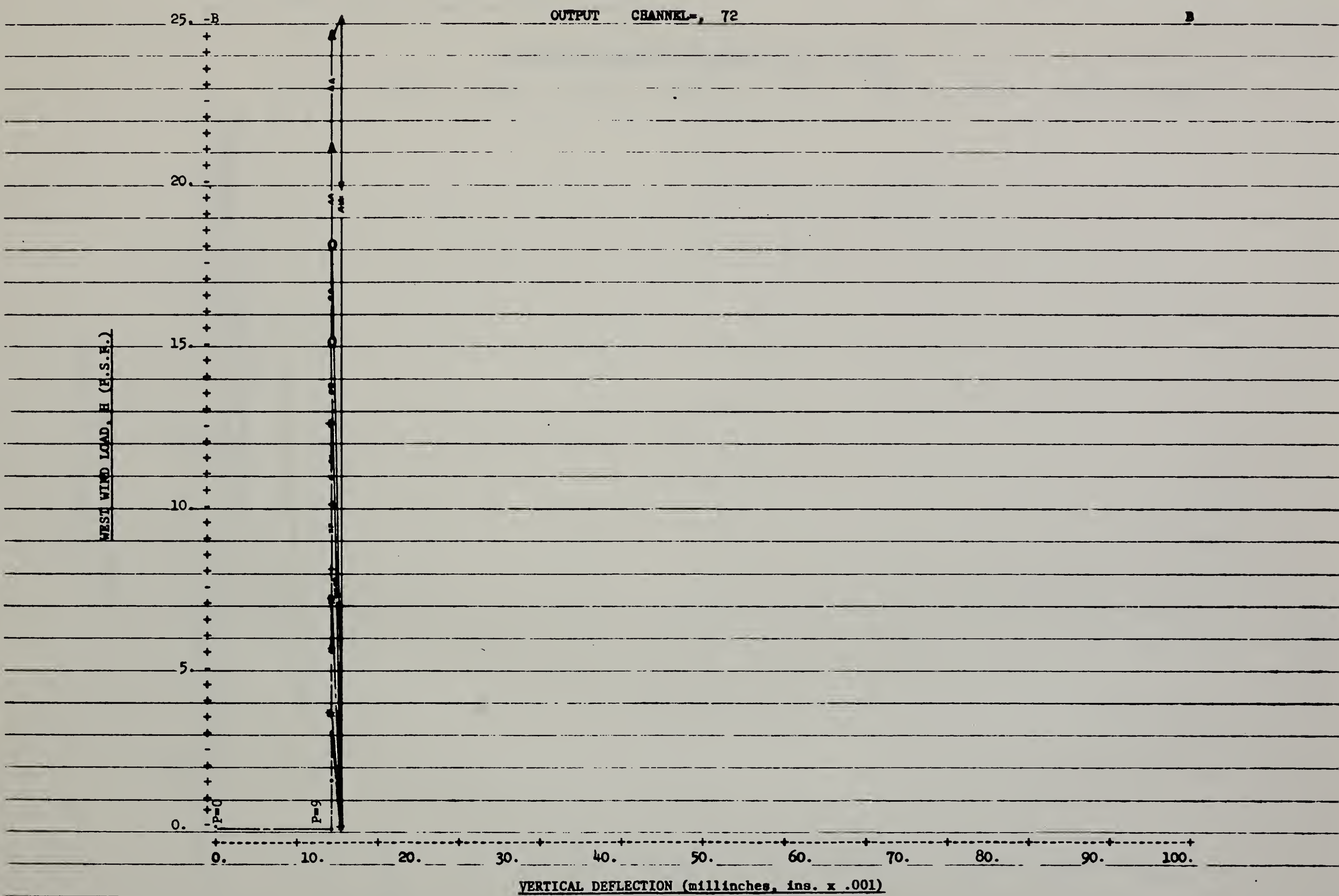


FIGURE NO. A.8 - TEST NO. 3, WEST WIND LOAD VS. BEAM DEFLECTION

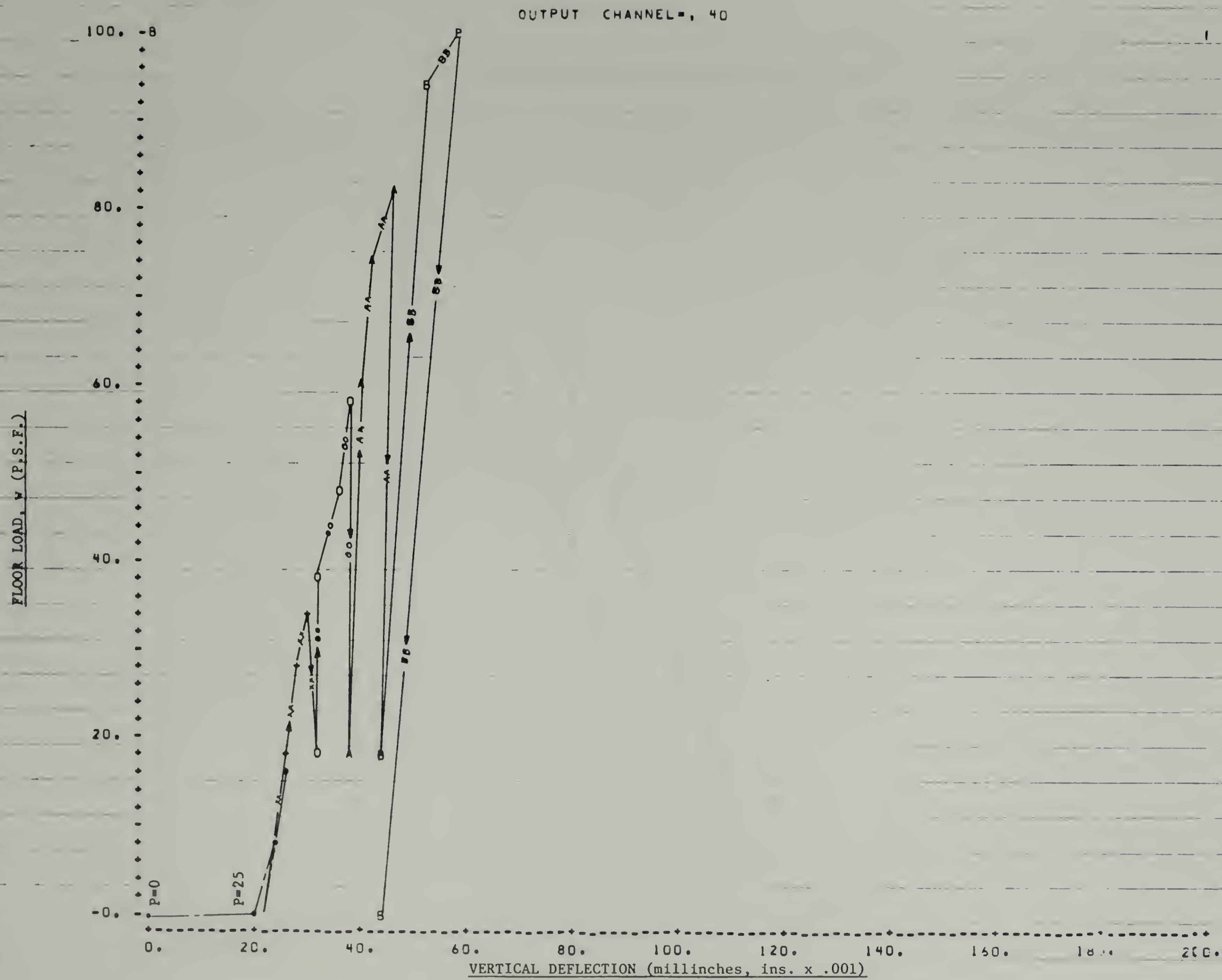


FIGURE NO. A.9 - TEST NO. 4, FLOOR LOAD VS. BEAM DEFLECTION

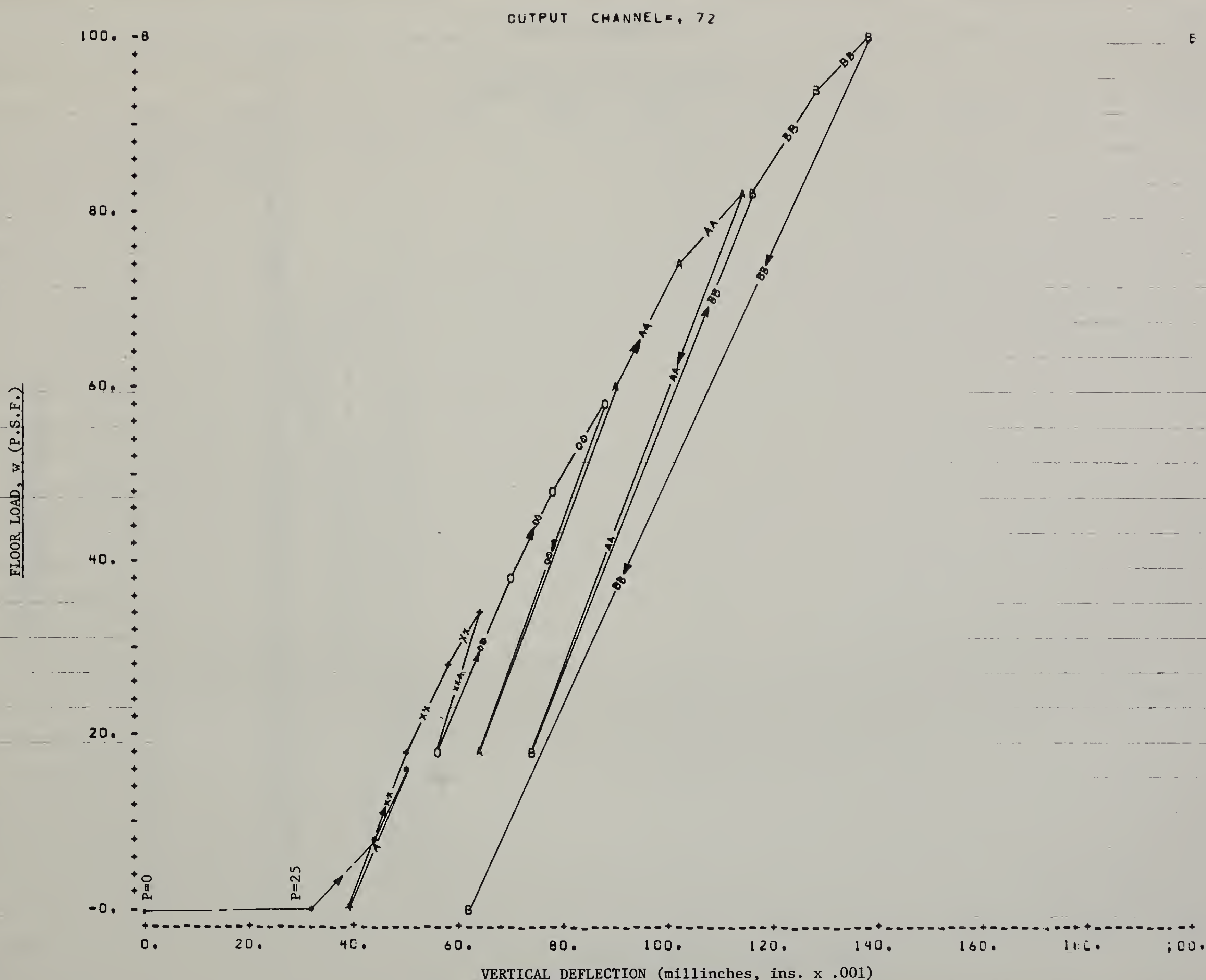


FIGURE NO. A.10 - TEST NO. 4, FLOOR LOAD VS. BEAM DEFLECTION

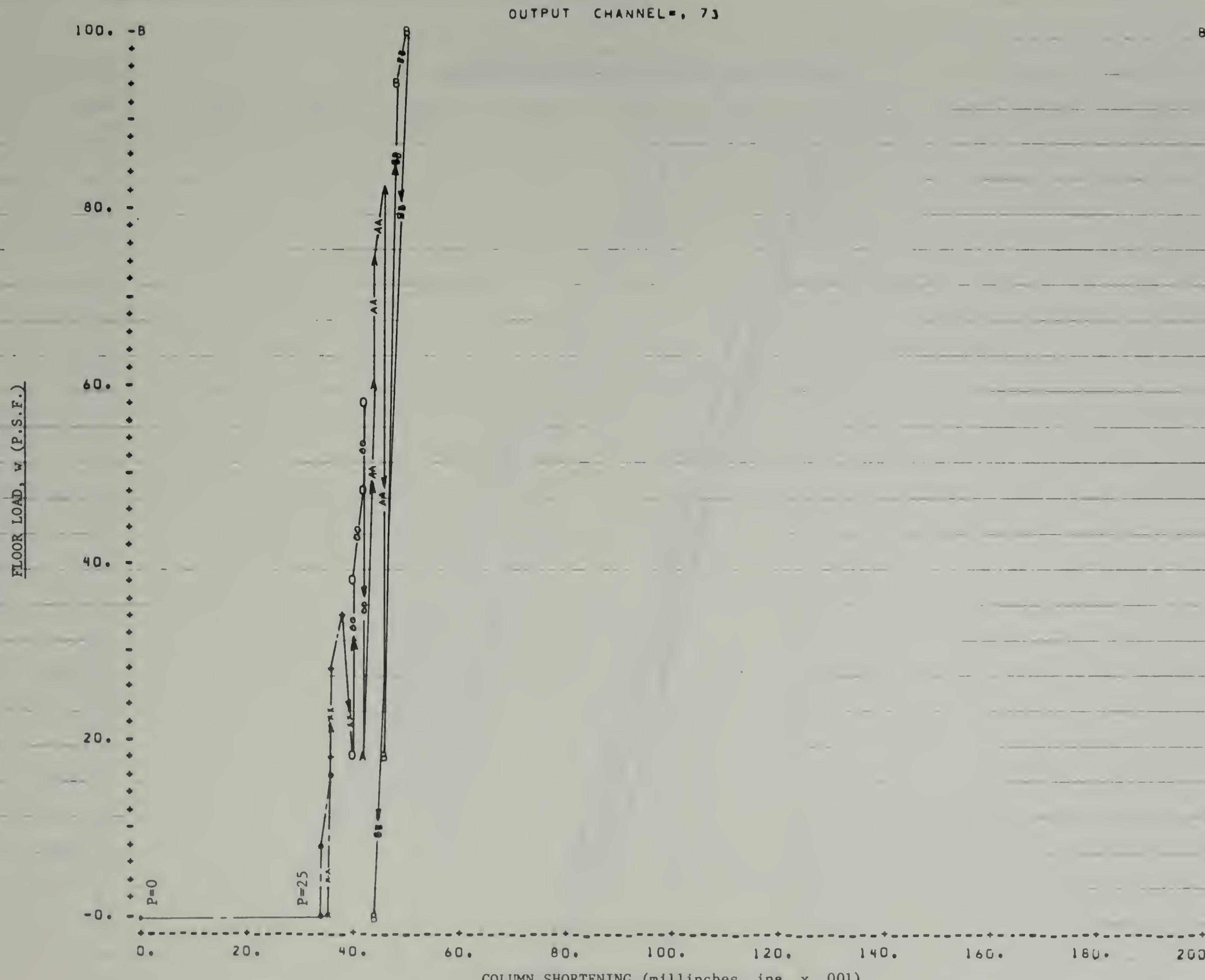


FIGURE NO. A.11 - TEST NO. 4, FLOOR LOAD VS. COLUMN SHORTENING

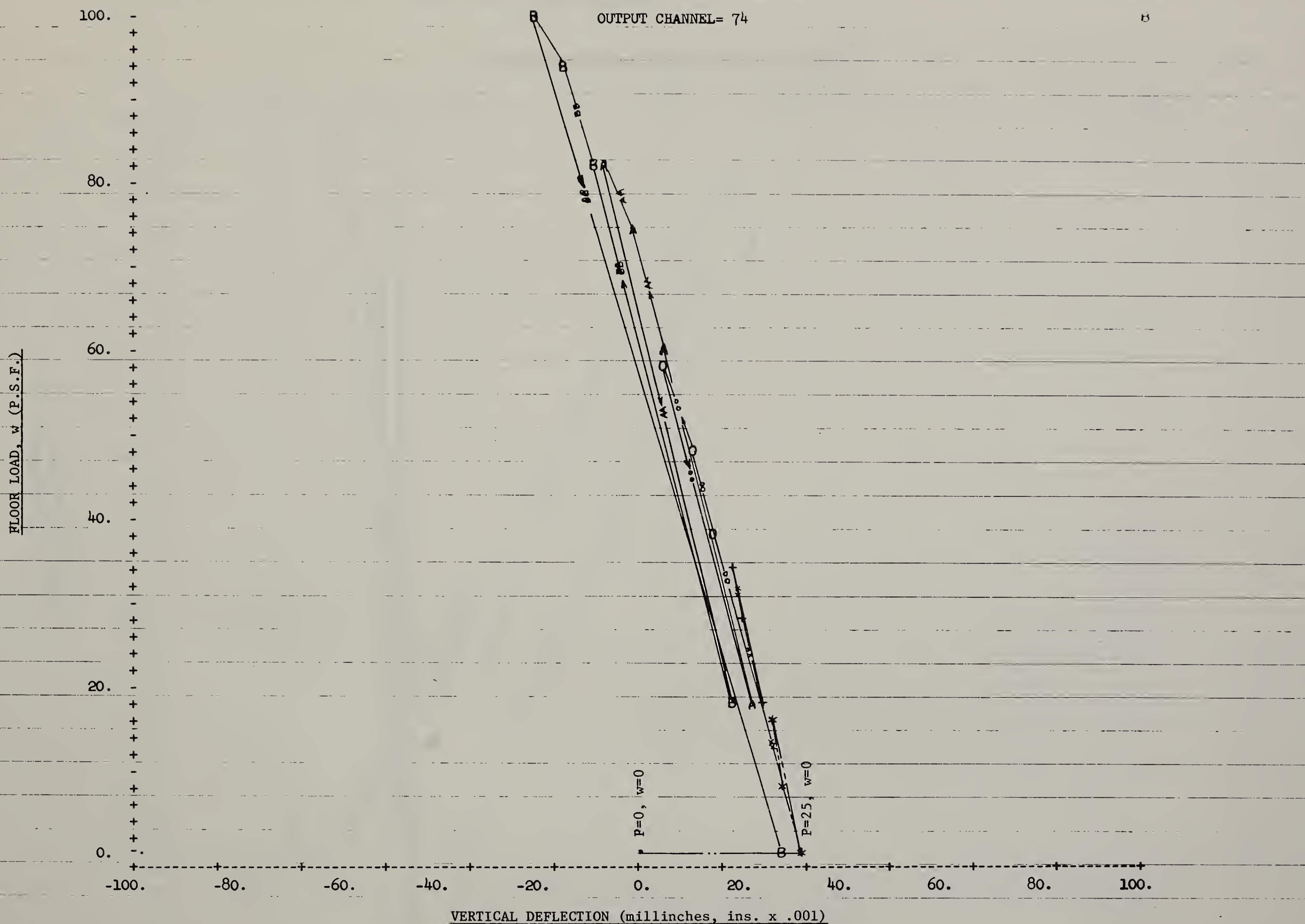


FIGURE NO. A.12 - TEST NO. 4, FLOOR LOAD VS. BEAM DEFLECTION

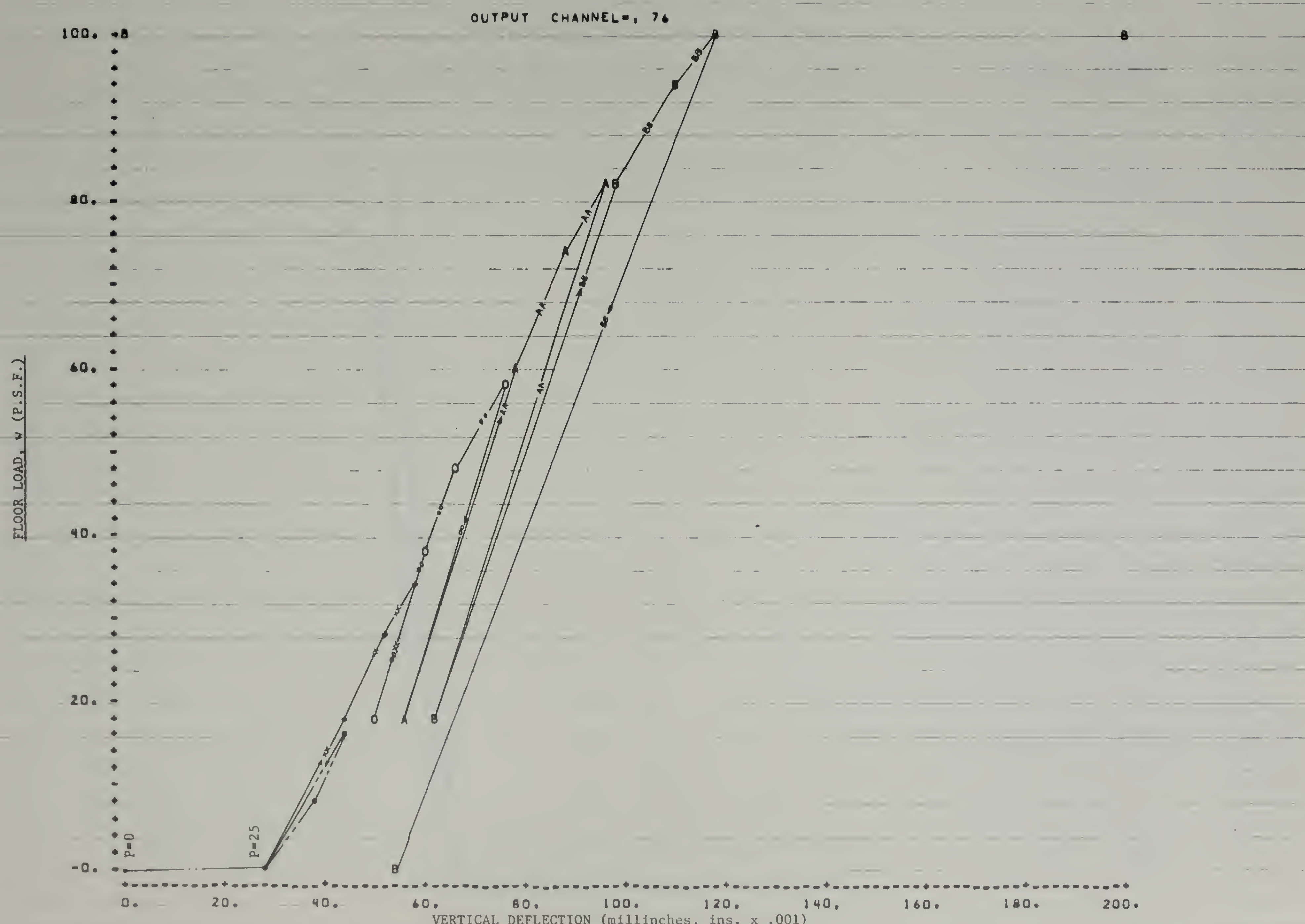


FIGURE NO. A.13 - TEST NO. 4, FLOOR LOAD VS. SLAB DEFLECTION

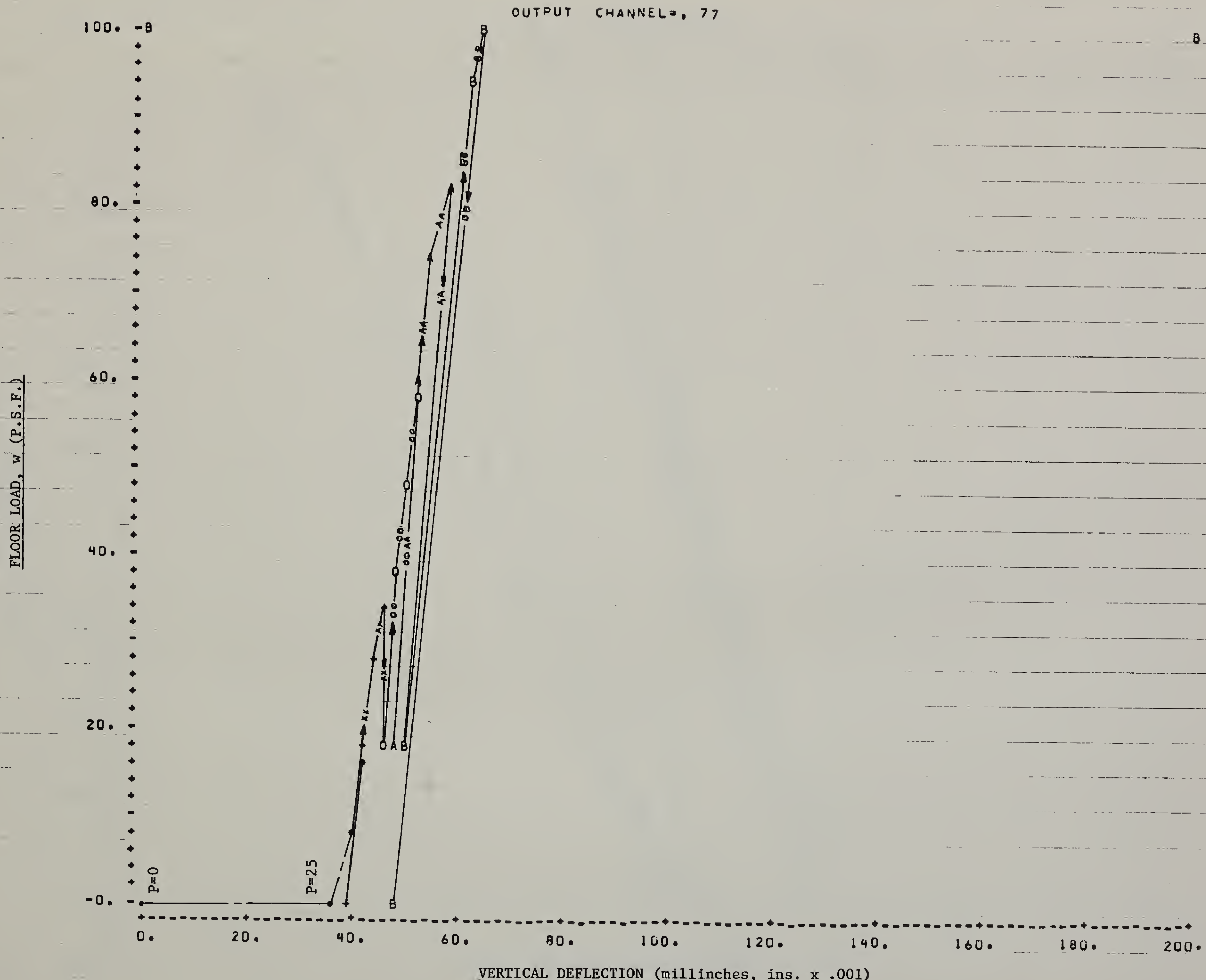


FIGURE NO. A.14 - TEST NO. 4, FLOOR LOAD VS. BEAM DEFLECTION

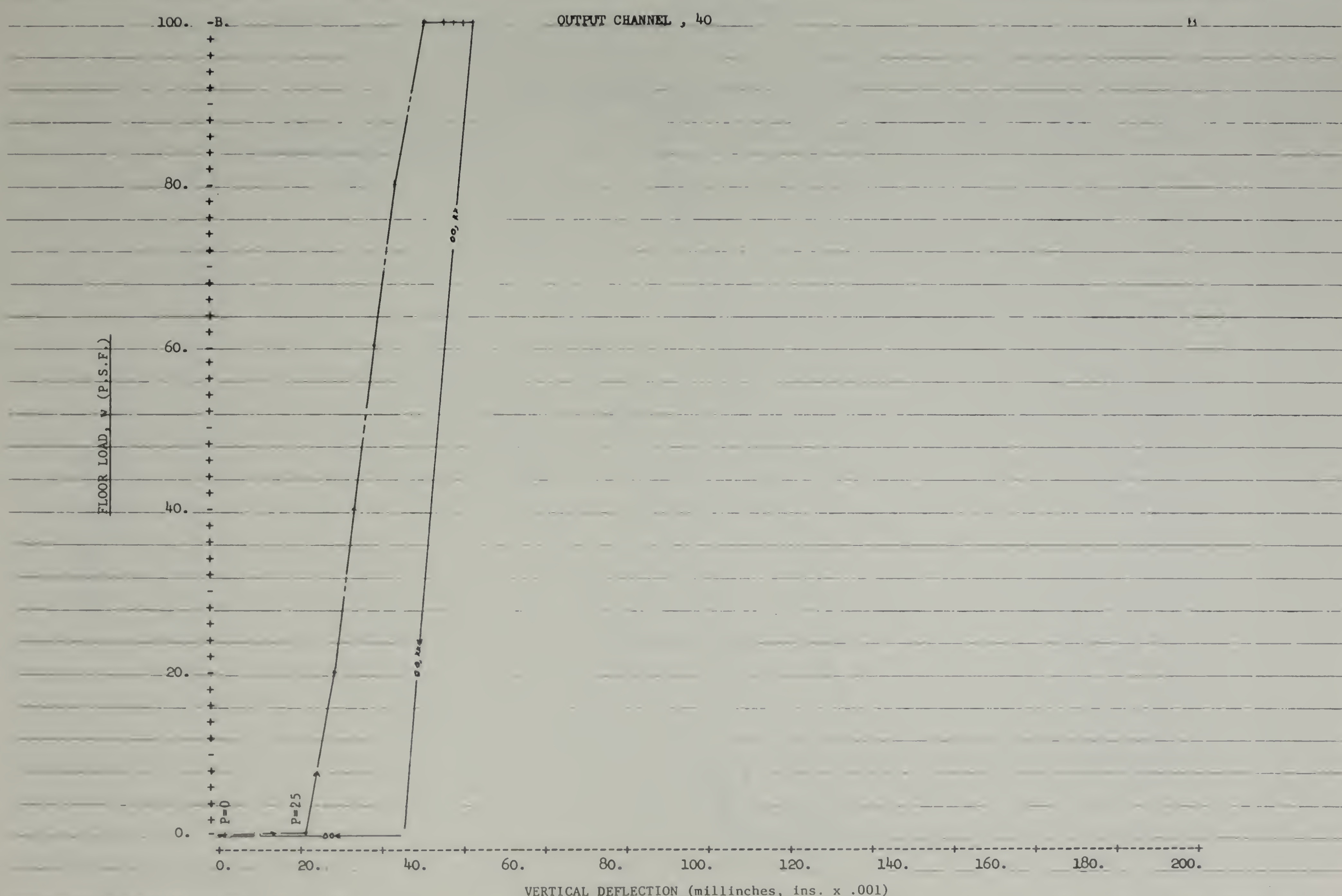


FIGURE NO. A.15 - TEST NO. 5, SUSTAINED FLOOR LOAD VS. BEAM DEFLECTION

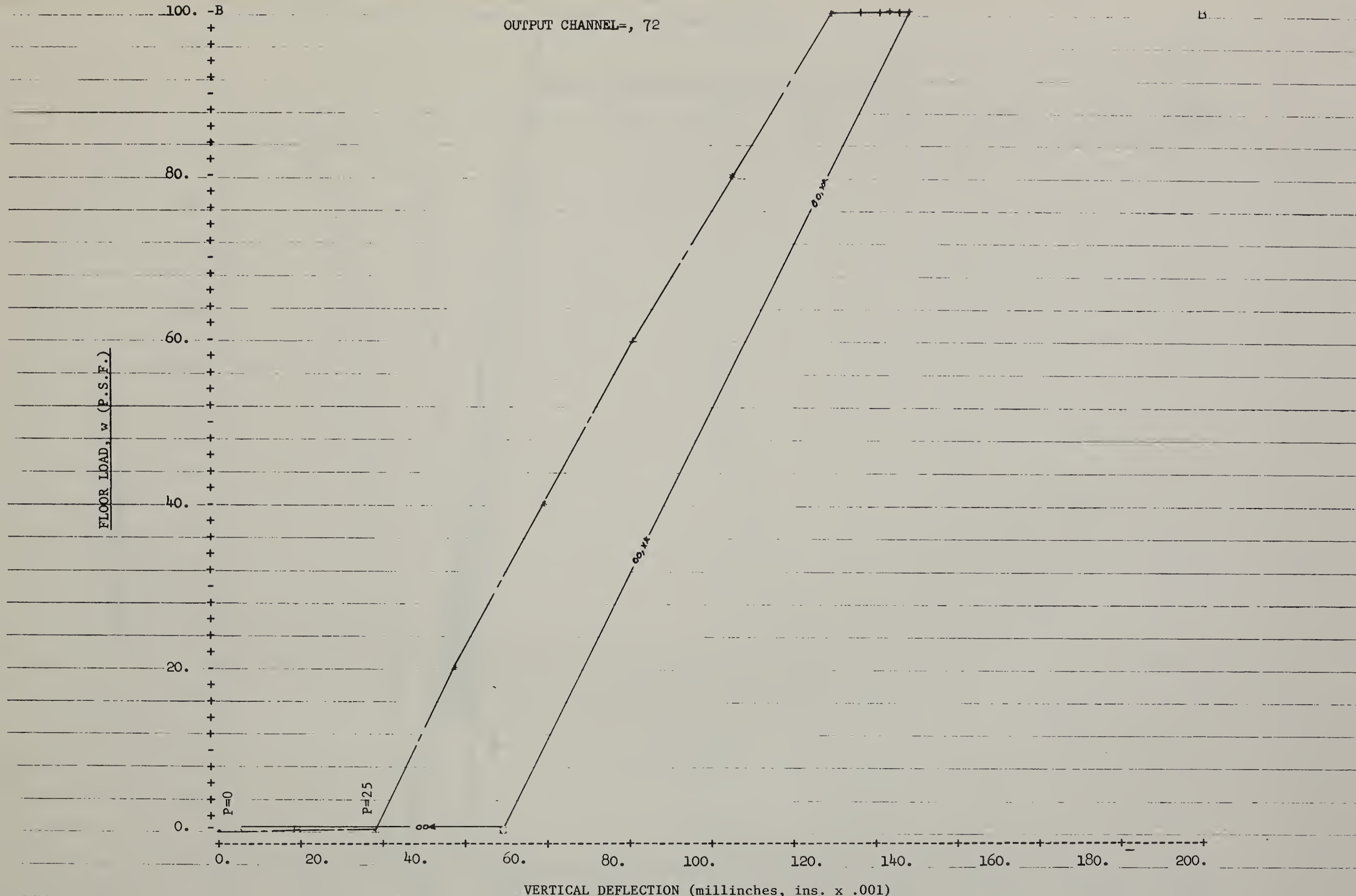


FIGURE NO. A.16 - TEST NO. 5, SUSTAINED FLOOR LOAD VS. BEAM DEFLECTION

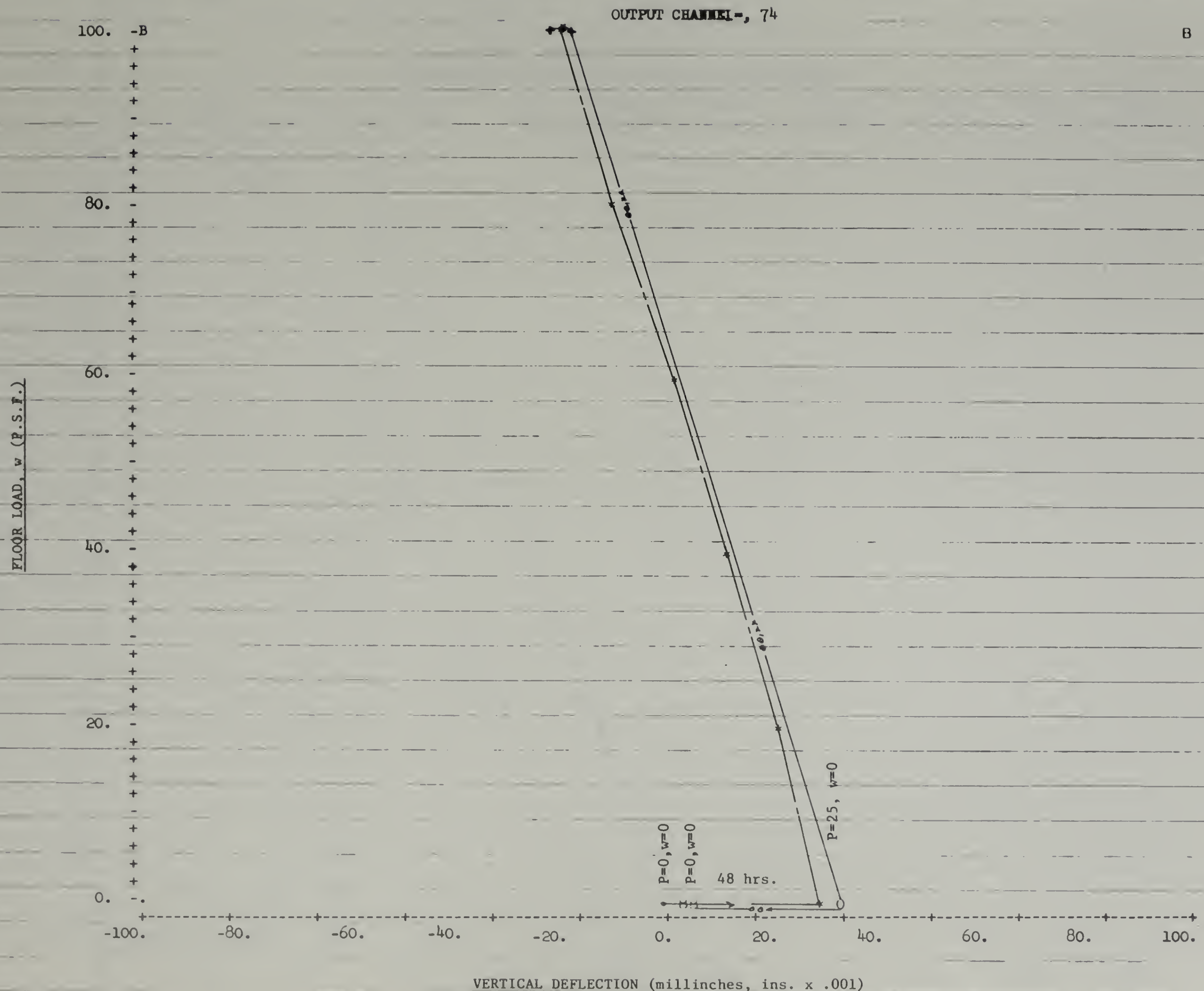


FIGURE NO. A.17 - TEST NO. 5, SUSTAINED FLOOR LOAD VS. BEAM DEFLECTION

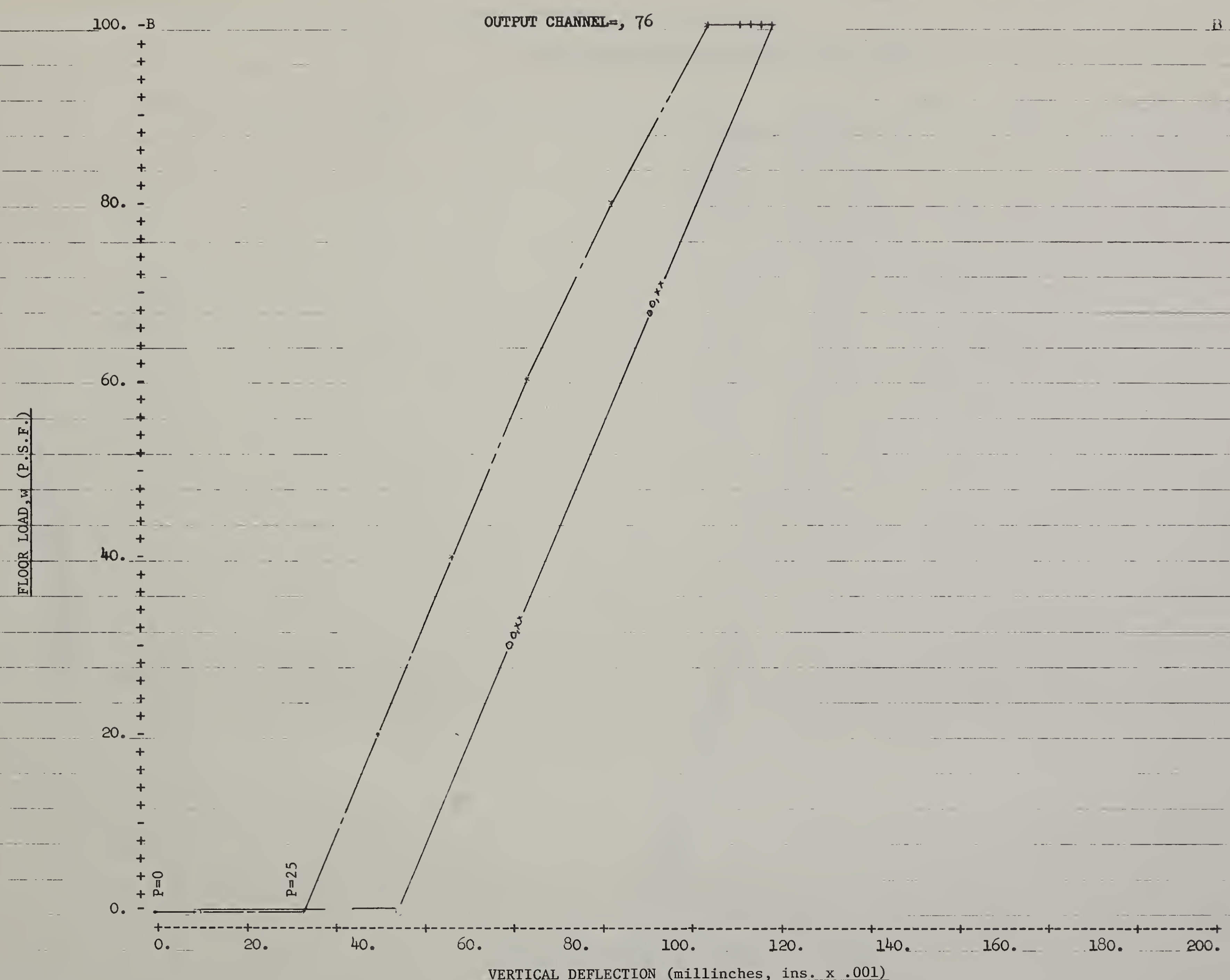


FIGURE NO. A.18 _ TEST NO. 5, SUSTAINED FLOOR LOAD VS. SLAB DEFLECTION

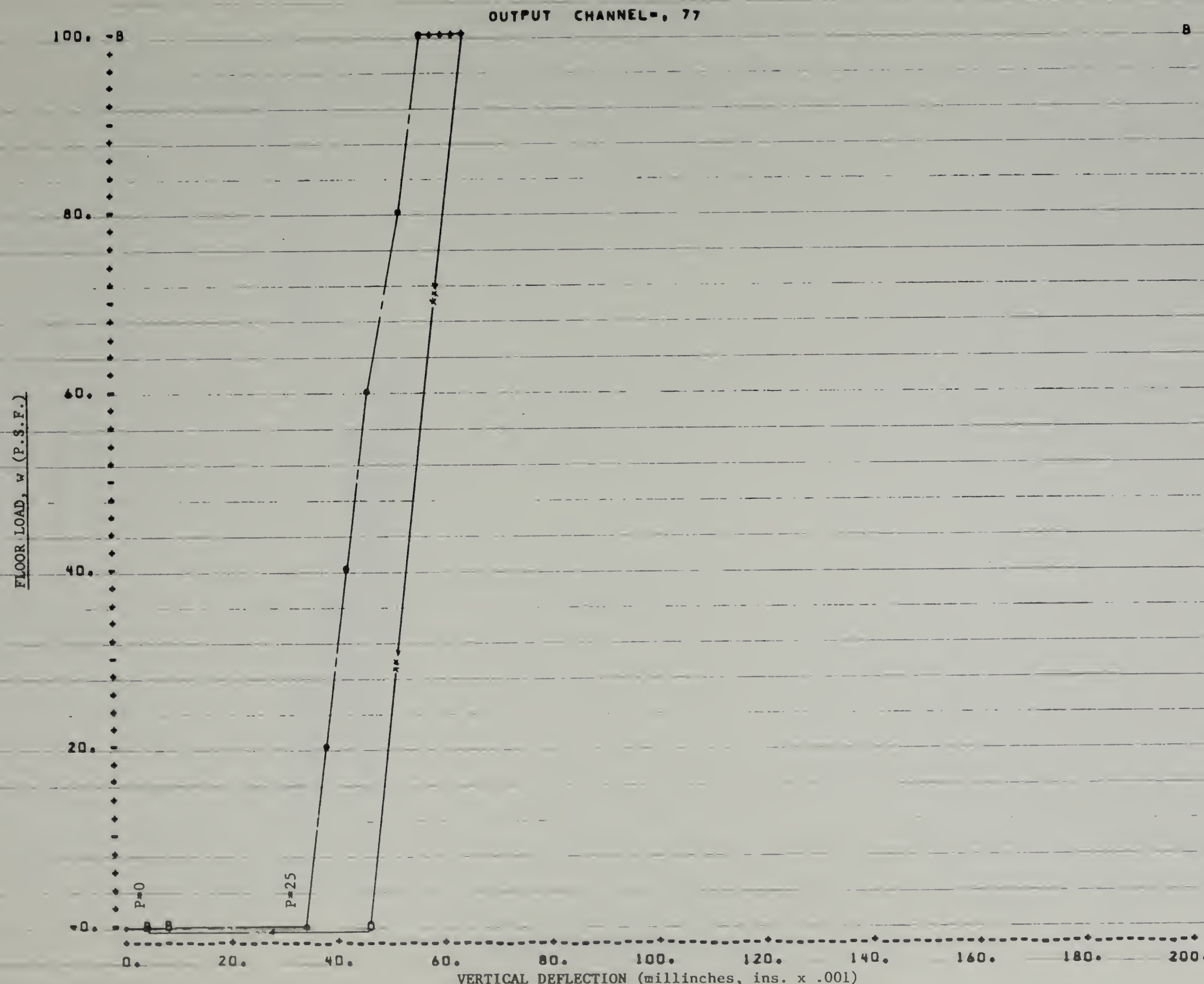


FIGURE NO. A.19 - TEST NO. 5, SUSTAINED FLOOR LOAD VS. BEAM DEFLECTION

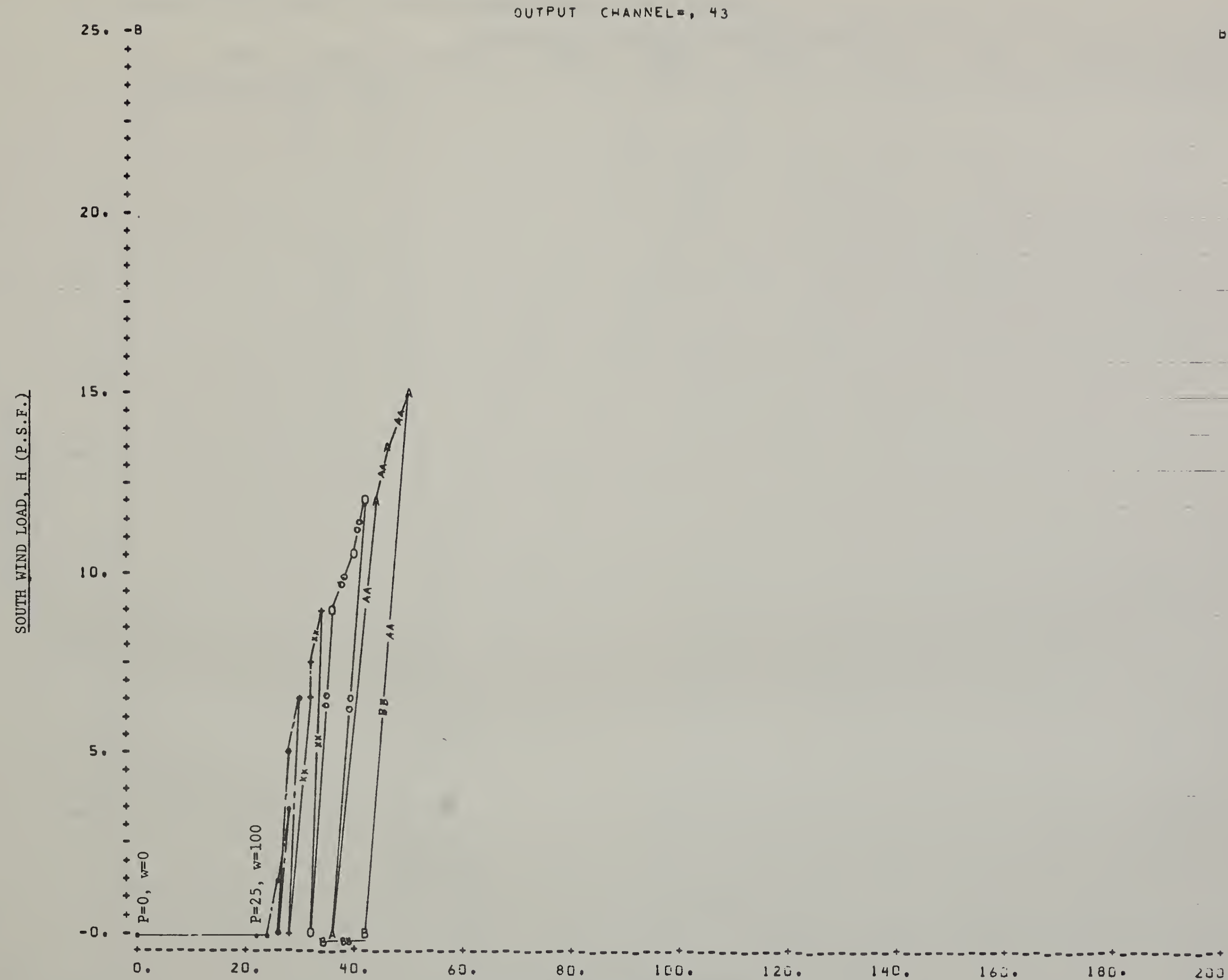


FIGURE NO. A.20 - TEST NO. 7, SOUTH WIND LOAD VS. TRANSLATION

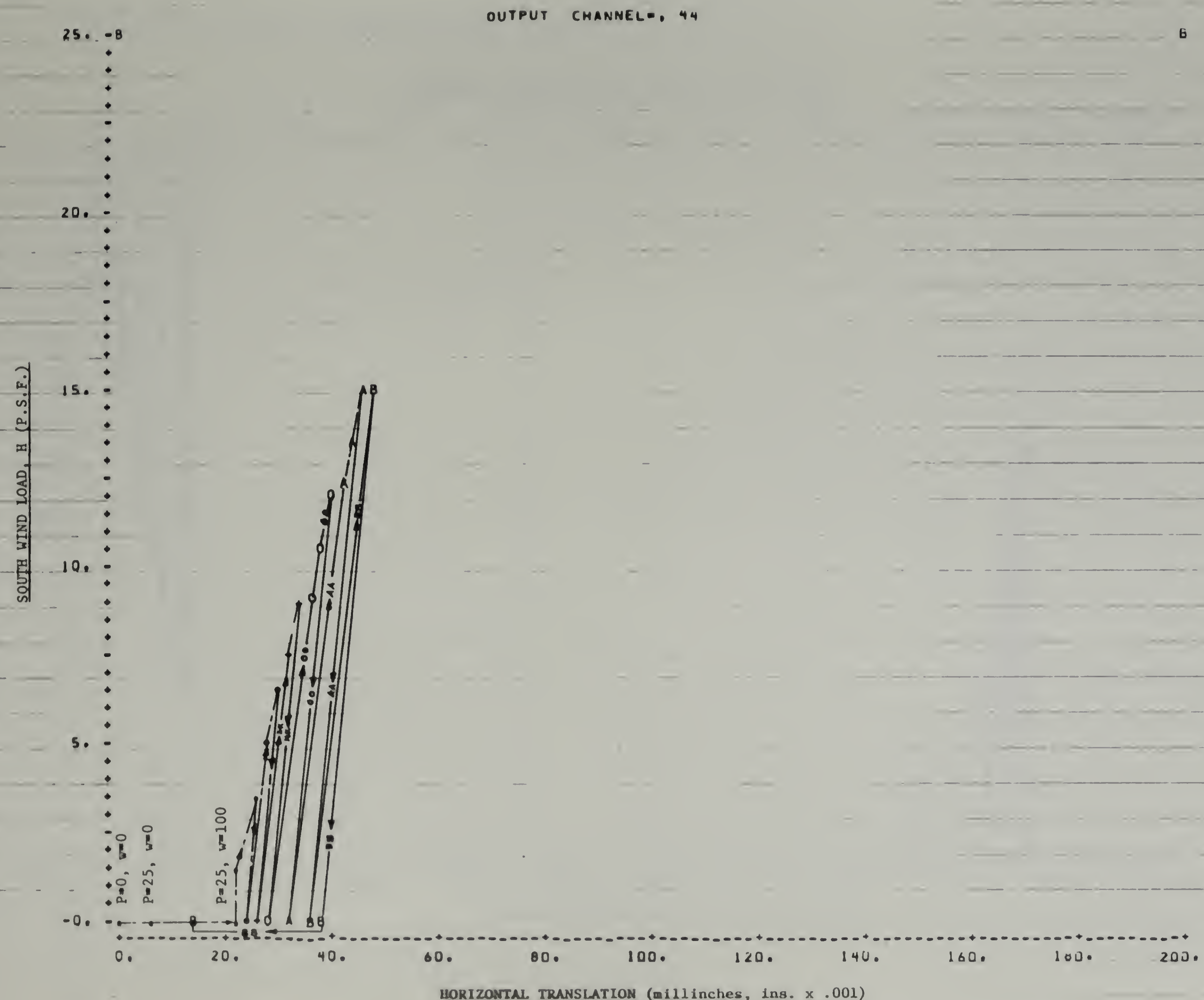


FIGURE NO. A.21 - TEST NO. 7, SOUTH WIND LOAD VS. TRANSLATION

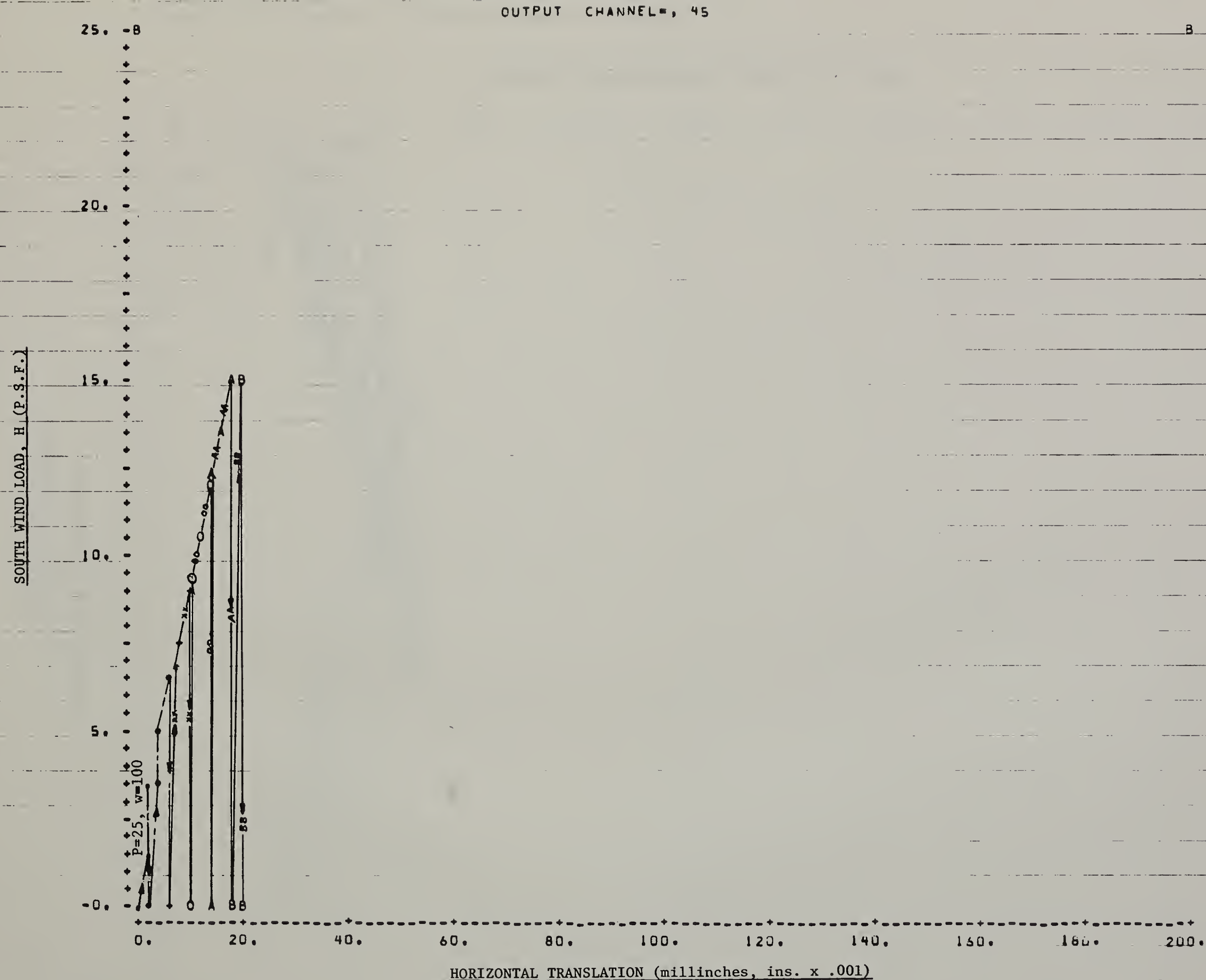


FIGURE NO. A.22 - TEST NO. 7, SOUTH WIND LOAD VS. TRANSLATION

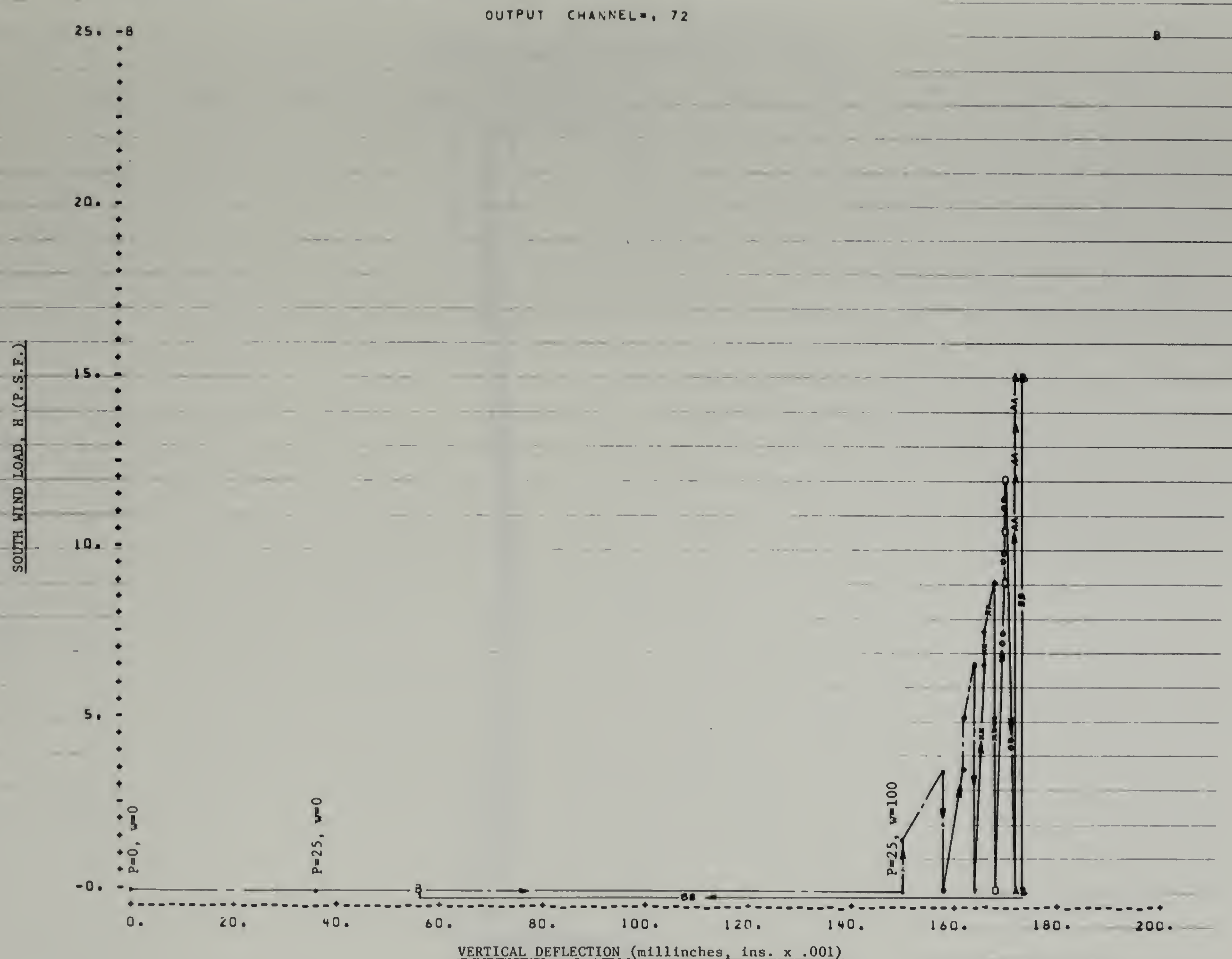


FIGURE NO. A.23 - TEST NO. 7, SOUTH WIND LOAD VS. BEAM DEFLECTION

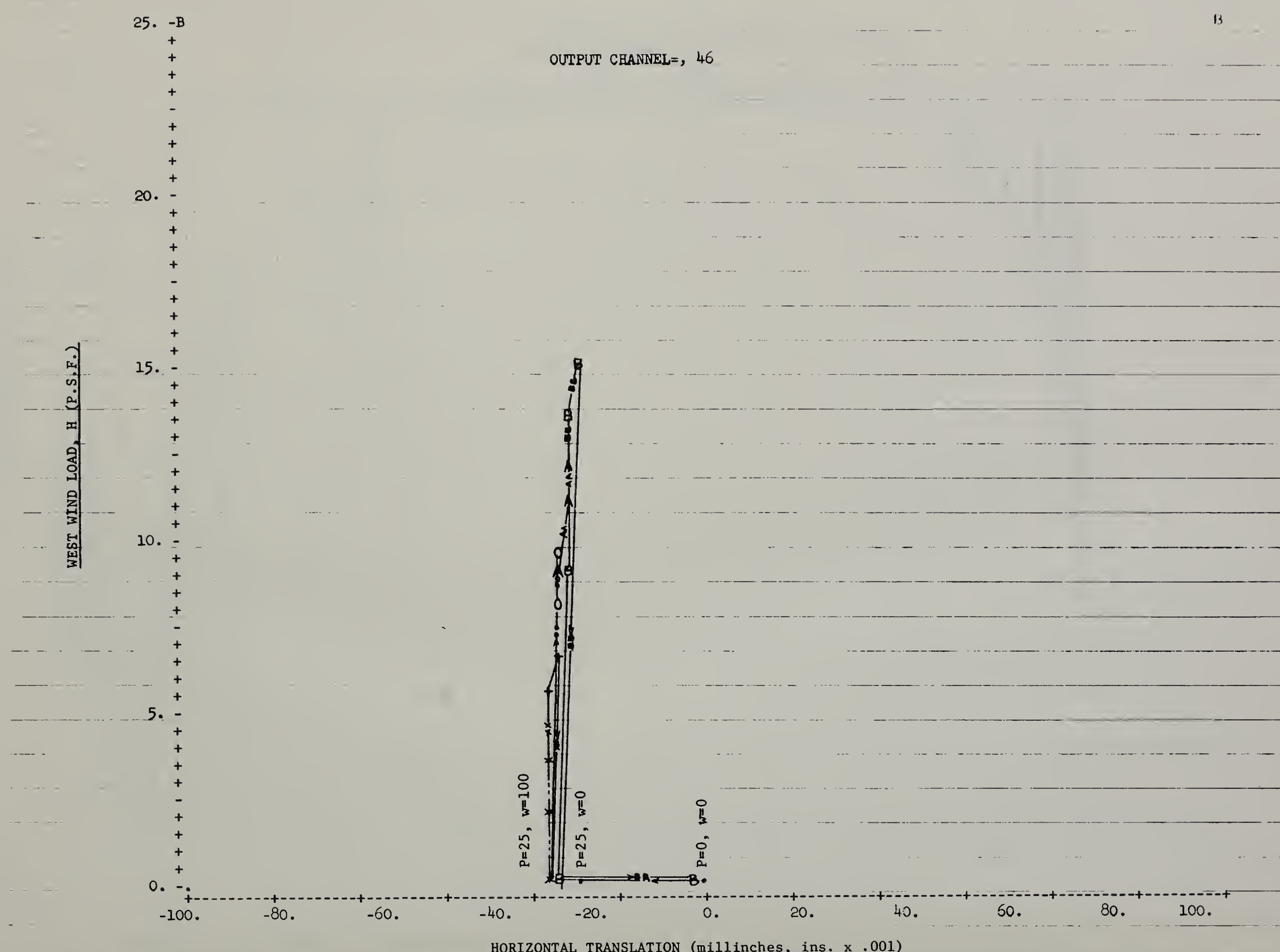


FIGURE NO. A.24 - TEST NO. 8, WEST WIND LOAD VS. TRANSLATION

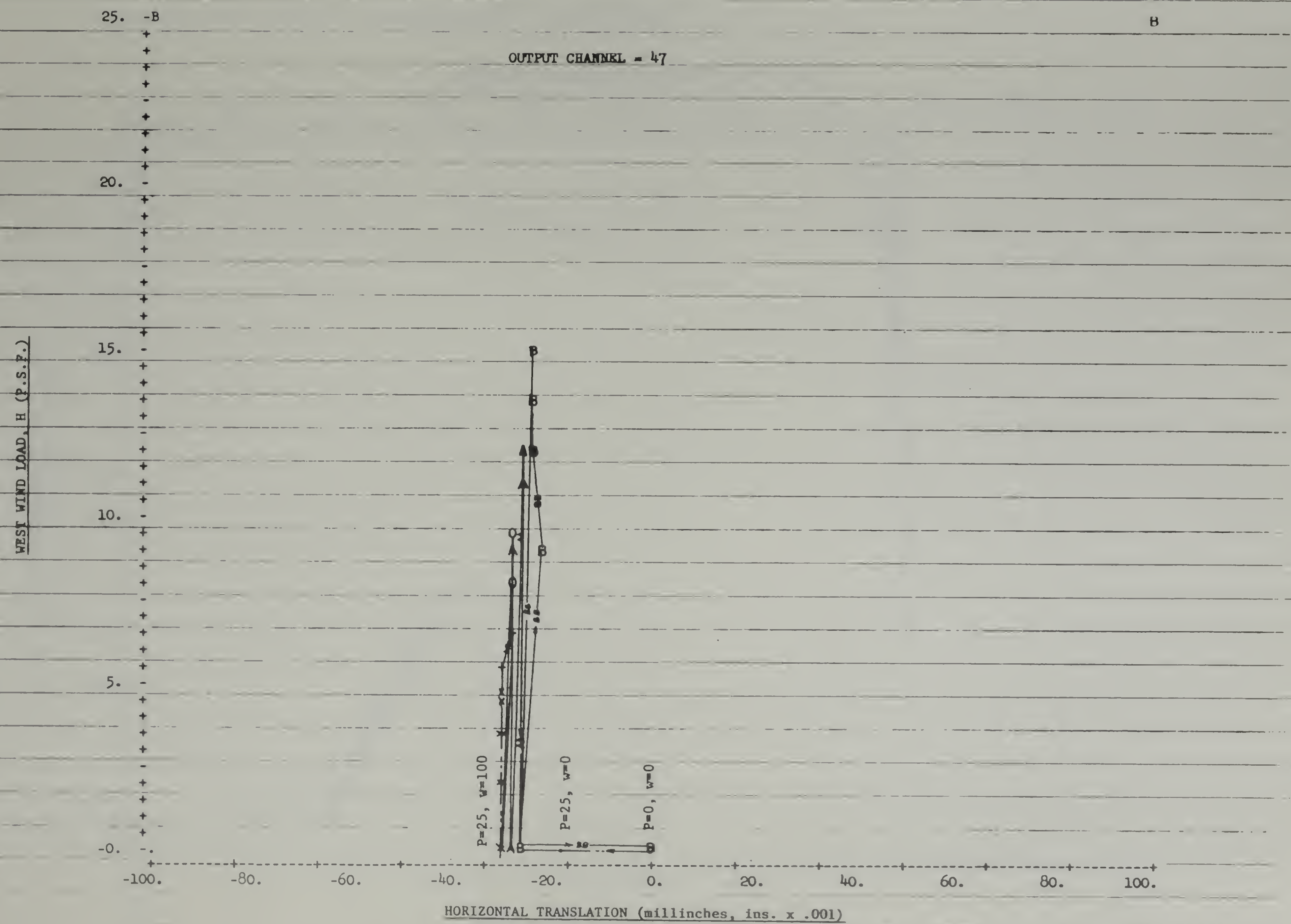


FIGURE NO. A.25 - TEST NO. 8, WEST WIND LOAD VS. TRANSLATION

OUTPUT CHANNEL=, 72

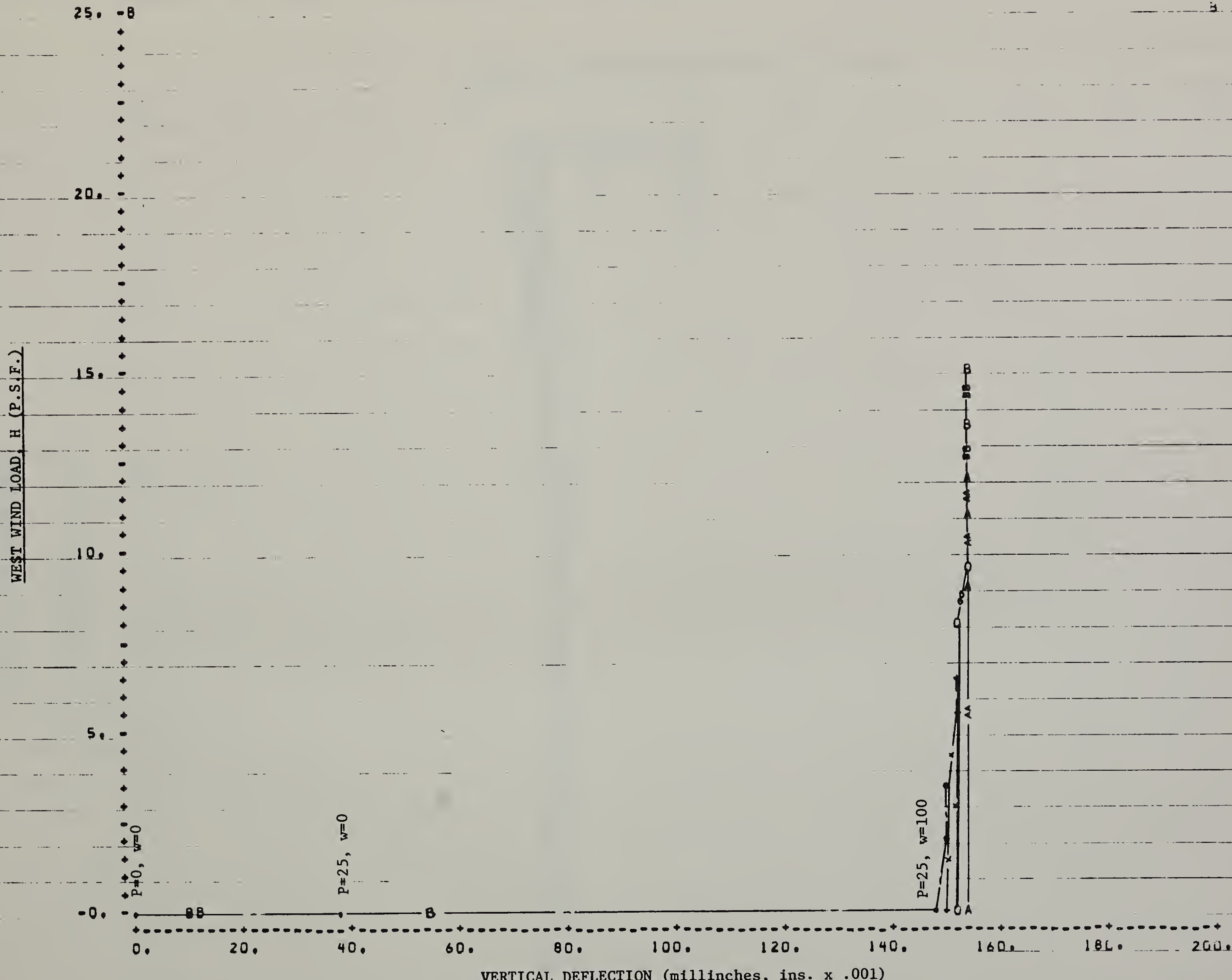


FIGURE NO. A.26 - TEST NO. 8 - WEST WIND LOAD VS. BEAM DEFLECTION

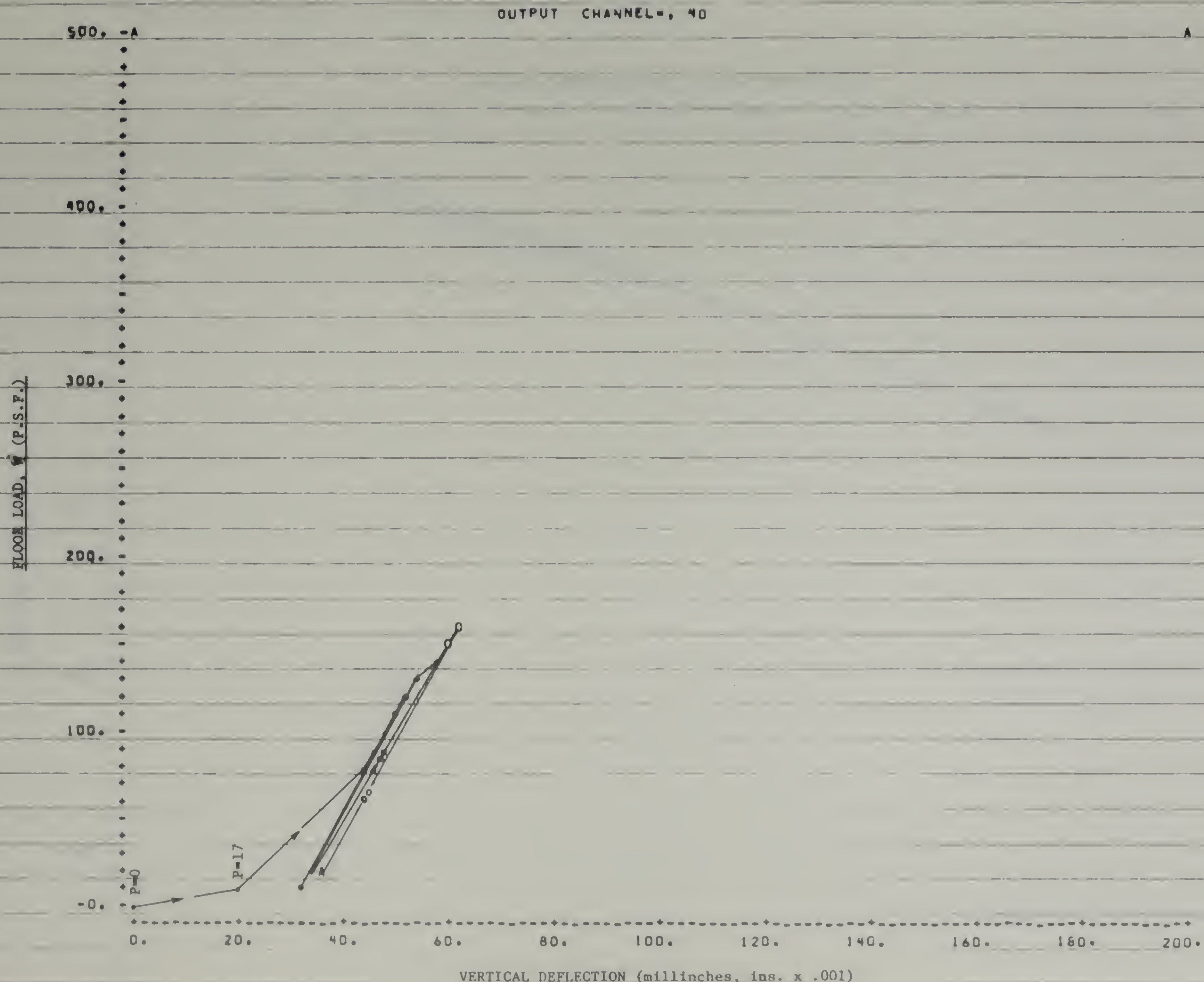


FIGURE NO. A.27 - TEST NO. 9, MAJOR FLOOR LOAD VS. BEAM DEFLECTION

OUTPUT CHANNEL=, 72

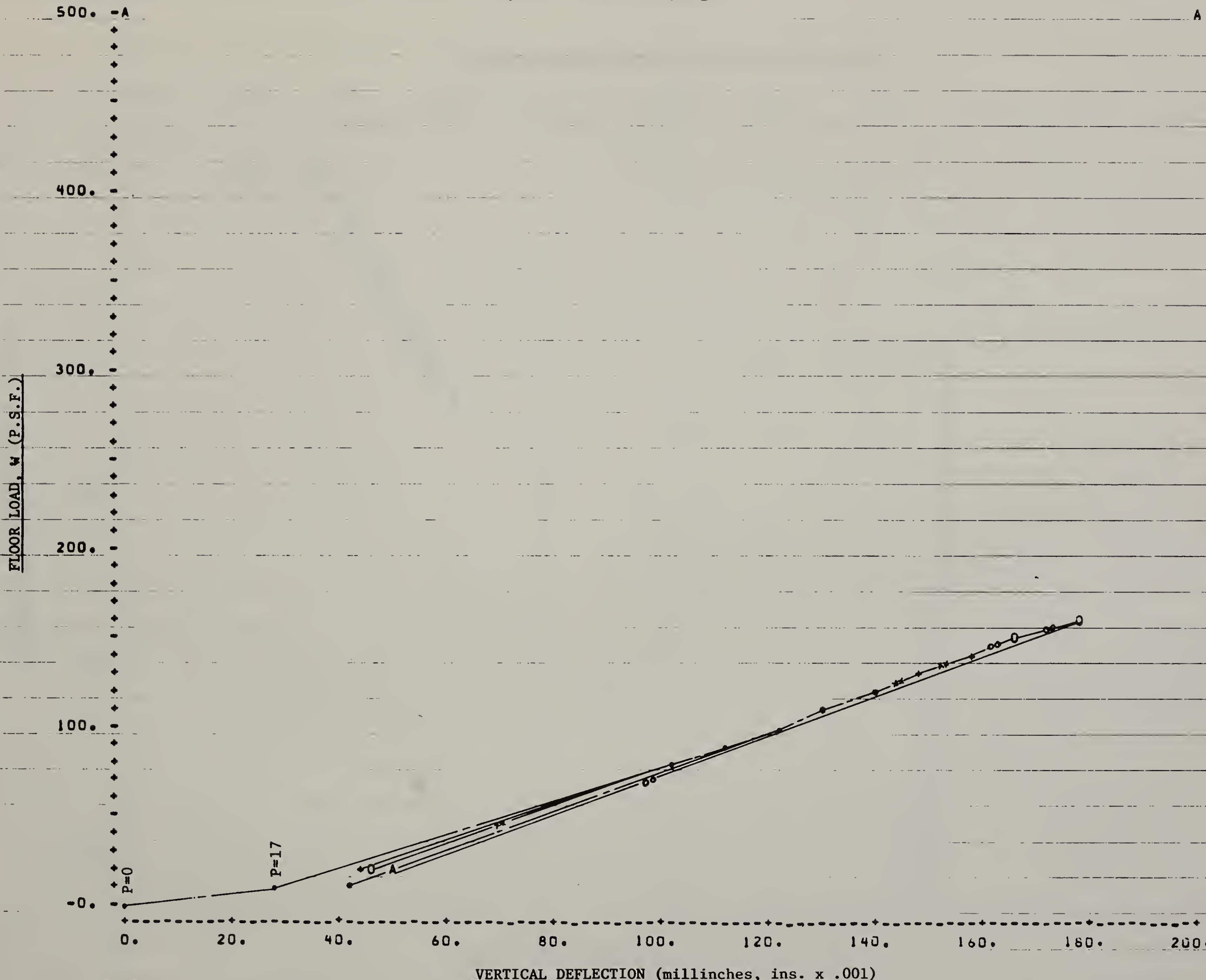


FIGURE NO. A.28 - TEST NO. 9, MAJOR FLOOR LOAD VS. BEAM DEFLECTION

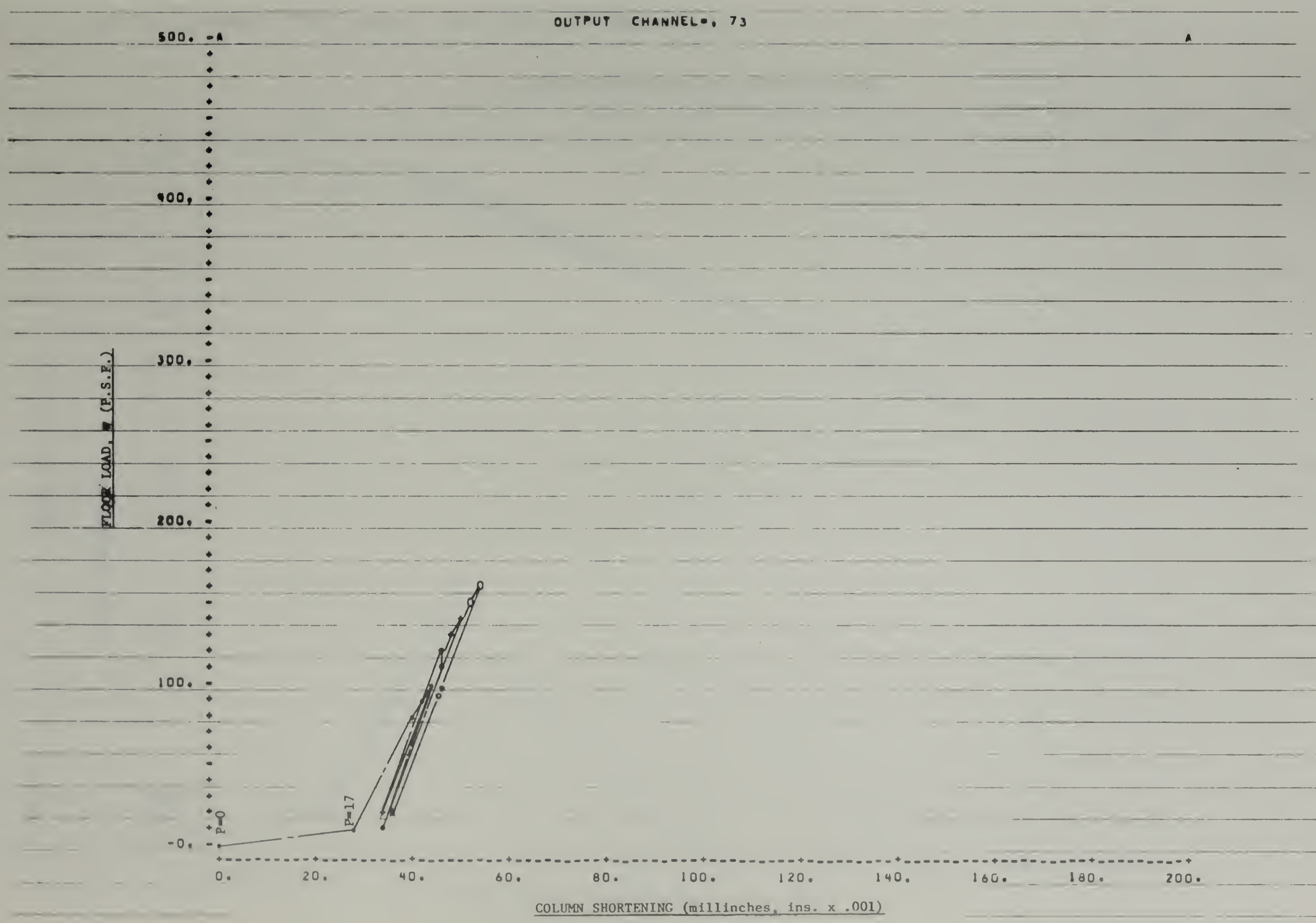


FIGURE NO. A.29 - TEST NO. 9, MAJOR FLOOR LOAD VS. COLUMN SHORTENING

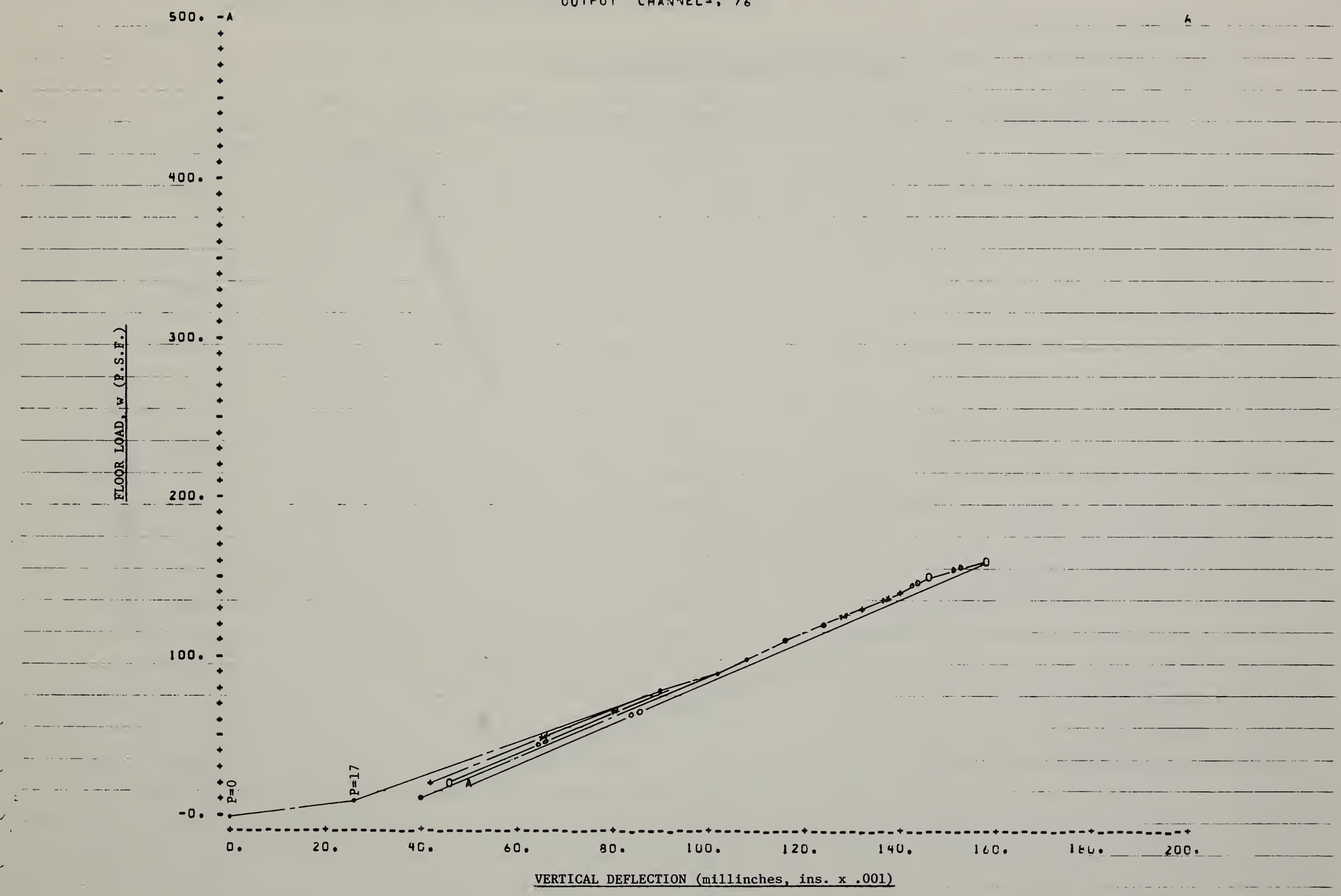


FIGURE NO. A.30 - TEST NO. 9, MAJOR FLOOR LOAD VS. SLAB DEFLECTION

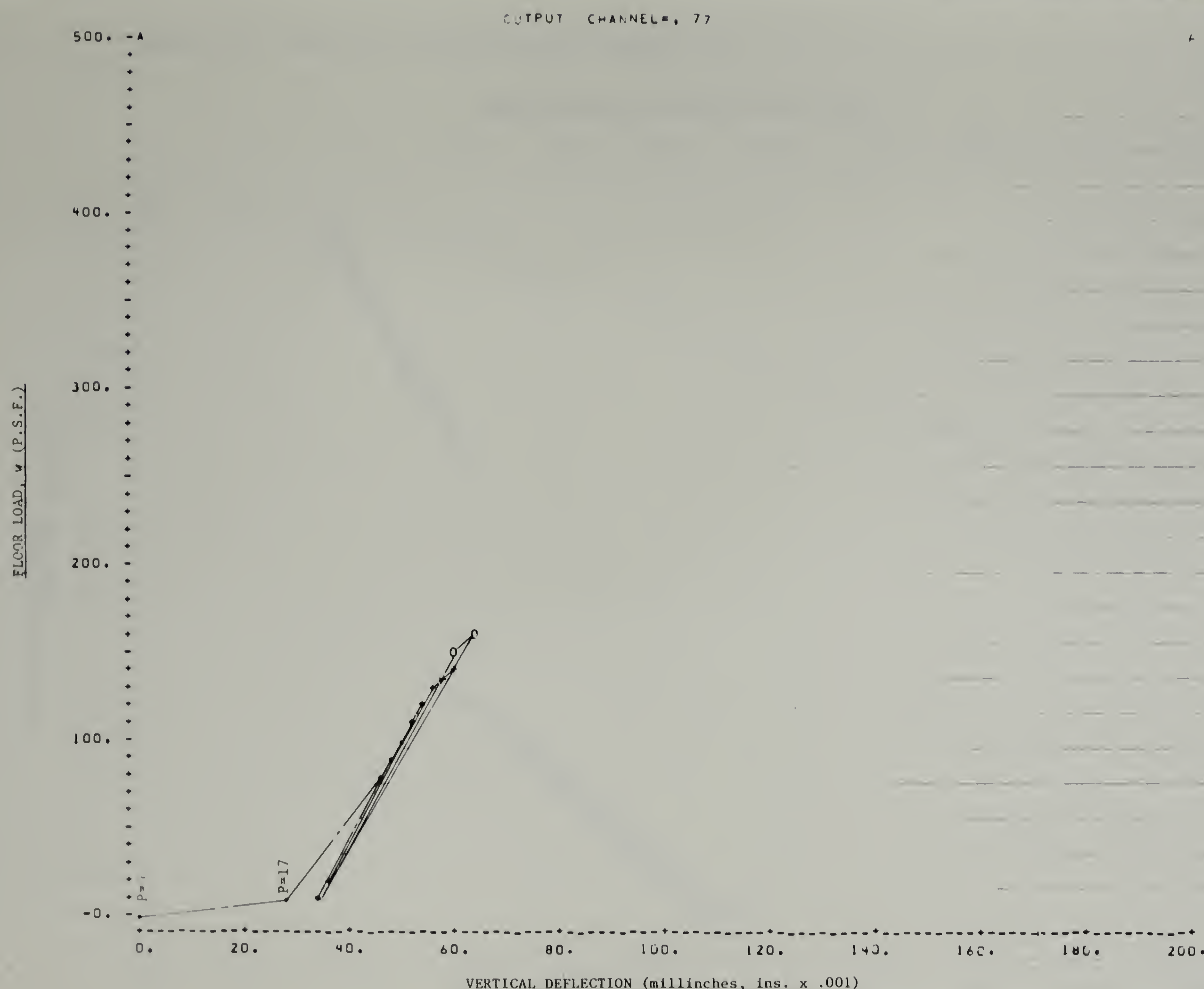


FIGURE NO. A.31 - TEST NO. 9, MAJOR FLOOR LOAD VS. BEAM DEFLECTION

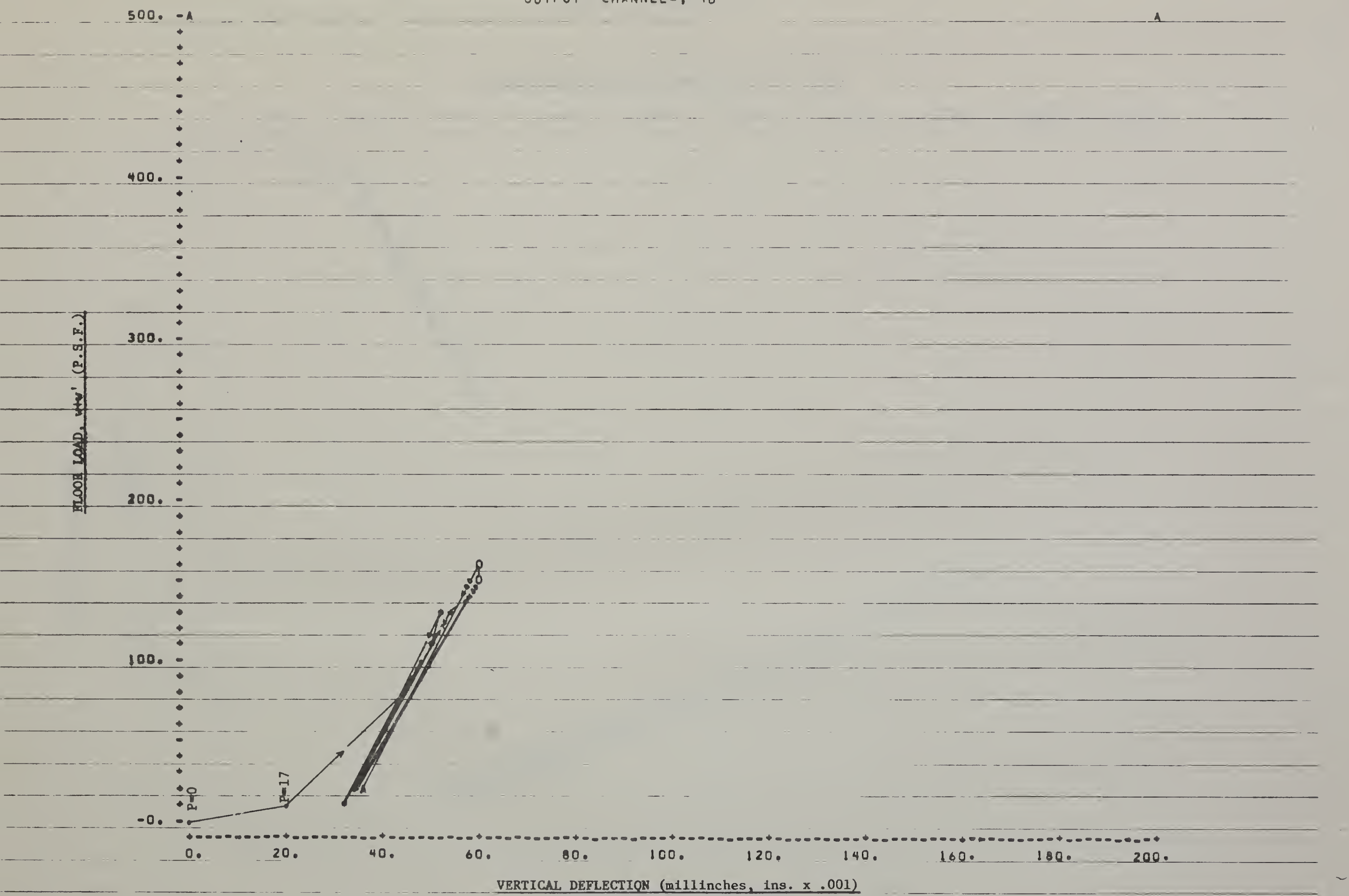


FIGURE NO. A.32 - TEST NO. 9A, FLOOR LOADS VS. BEAM DEFLECTION

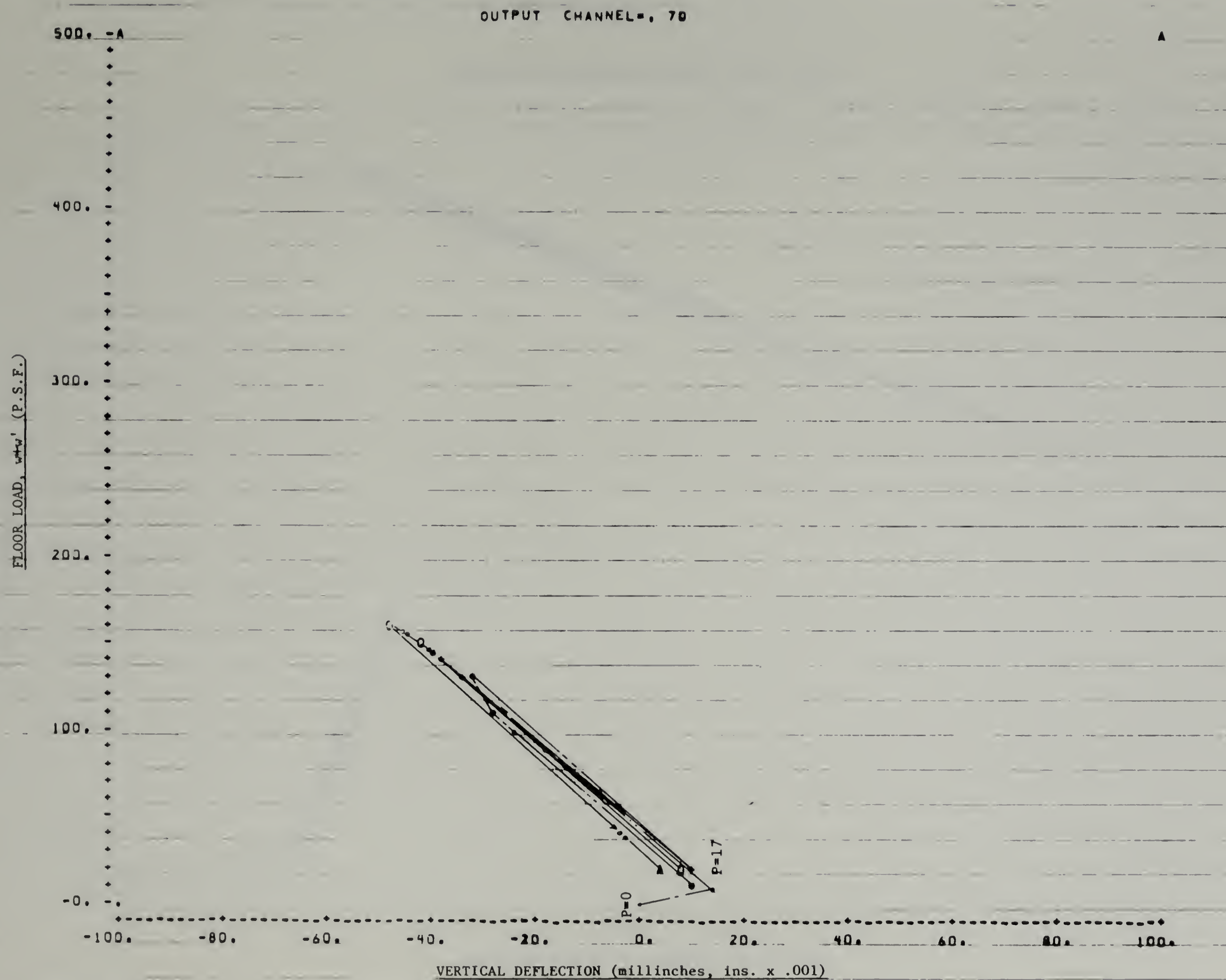


FIGURE NO. A.33 - TEST NO. 9A, FLOOR LOADS VS. BEAM DEFLECTION

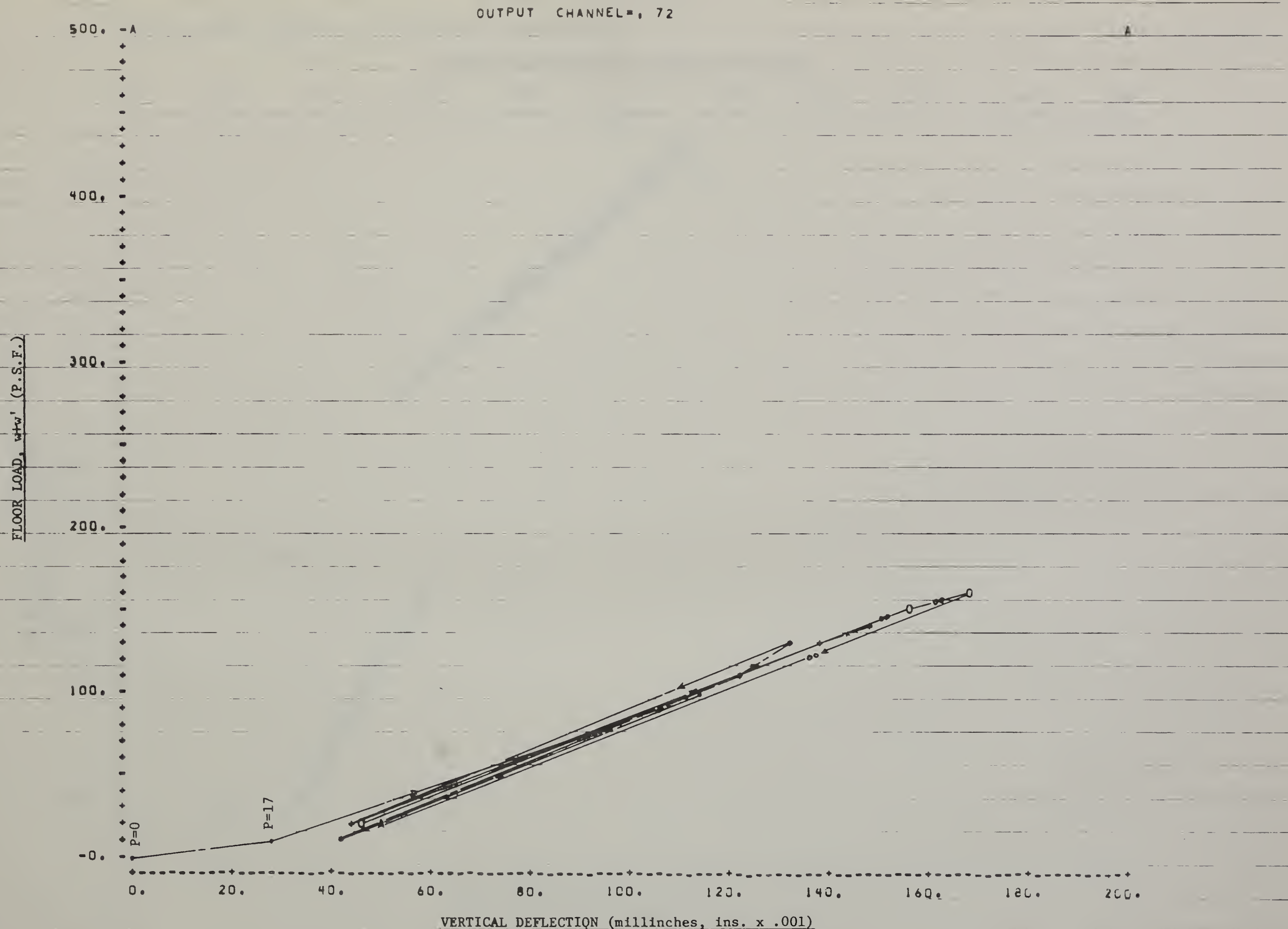


FIGURE NO. A.34 - TEST NO. 9A, FLOOR LOADS VS. BEAM DEFLECTION

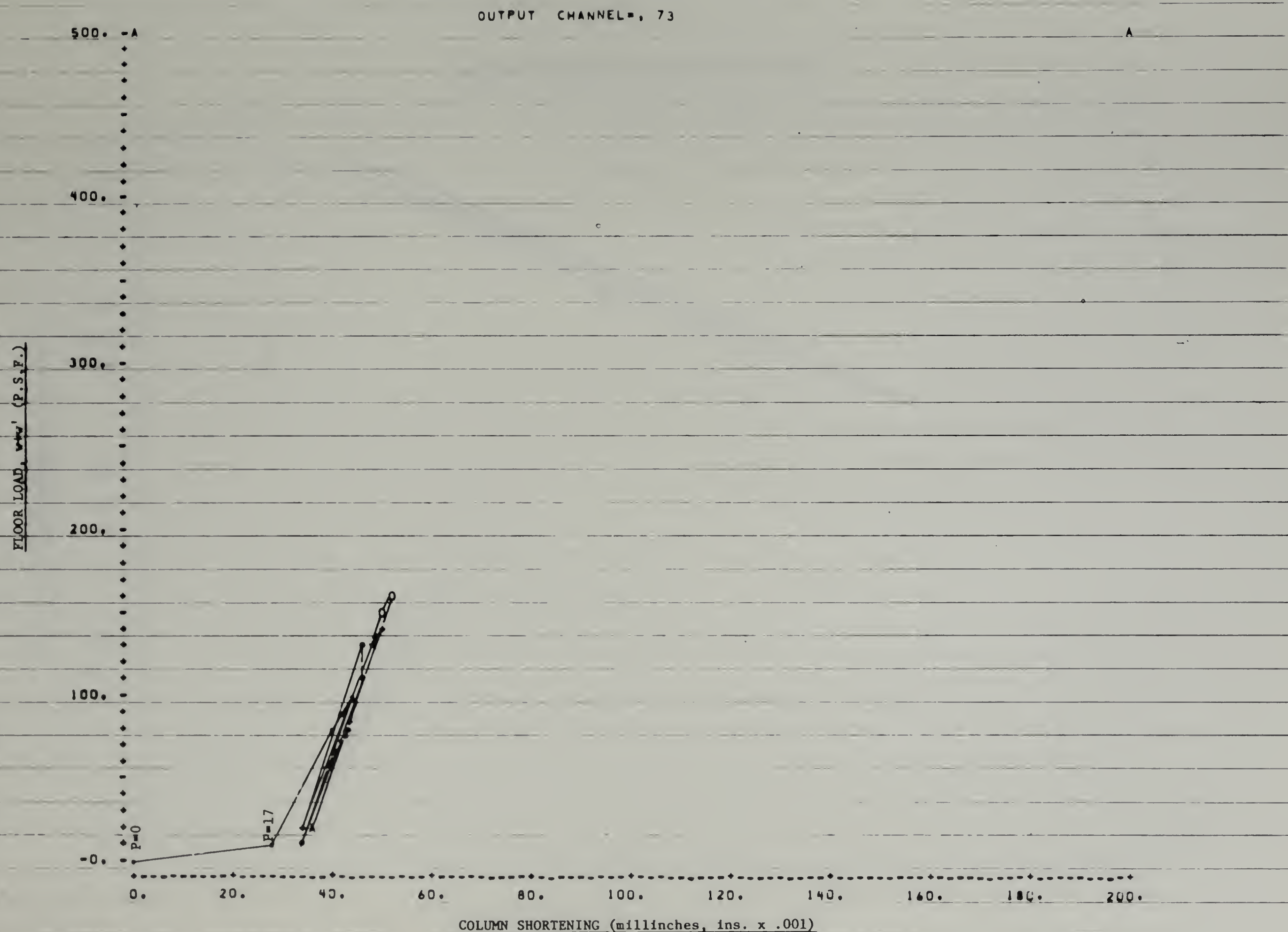


FIGURE NO. A.35 - TEST NO. 9A, FLOOR LOAD VS. COLUMN SHORTENING

OUTPUT CHANNEL=, 76

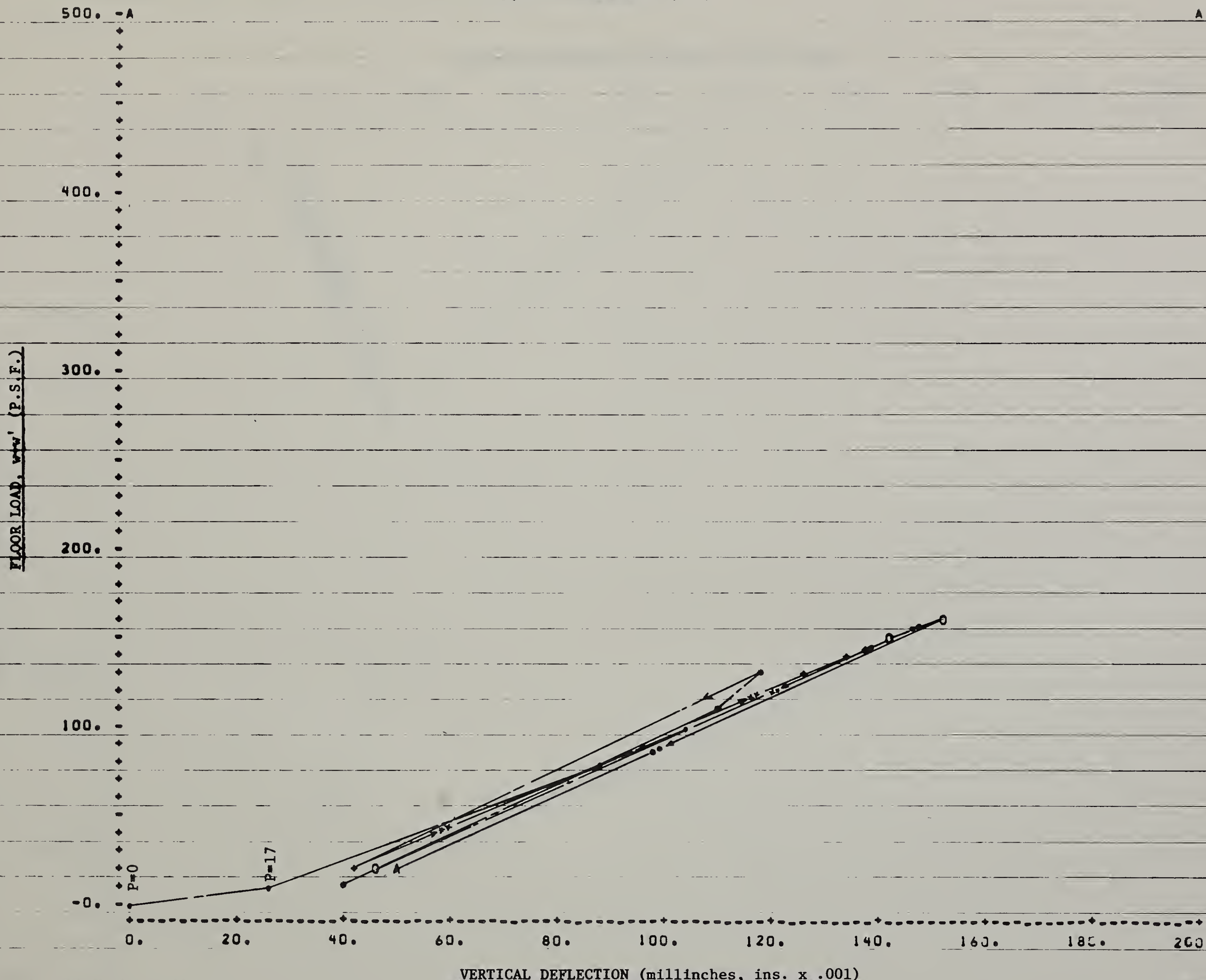


FIGURE NO. A.36 - TEST NO. 9A, FLOOR LOAD VS. SLAB DEFLECTION

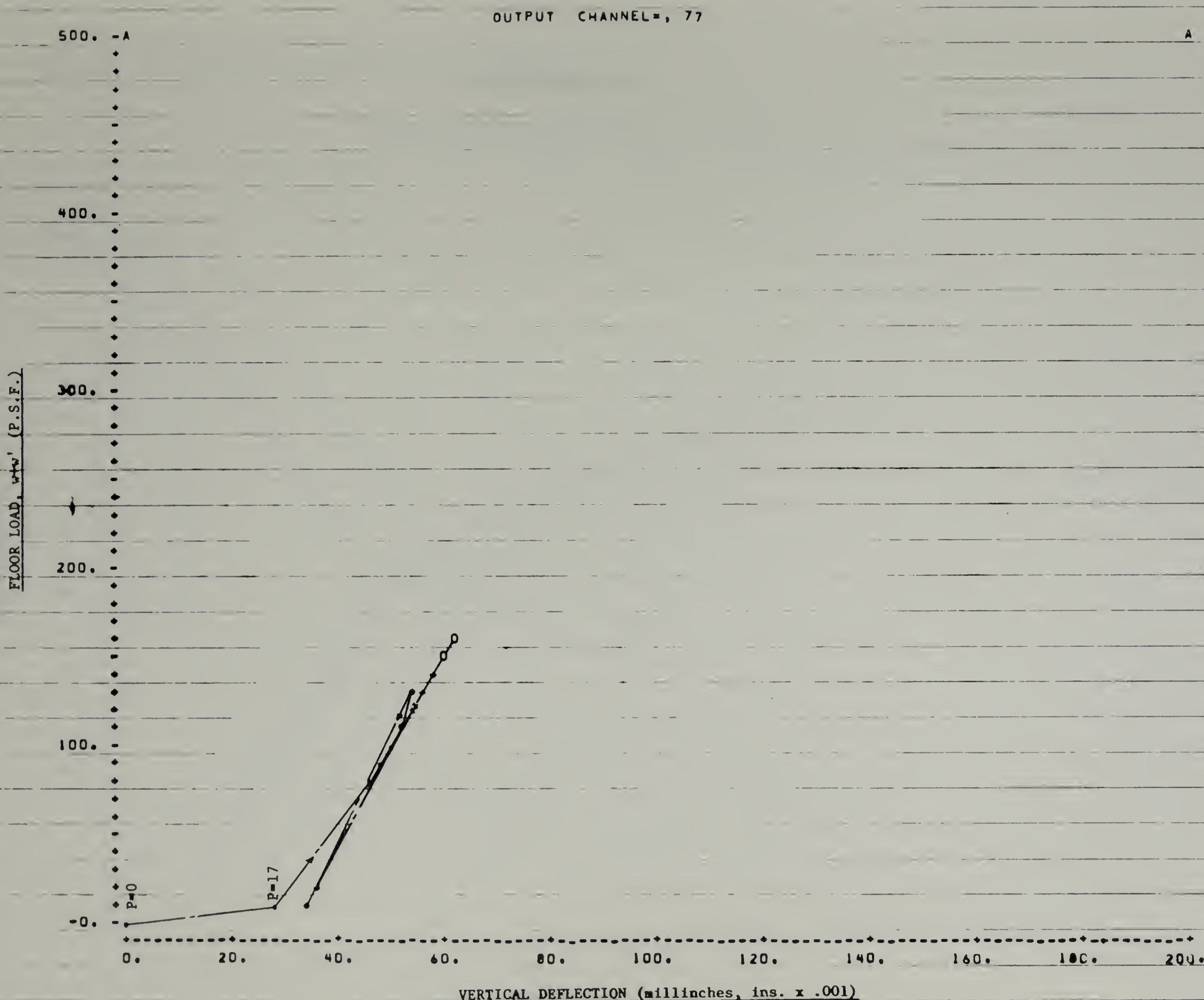


FIGURE NO. A.37 - TEST NO. 9A, FLOOR LOAD VS. BEAM DEFLECTION

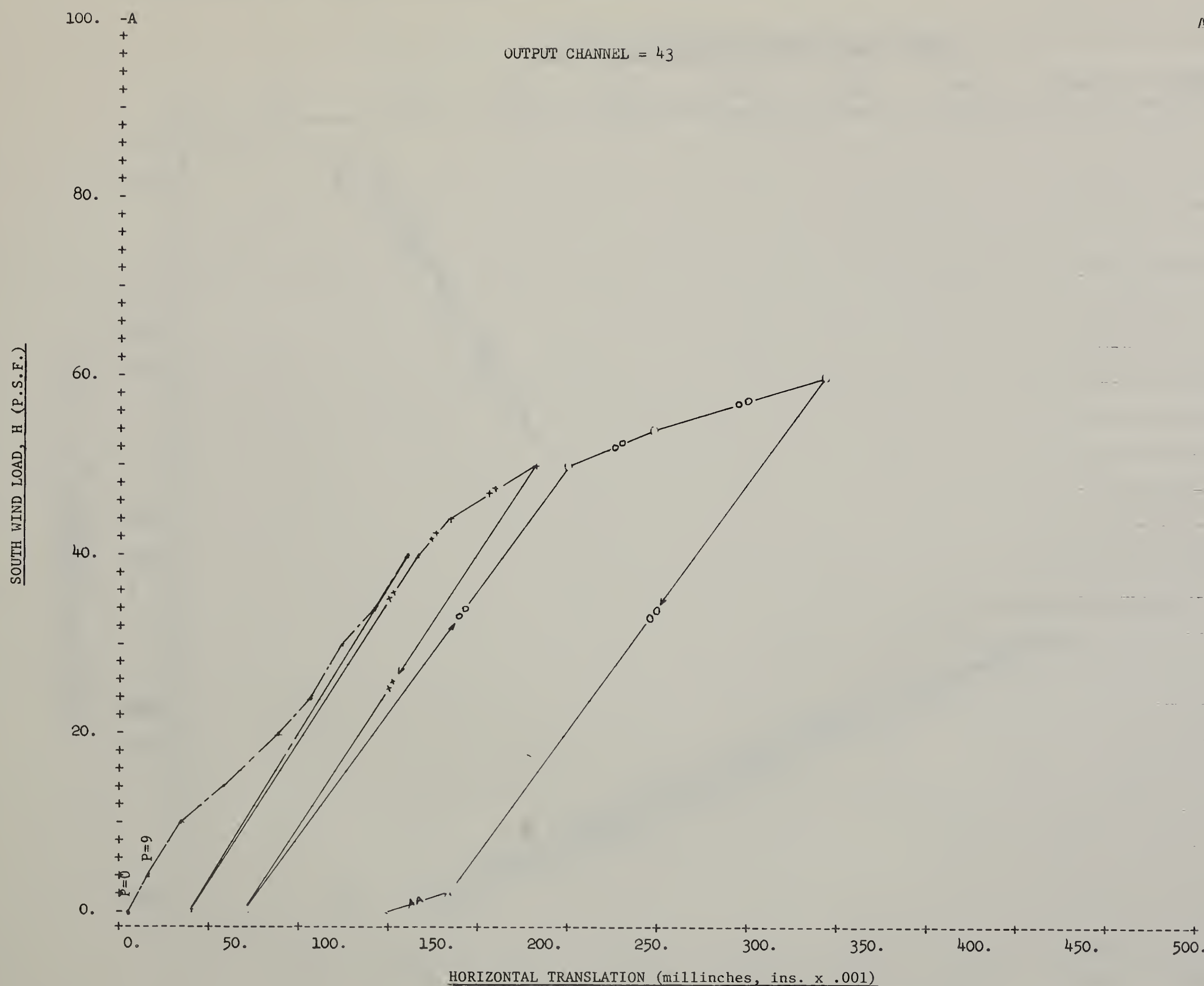


FIGURE NO. A.38 - TEST NO. 10, SOUTH WIND LOAD VS. TRANSLATION

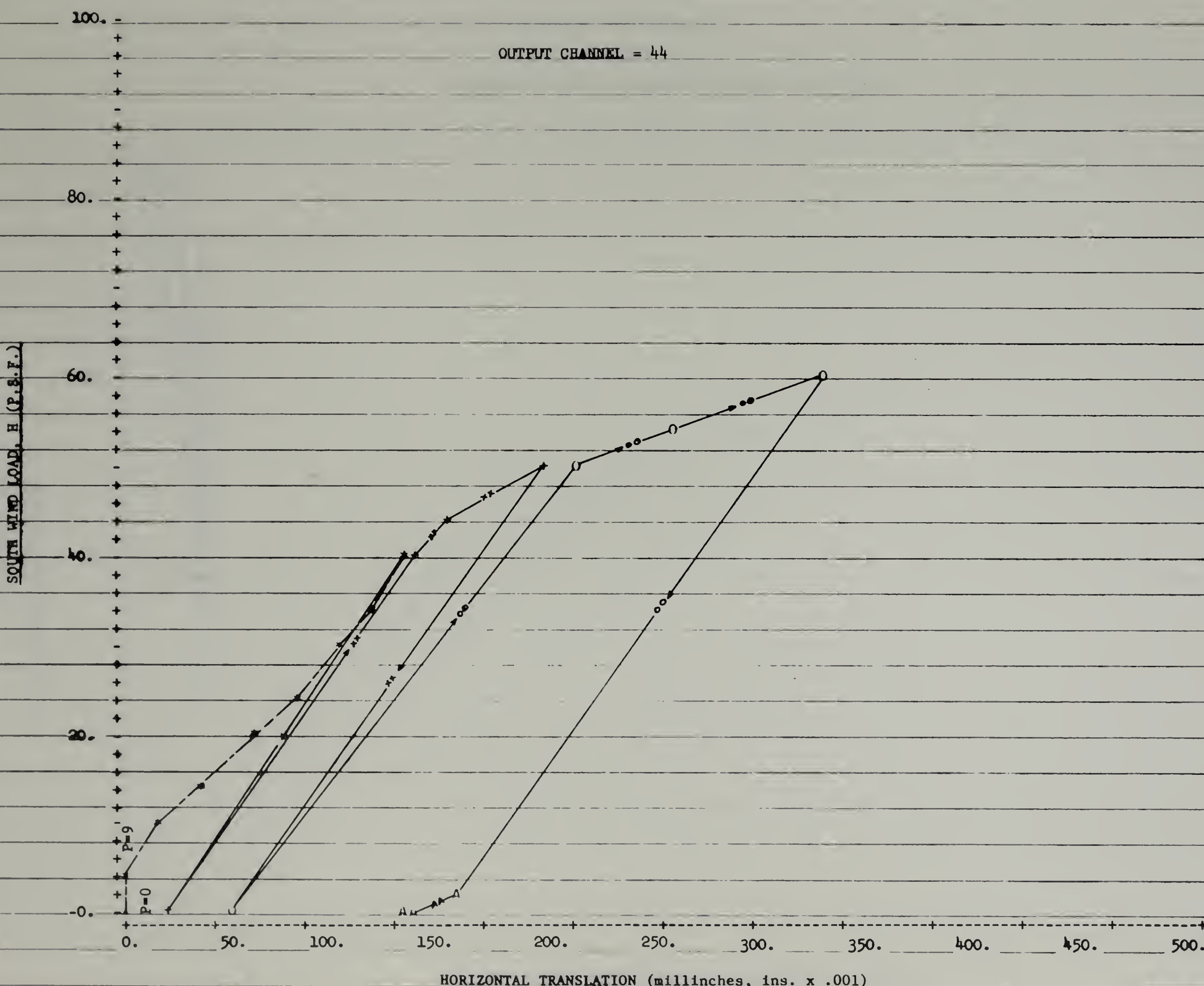


FIGURE NO. A.39 - TEST NO. 10, SOUTH WIND LOAD VS. TRANSLATION

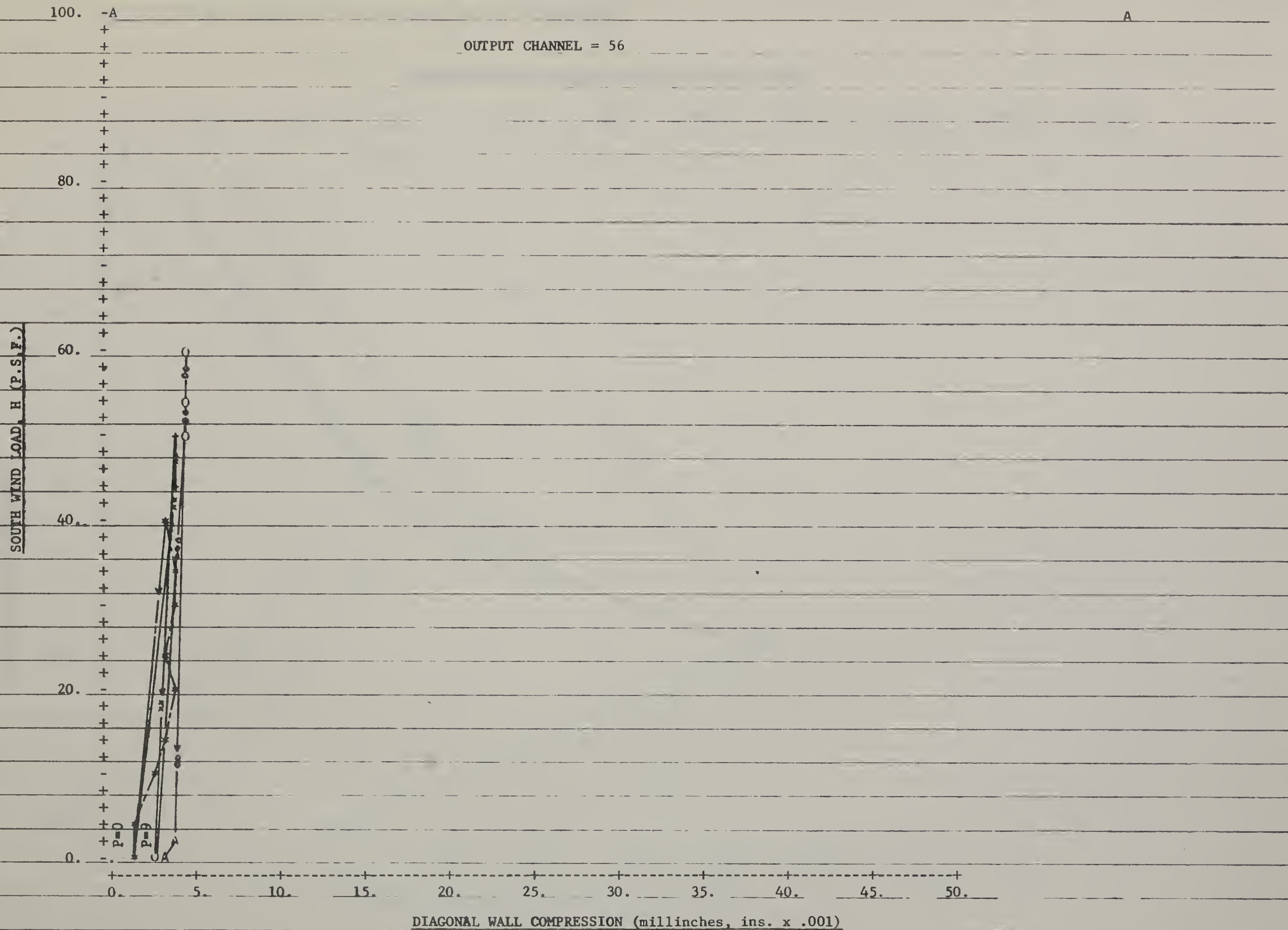


FIGURE NO. A.40 - TEST NO. 10, SOUTH WIND LOAD VS. WALL COMPRESSION

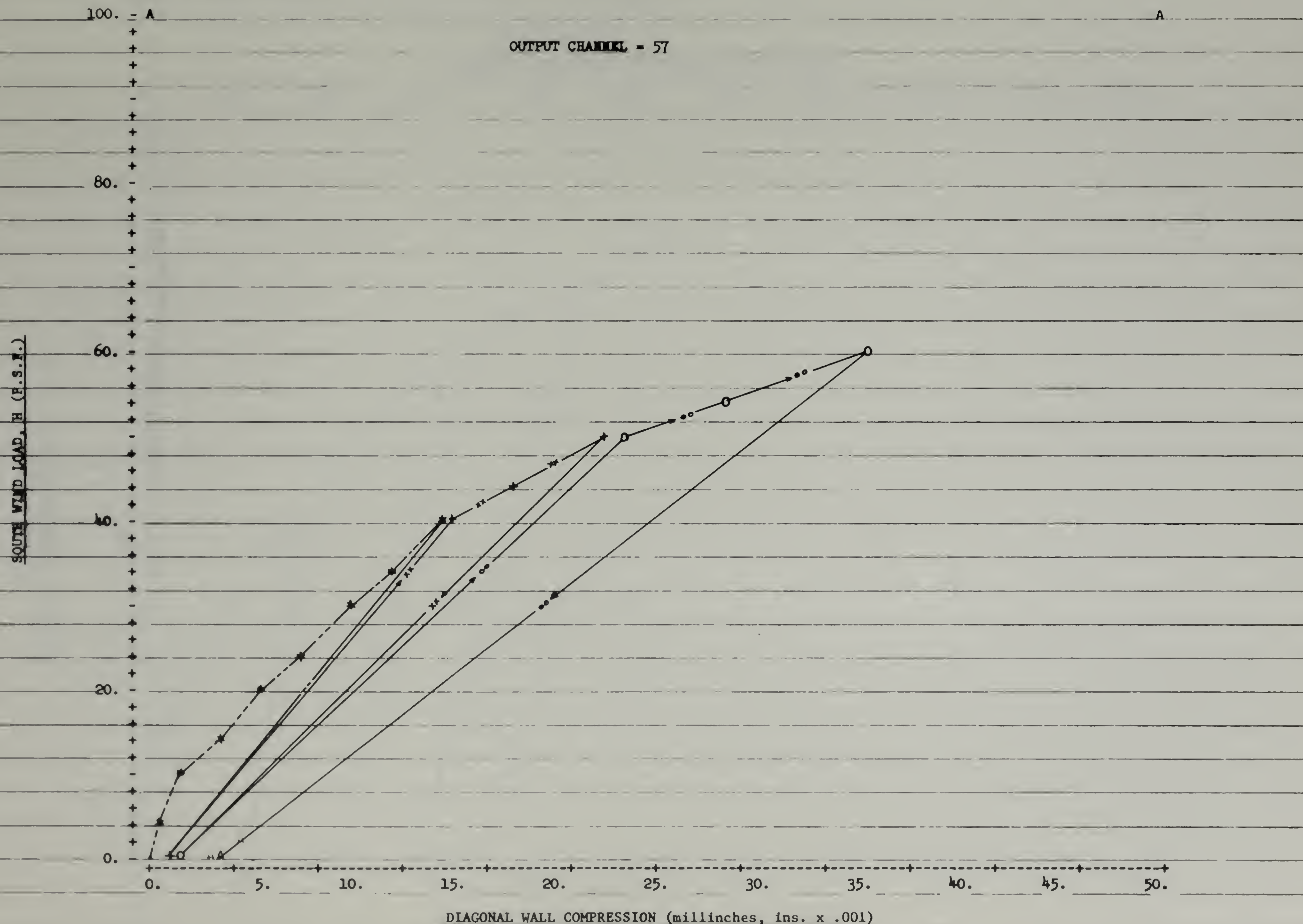


FIGURE NO. A.41 - TEST NO. 10, SOUTH WIND LOAD VS. WALL COMPRESSION

OUTPUT CHANNEL = 72

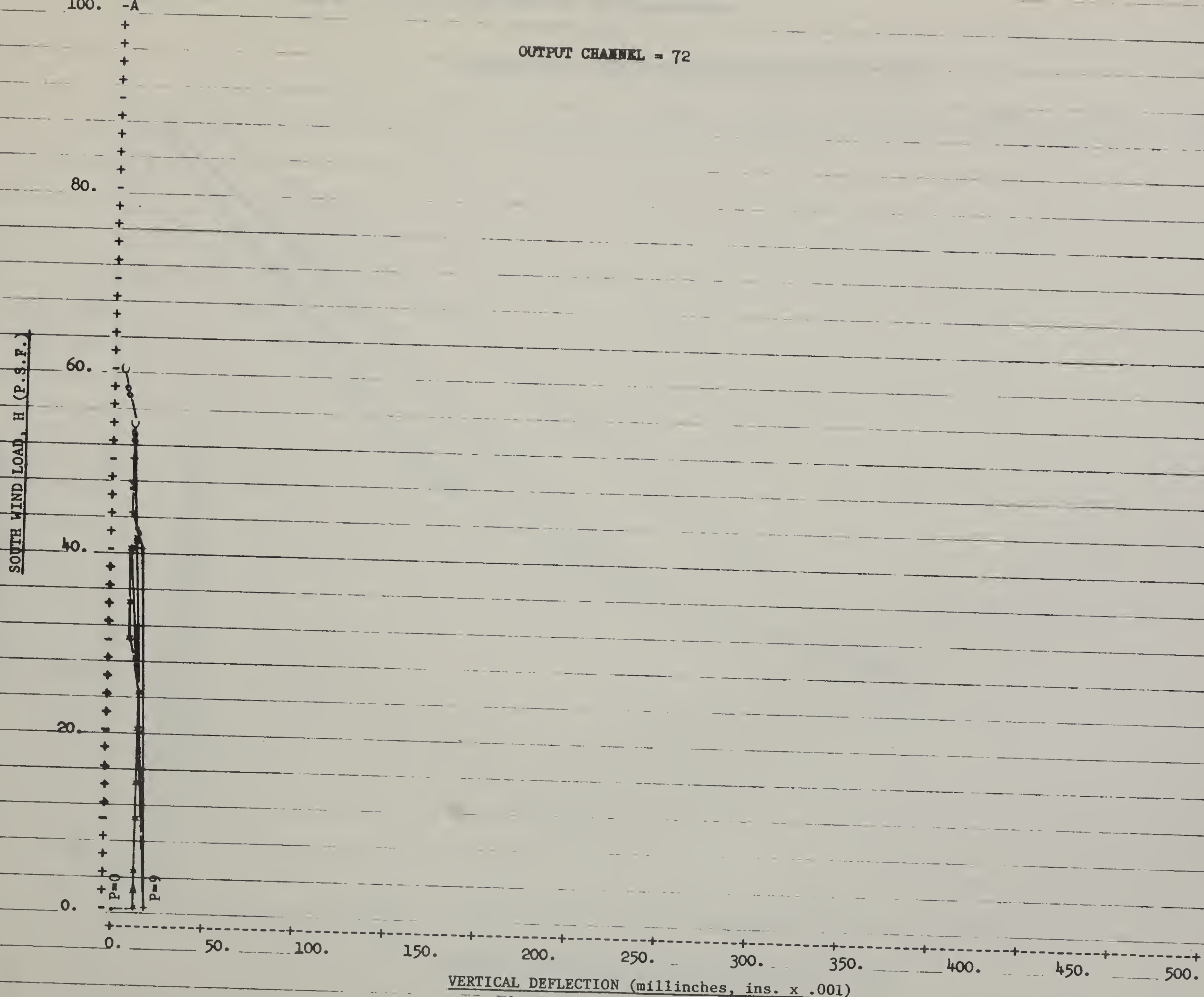


FIGURE NO. A.42 - TEST NO. 10, SOUTH WIND LOAD VS. BEAM DEFLECTION

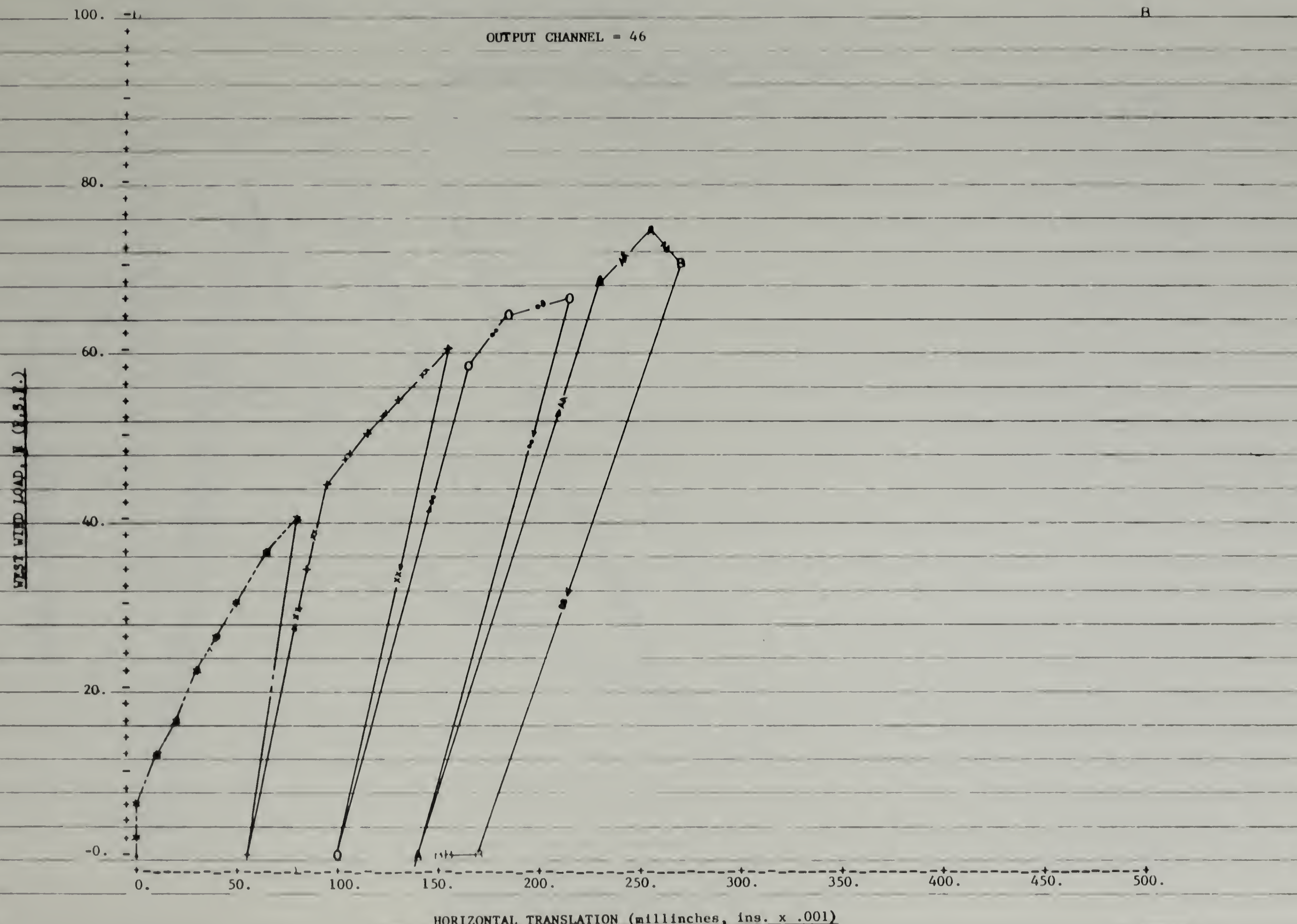


FIGURE NO. A.43 - TEST NO. 11, WEST WIND LOAD VS. TRANSLATION

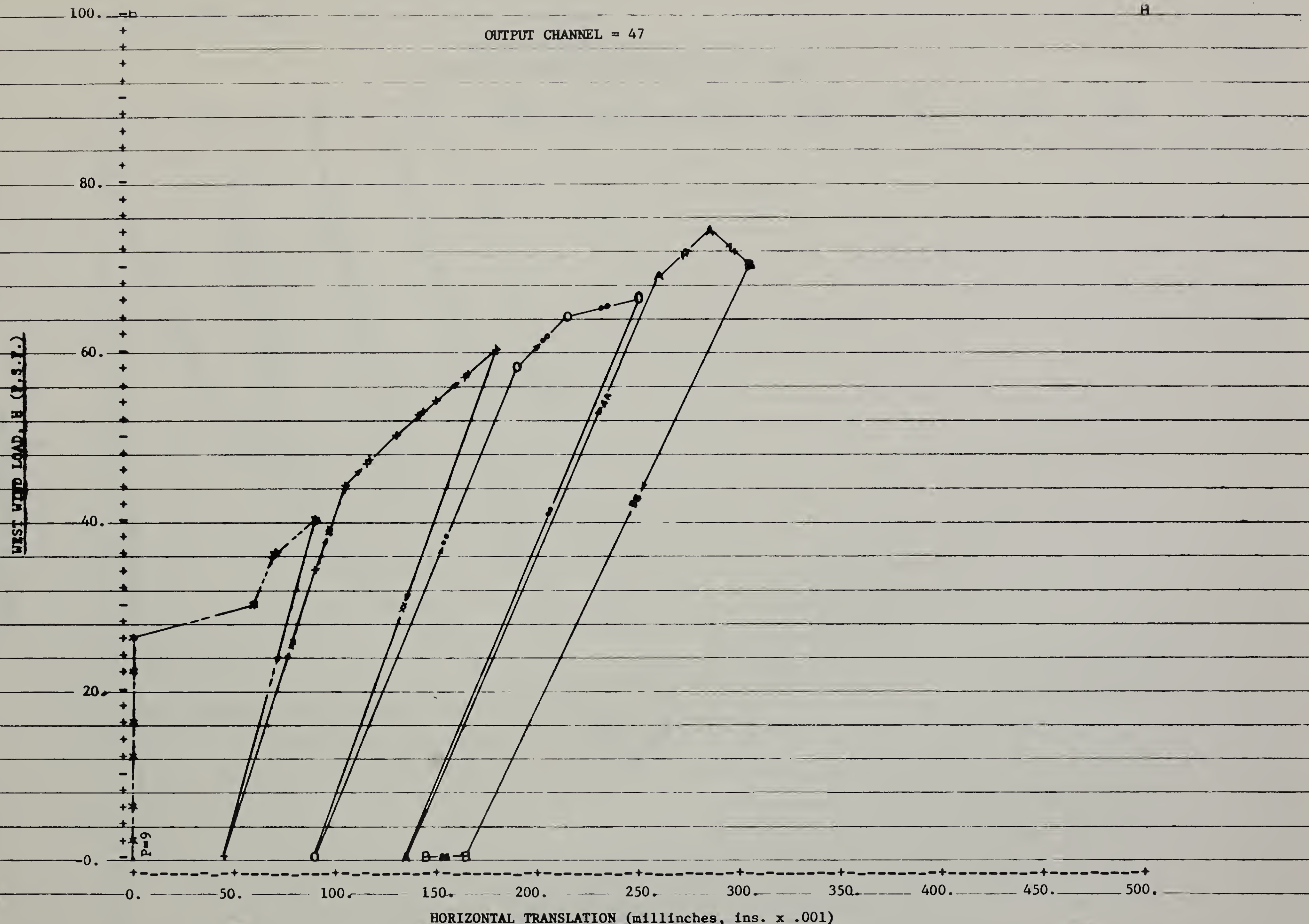


FIGURE NO. A.44 - TEST NO. 11, WEST WIND LOAD VS. TRANSLATION

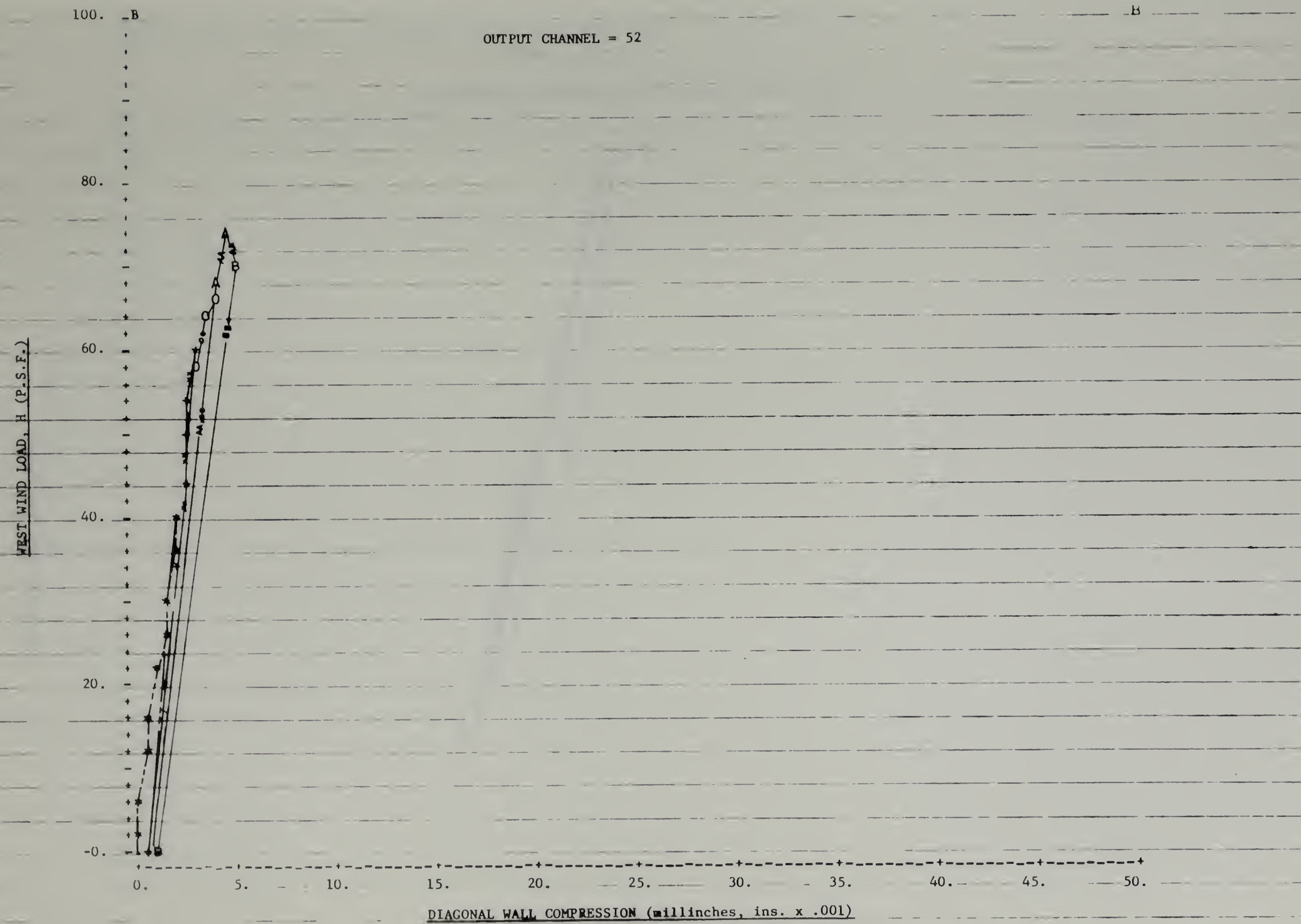


FIGURE NO. A.45 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

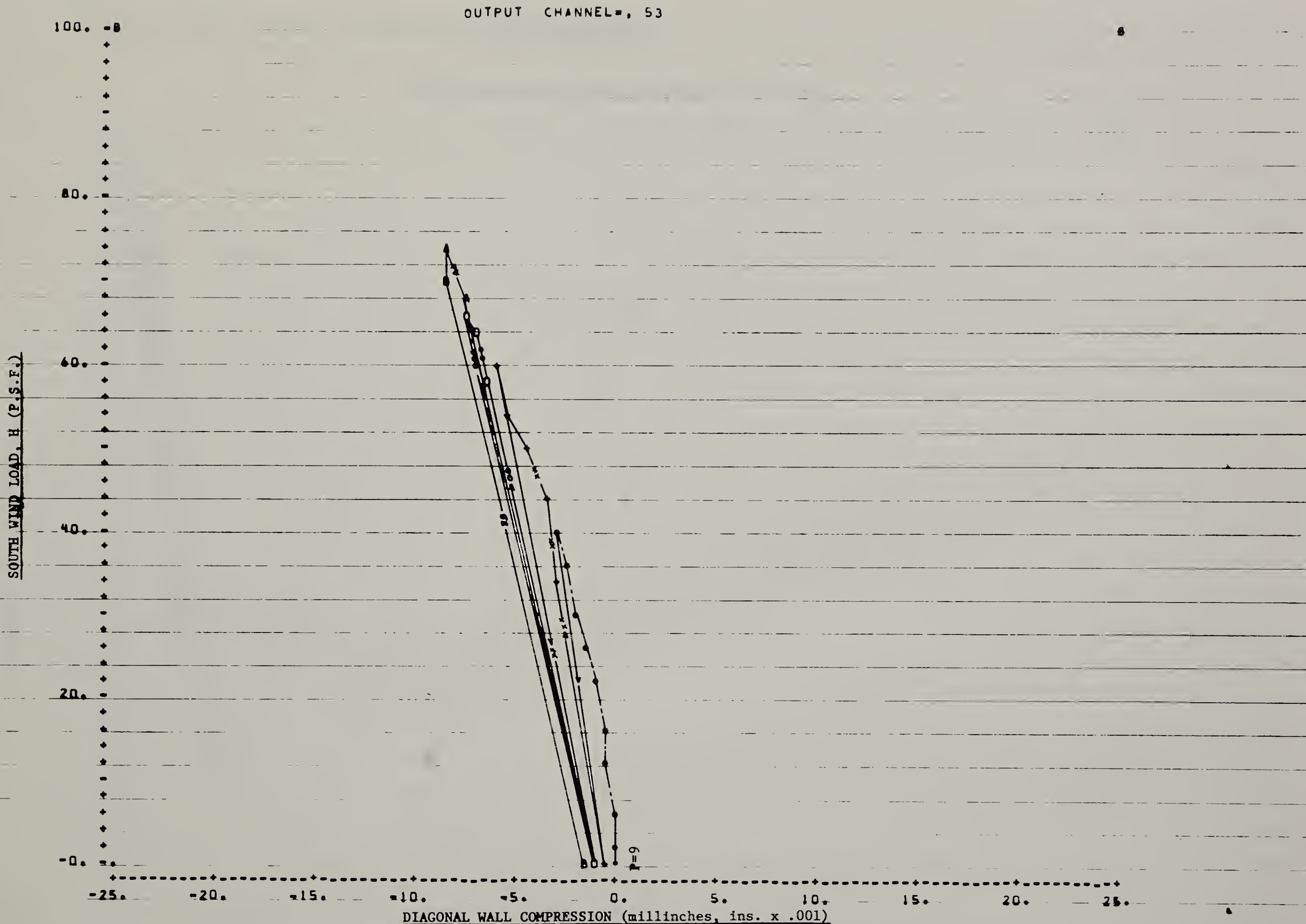


FIGURE NO. A.46 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

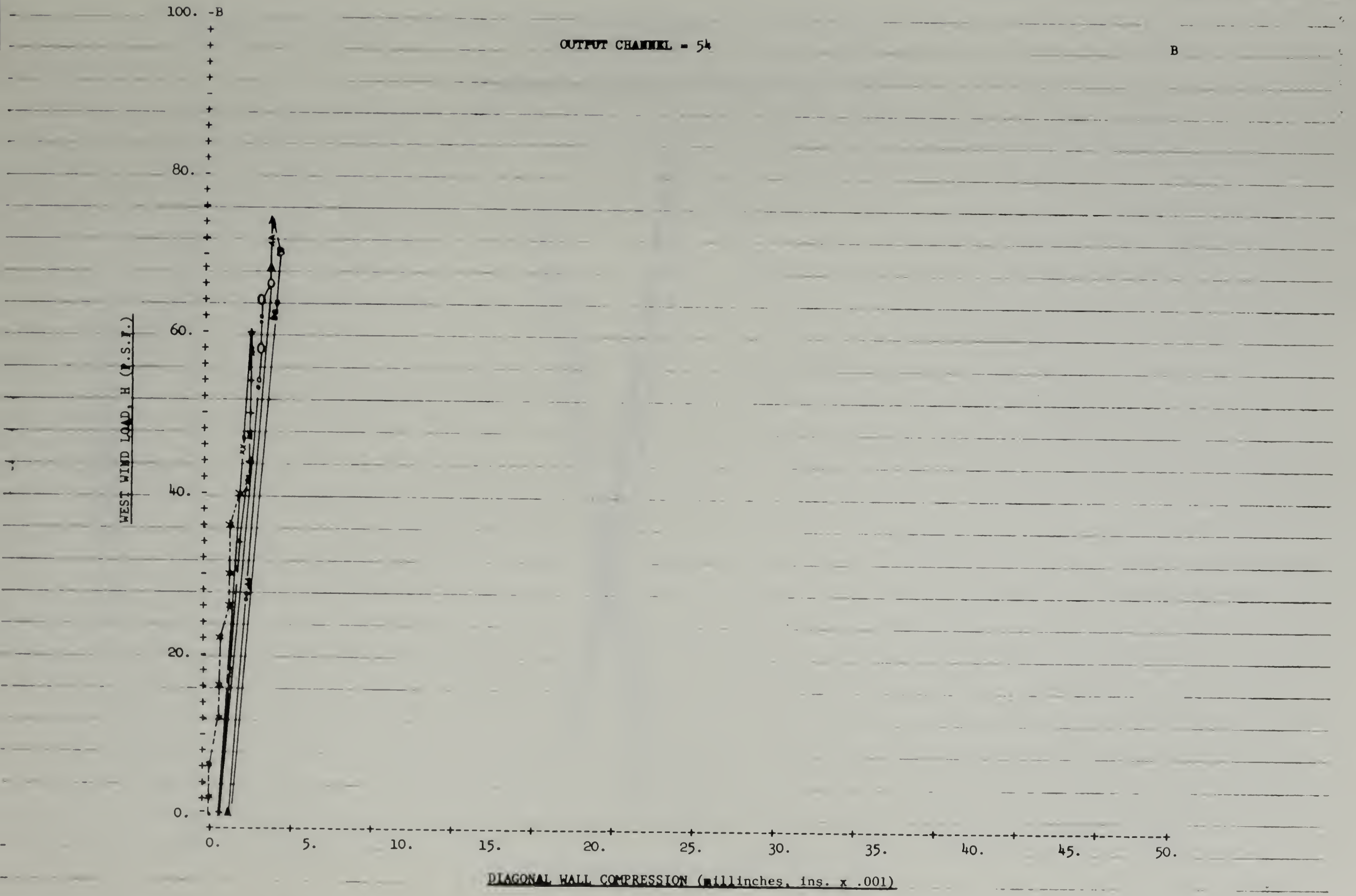


FIGURE NO. A.47, TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

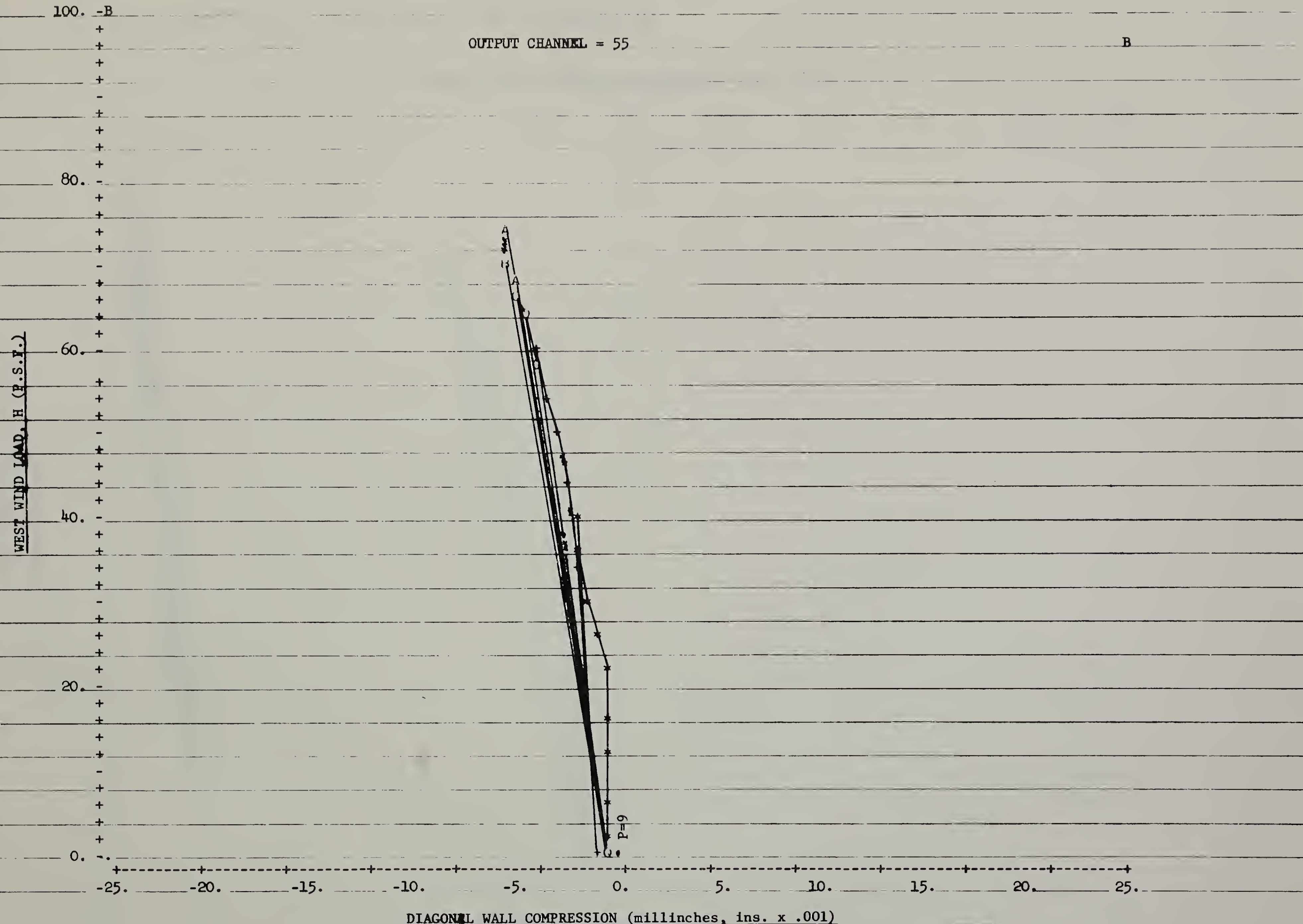


FIGURE NO. A.48 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

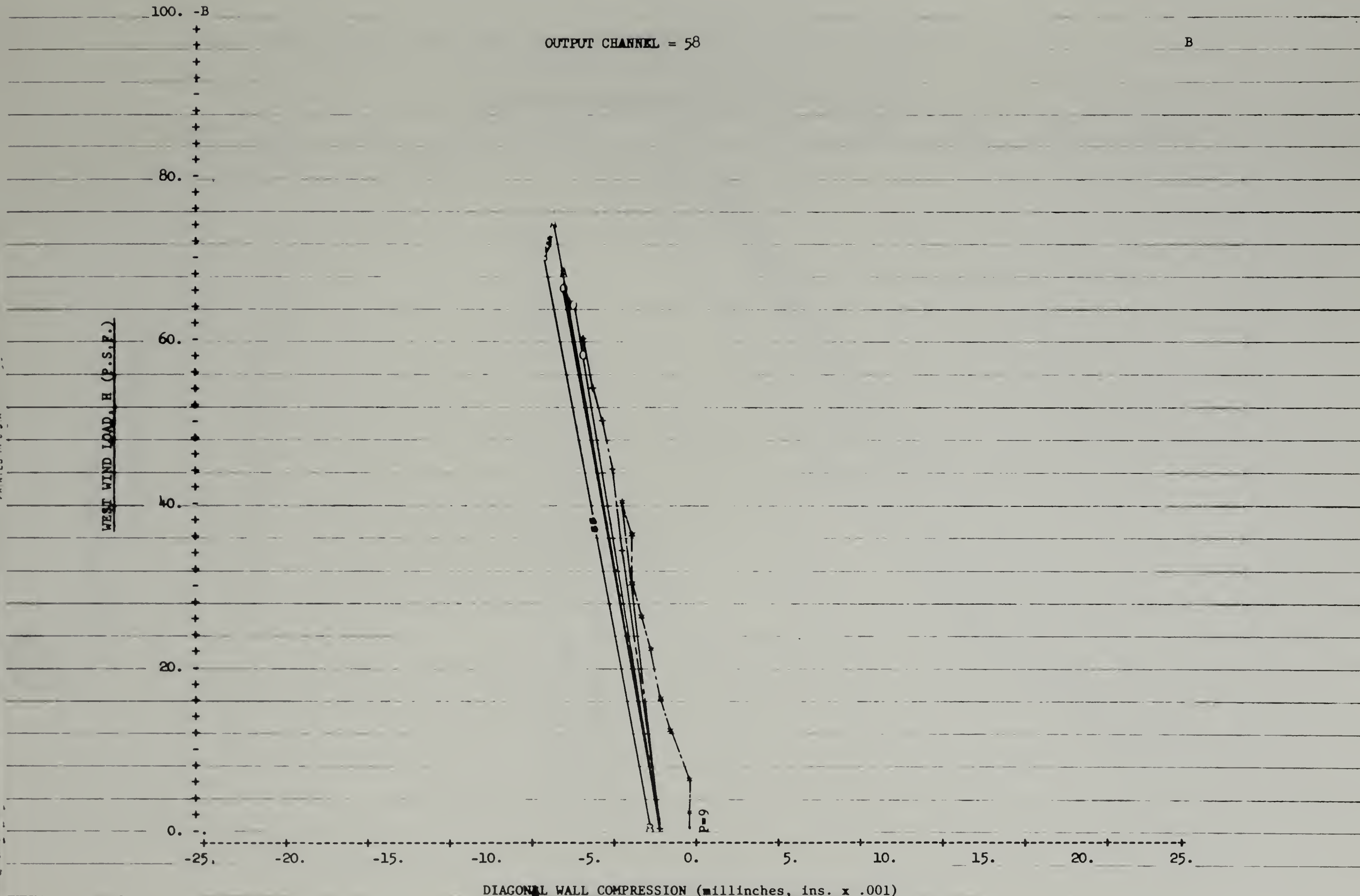


FIGURE NO. A.49 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

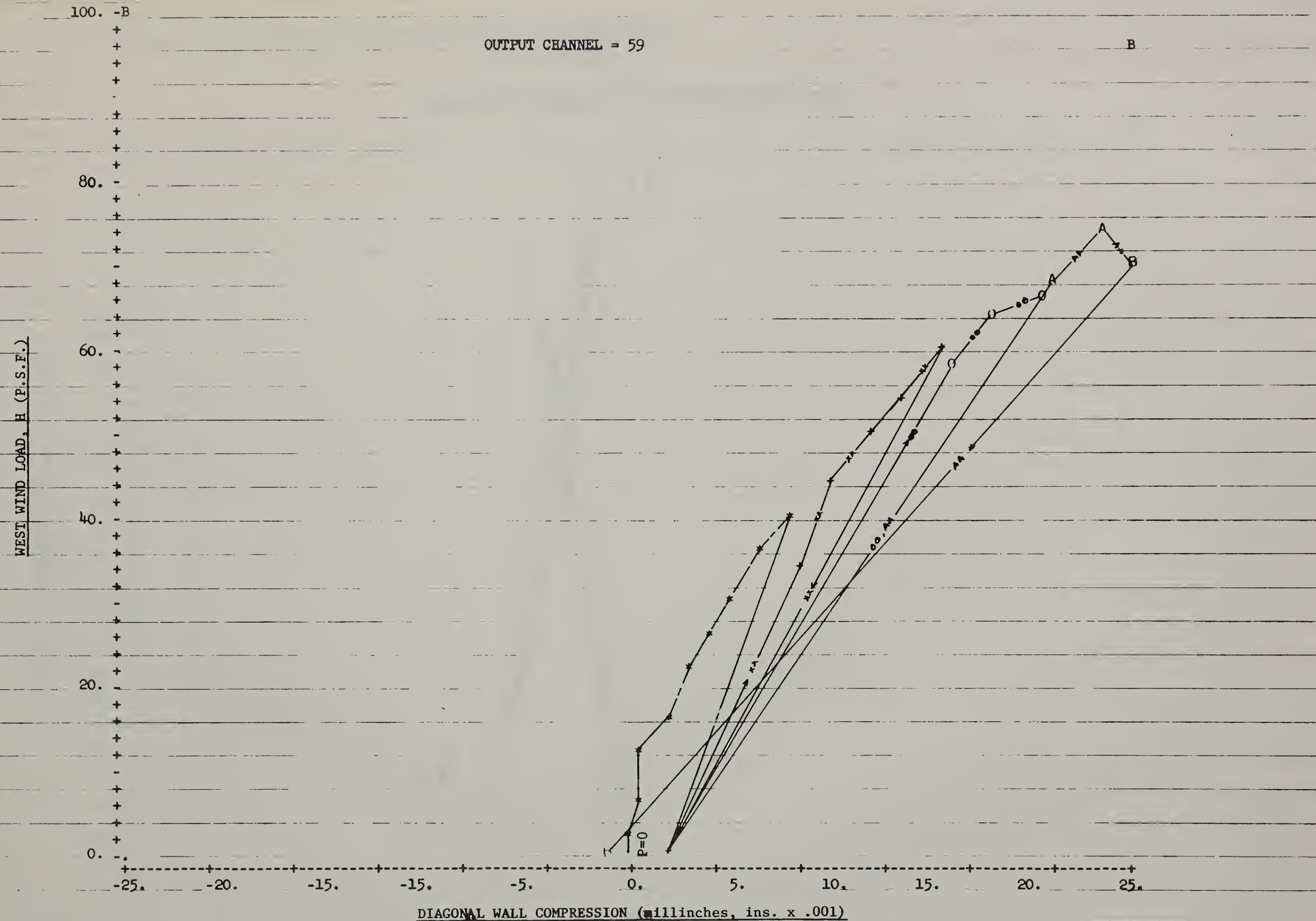


FIGURE NO. A.50 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

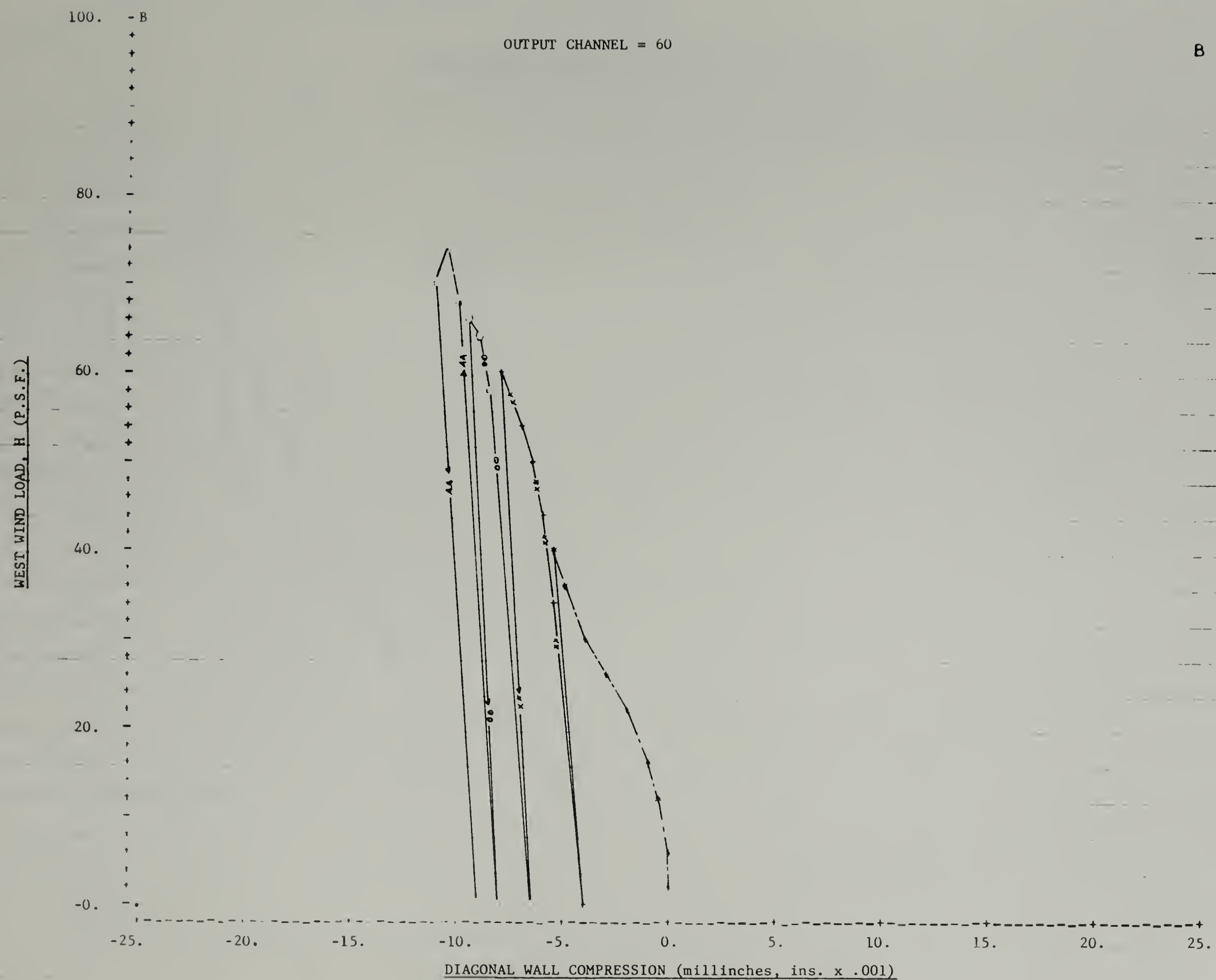


FIGURE NO. A.51 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

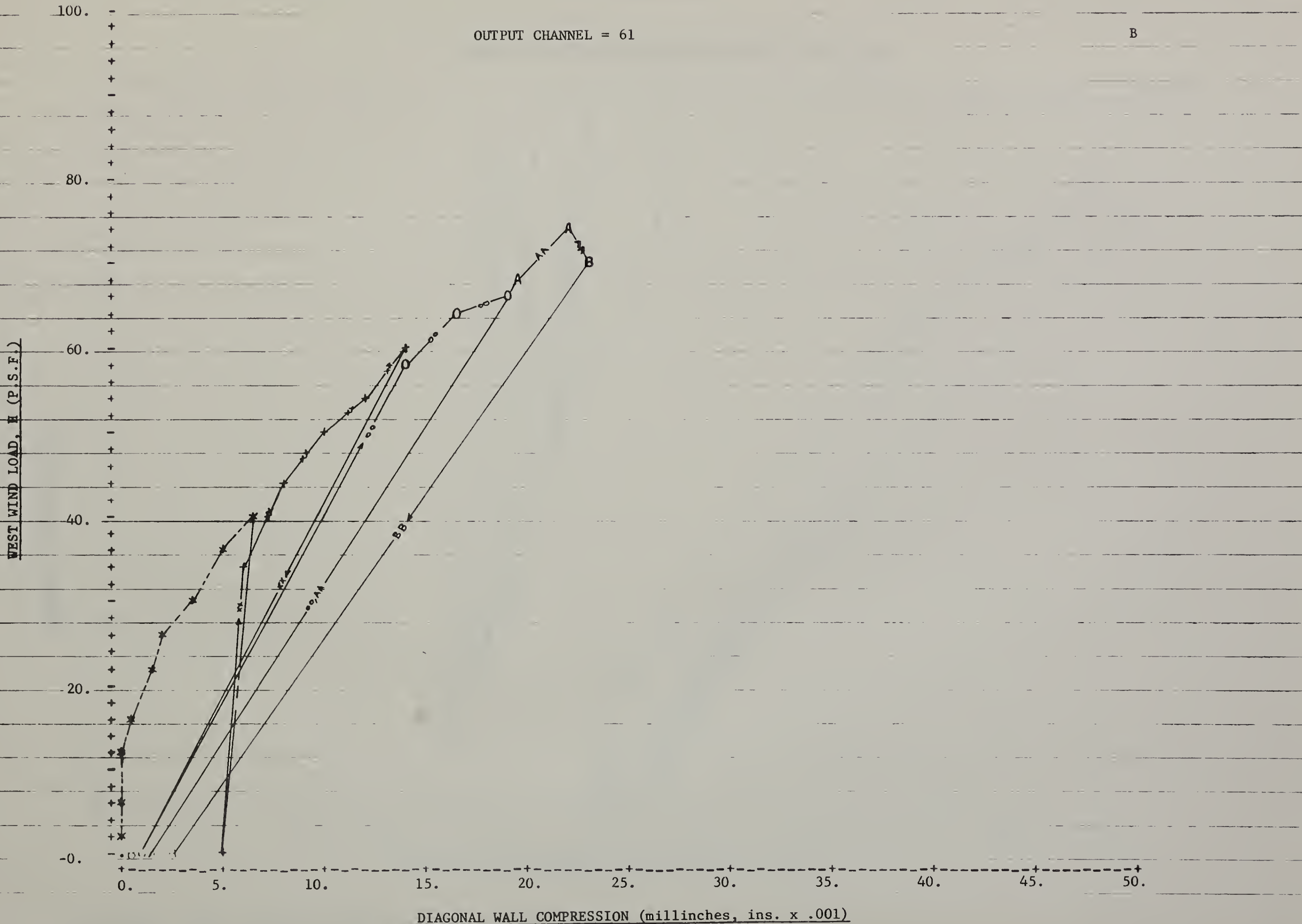


FIGURE NO. A.52 - TEST NO. 11, WEST WIND LOAD VS. WALL COMPRESSION

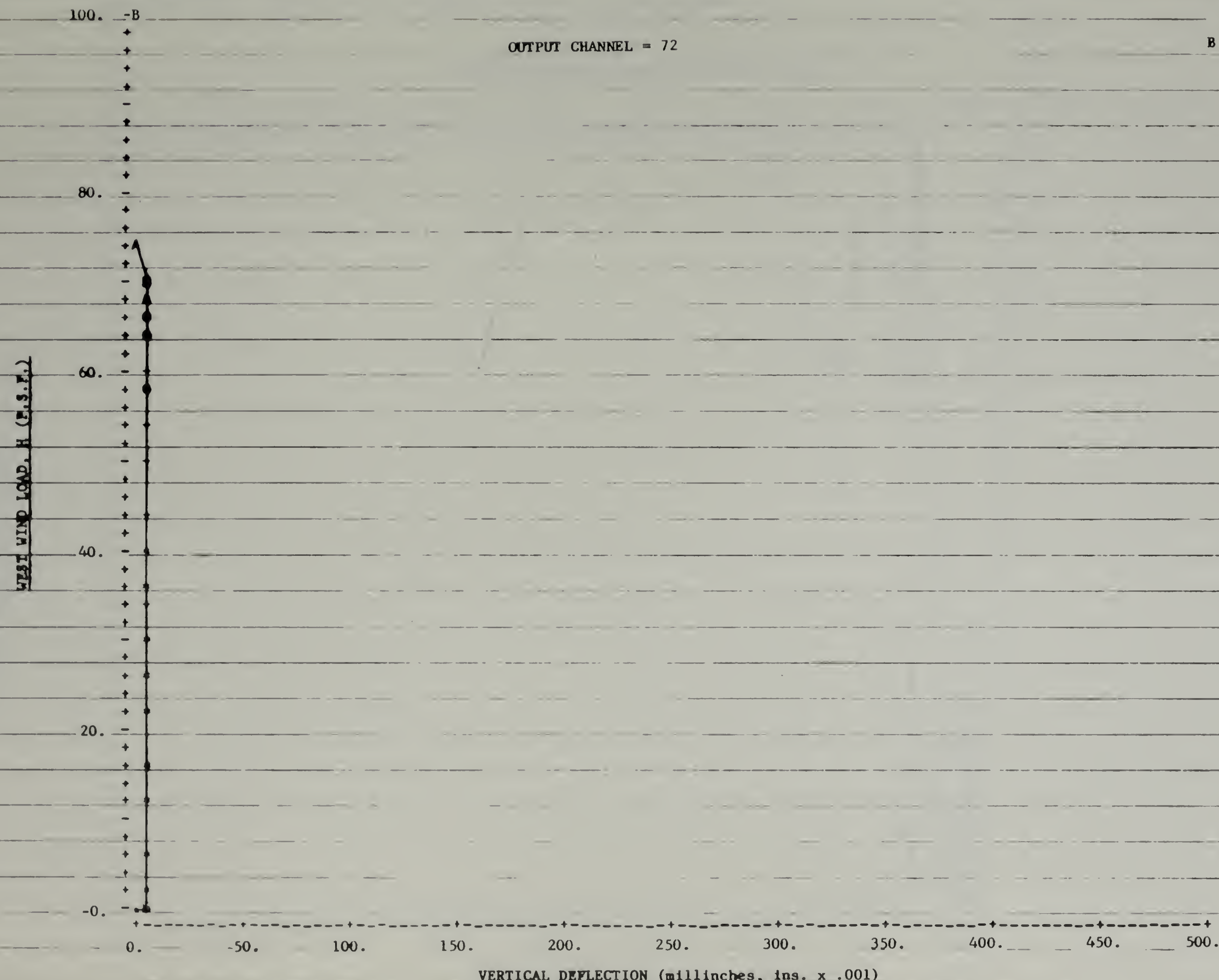


FIGURE NO. A.53 - TEST NO. 11, WEST WIND LOAD VS. BEAM DEFLECTION

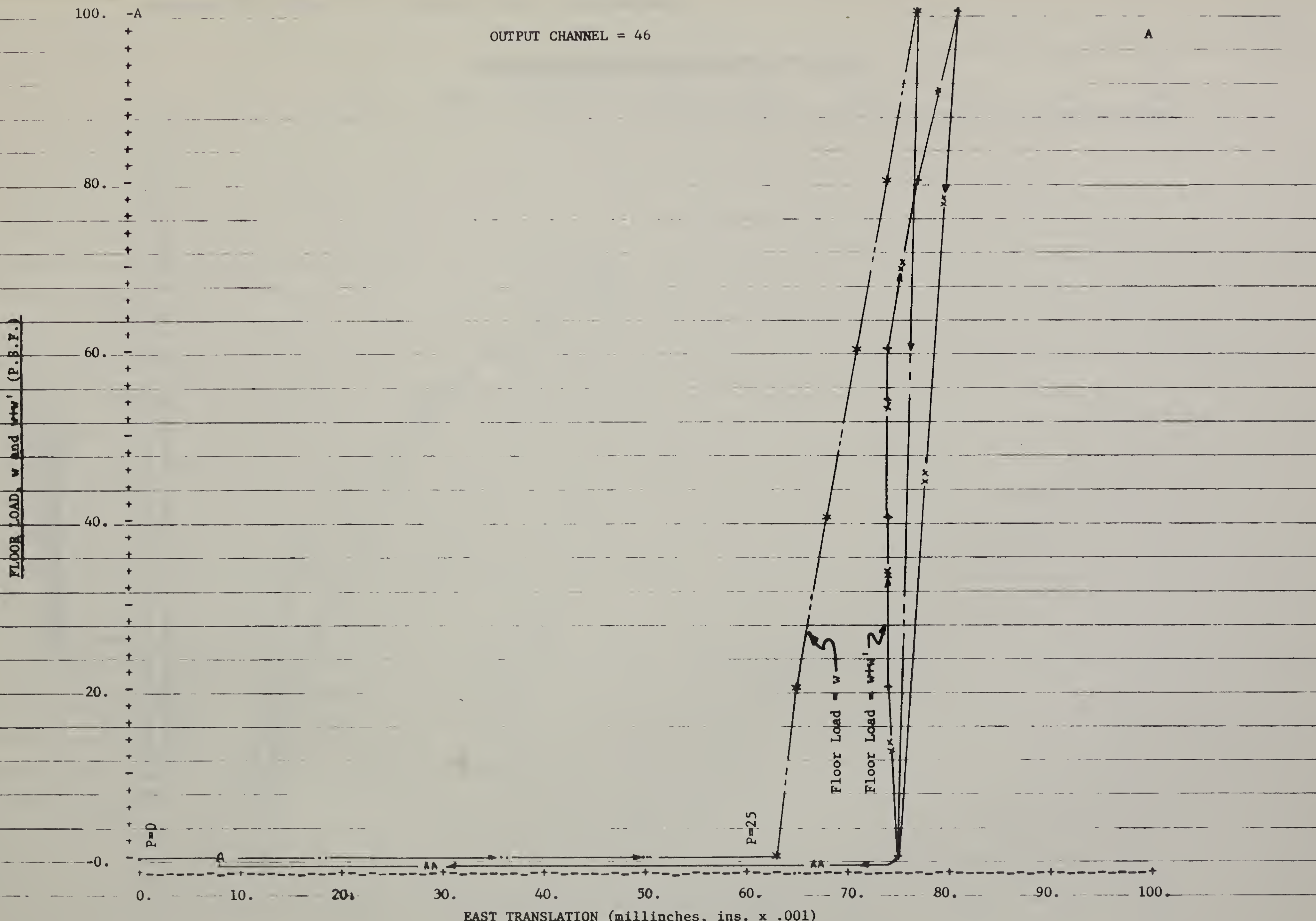


FIGURE NO. A.54- TEST NO. 12 & 13, VERTICAL LOAD VS. EAST TRANSLATION

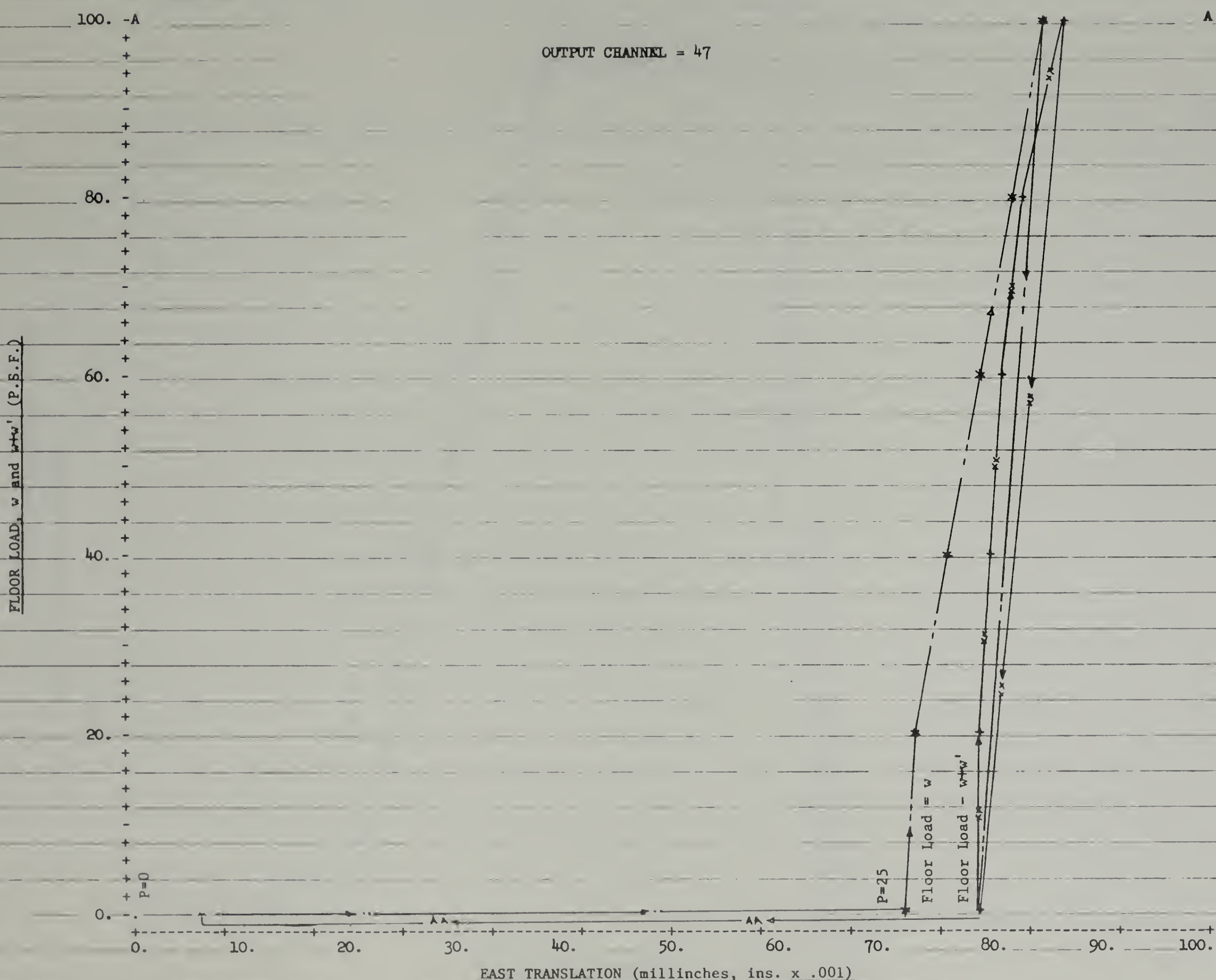


FIGURE NO. A.55 - TEST NO. 12 & 13, VERTICAL LOAD VS. EAST TRANSLATION

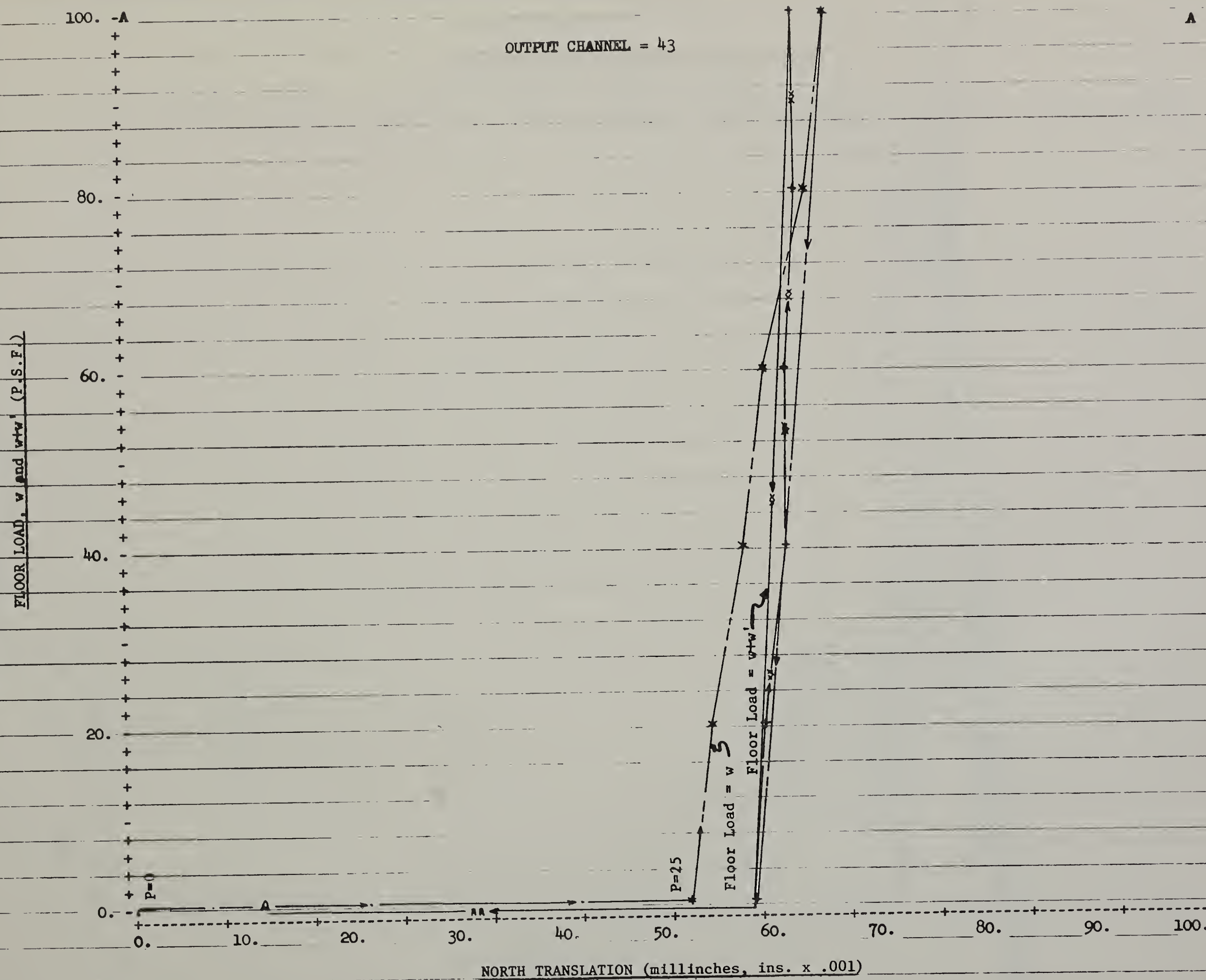


FIGURE NO. A.56 - TEST NO. 12A and 13A, VERTICAL LOAD VS. NORTH TRANSLATION

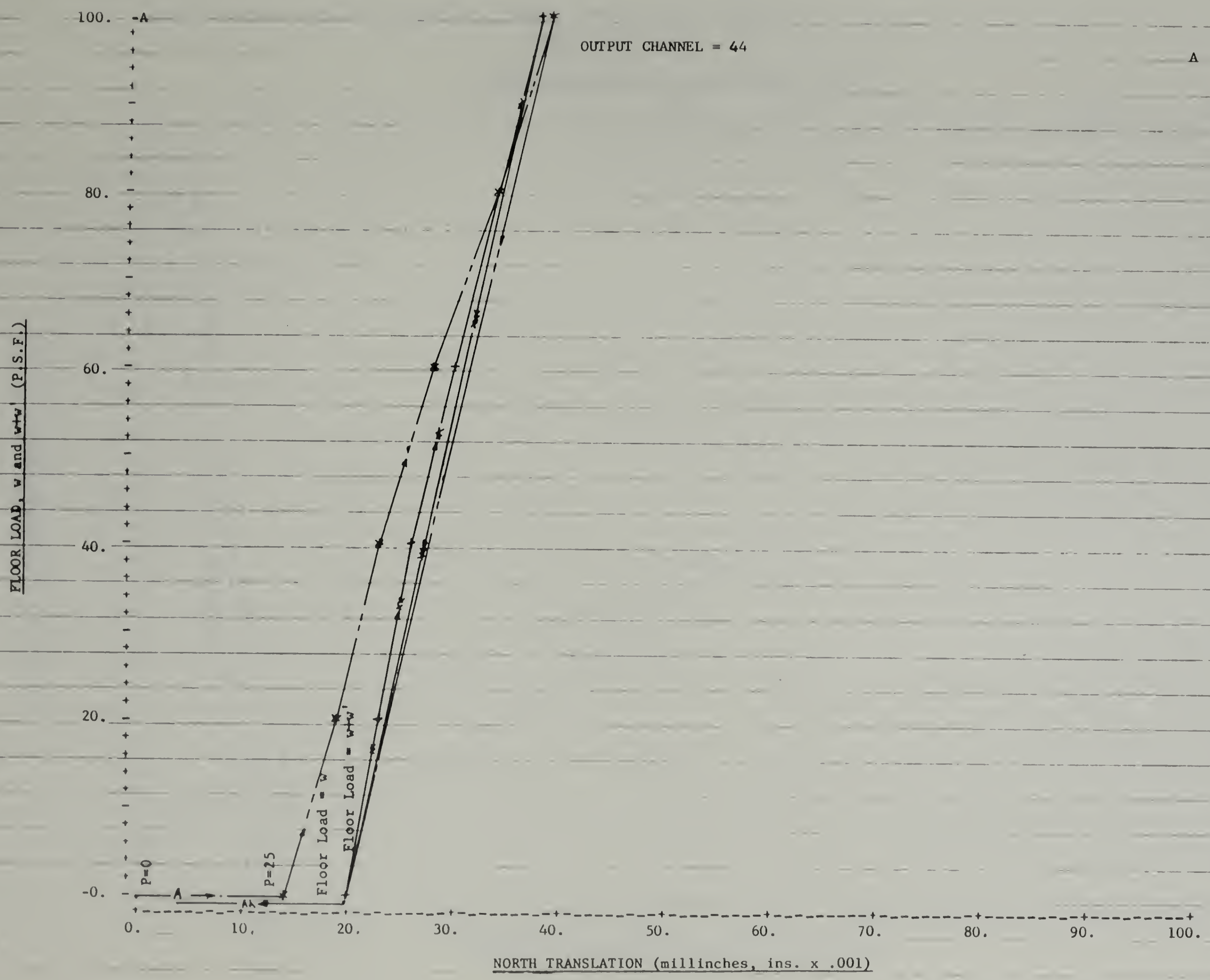


FIGURE NO. A.57 - TEST NO. 12A and 13A, VERTICAL LOAD VS. NORTH TRANSLATION

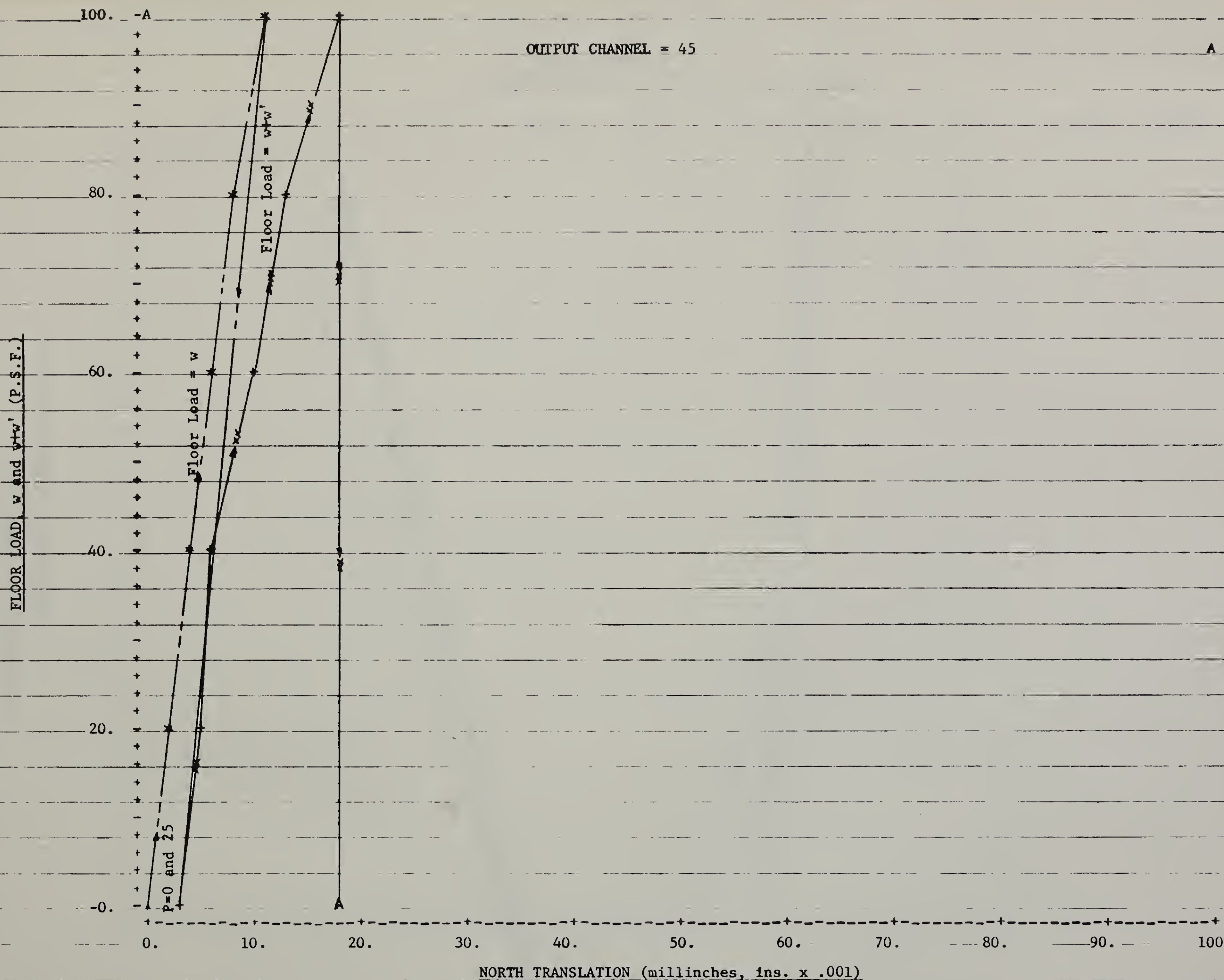


FIGURE NO. A.58 - TEST NO. 12A AND 13A, VERTICAL LOAD VS. NORTH TRANSLATION

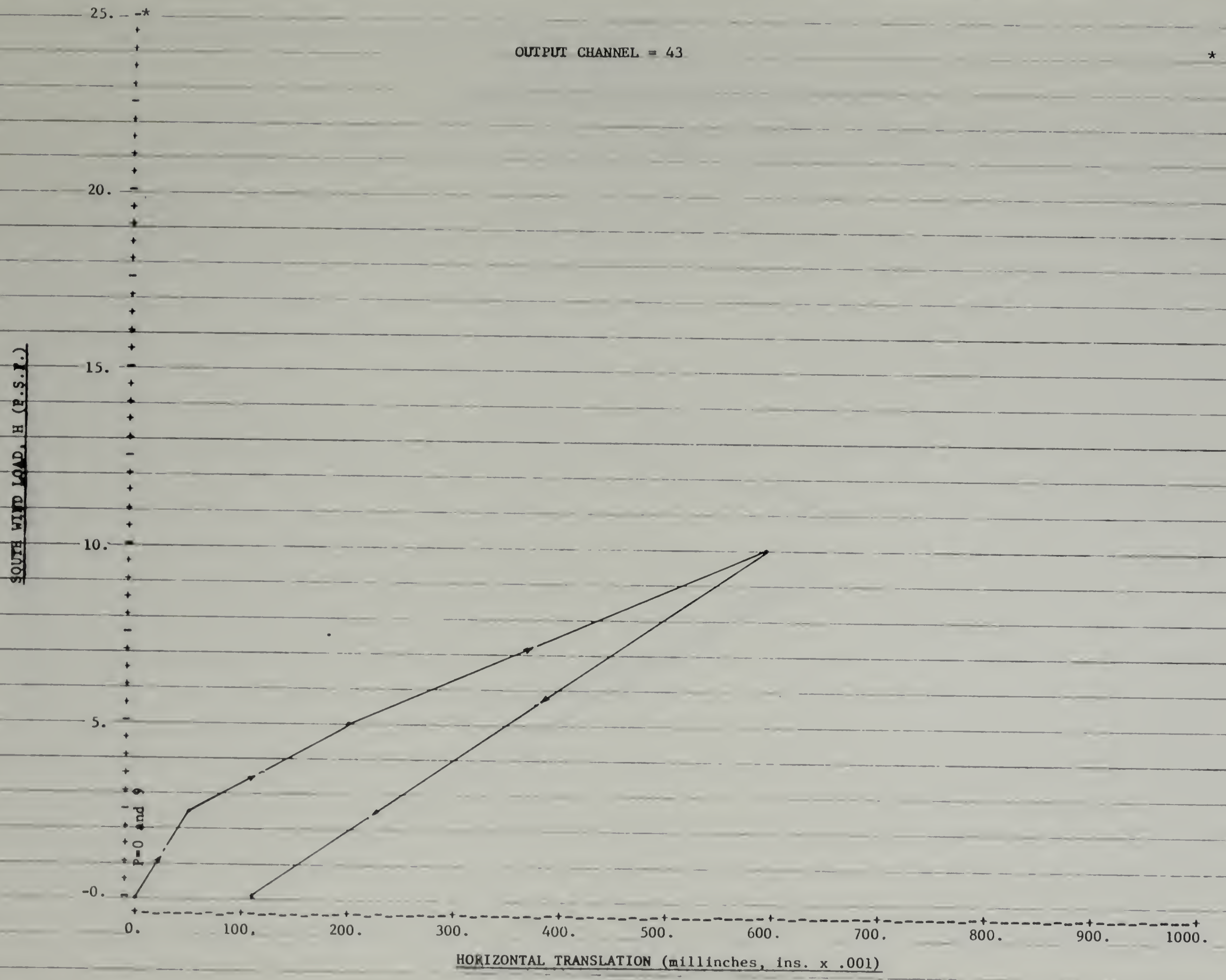


FIGURE NO. A.59 - TEST NO. 14, SOUTH WIND LOAD VS. TRANSLATION

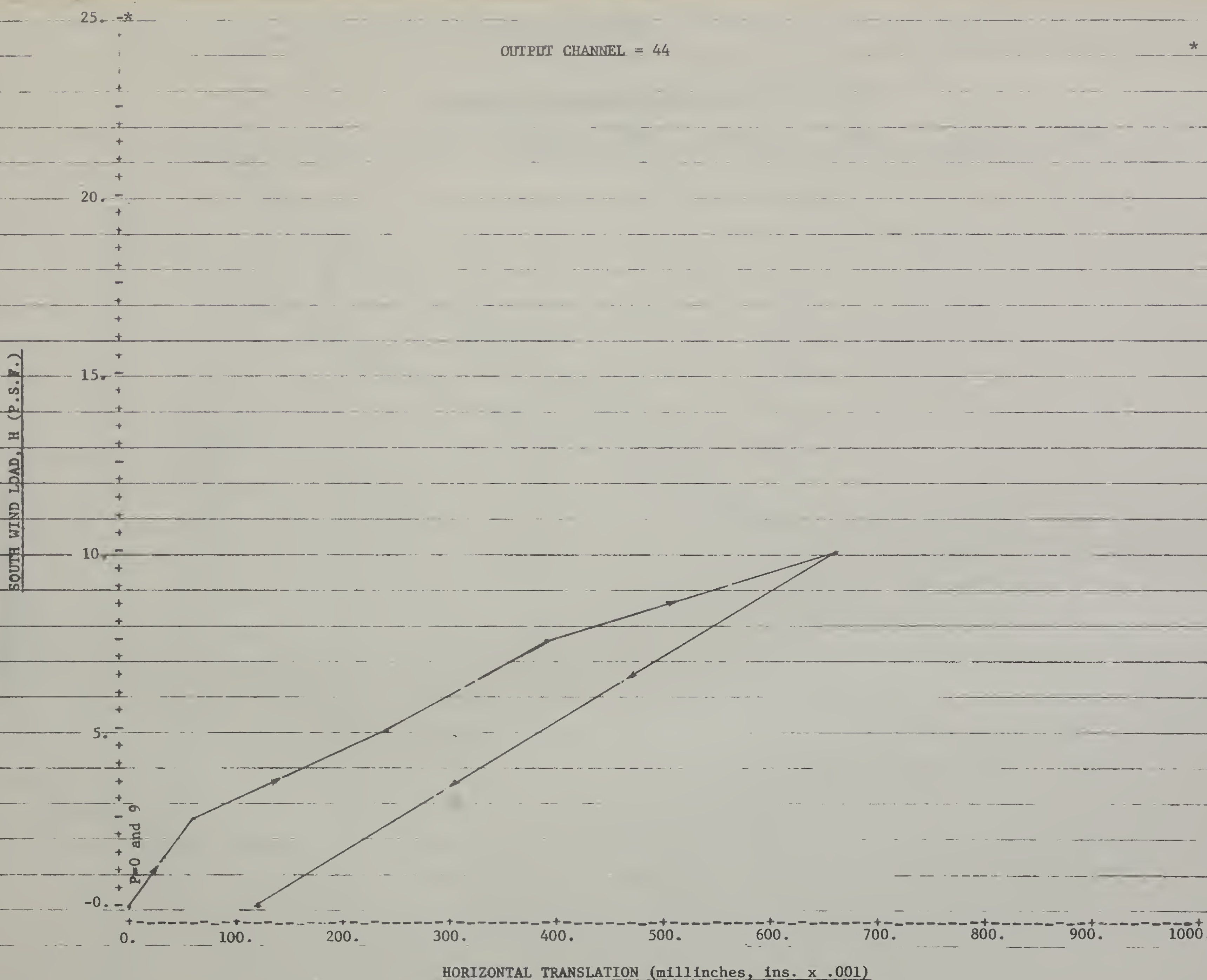


FIGURE NO. A.60 - TEST NO. 14, SOUTH WIND LOAD VS. TRANSLATION

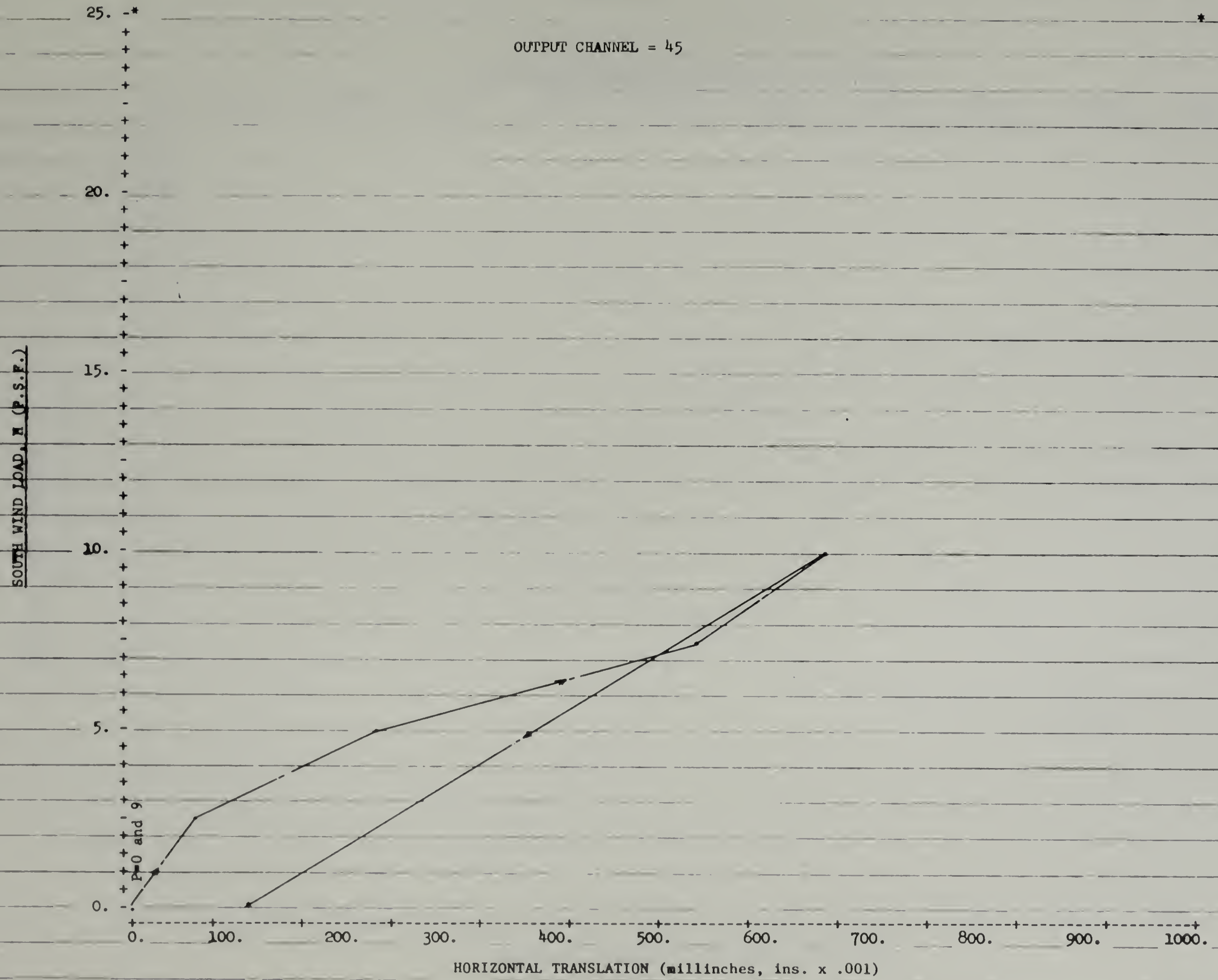


FIGURE NO. A.61 - TEST NO. 14, SOUTH WIND LOAD VS. TRANSLATION

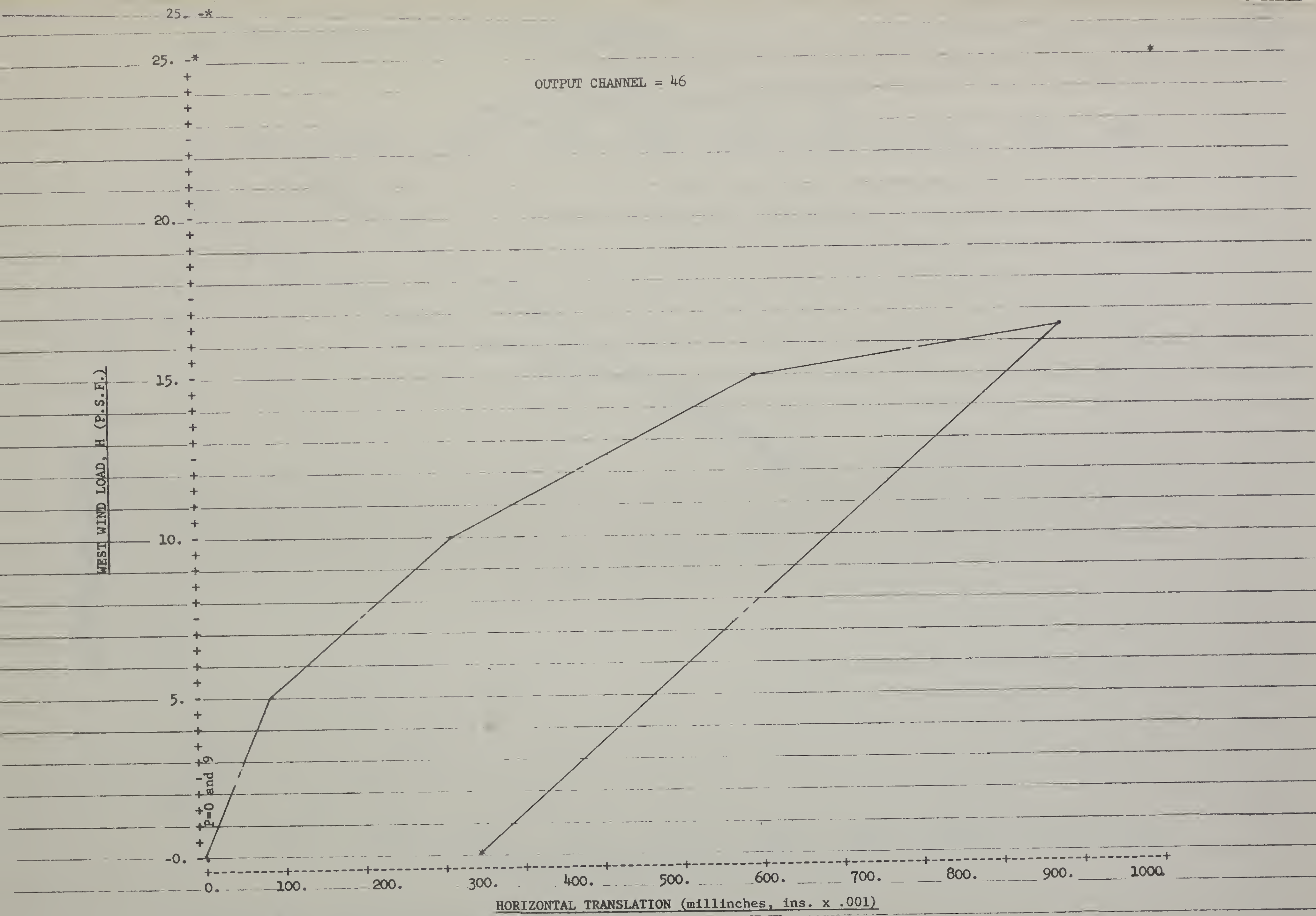


FIGURE NO. A.62 - TEST NO. 15, WEST WIND LOAD VS. TRANSLATION

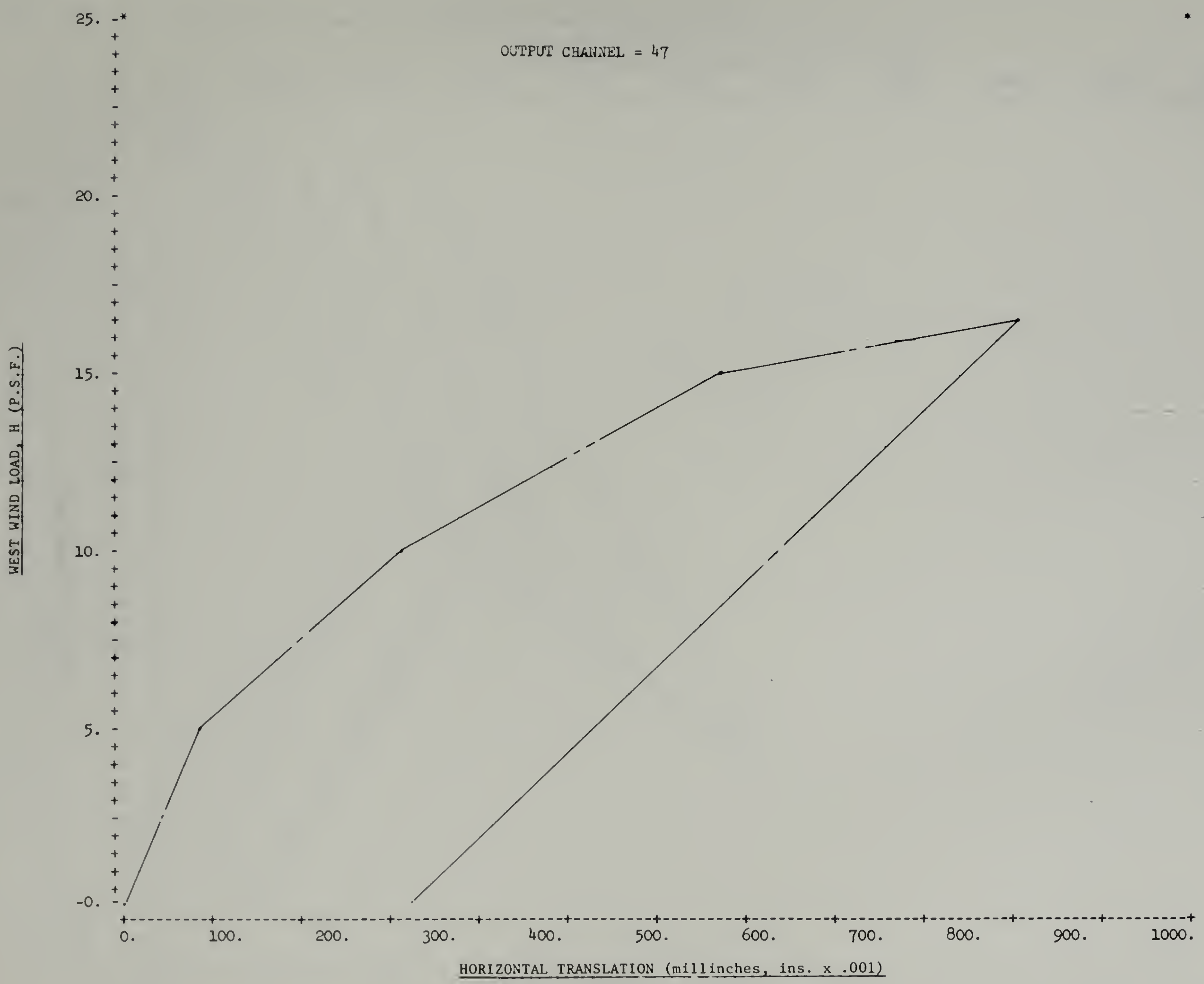


FIGURE NO. A.63 - TEST NO. 15, WEST WIND LOAD VS. TRANSLATION

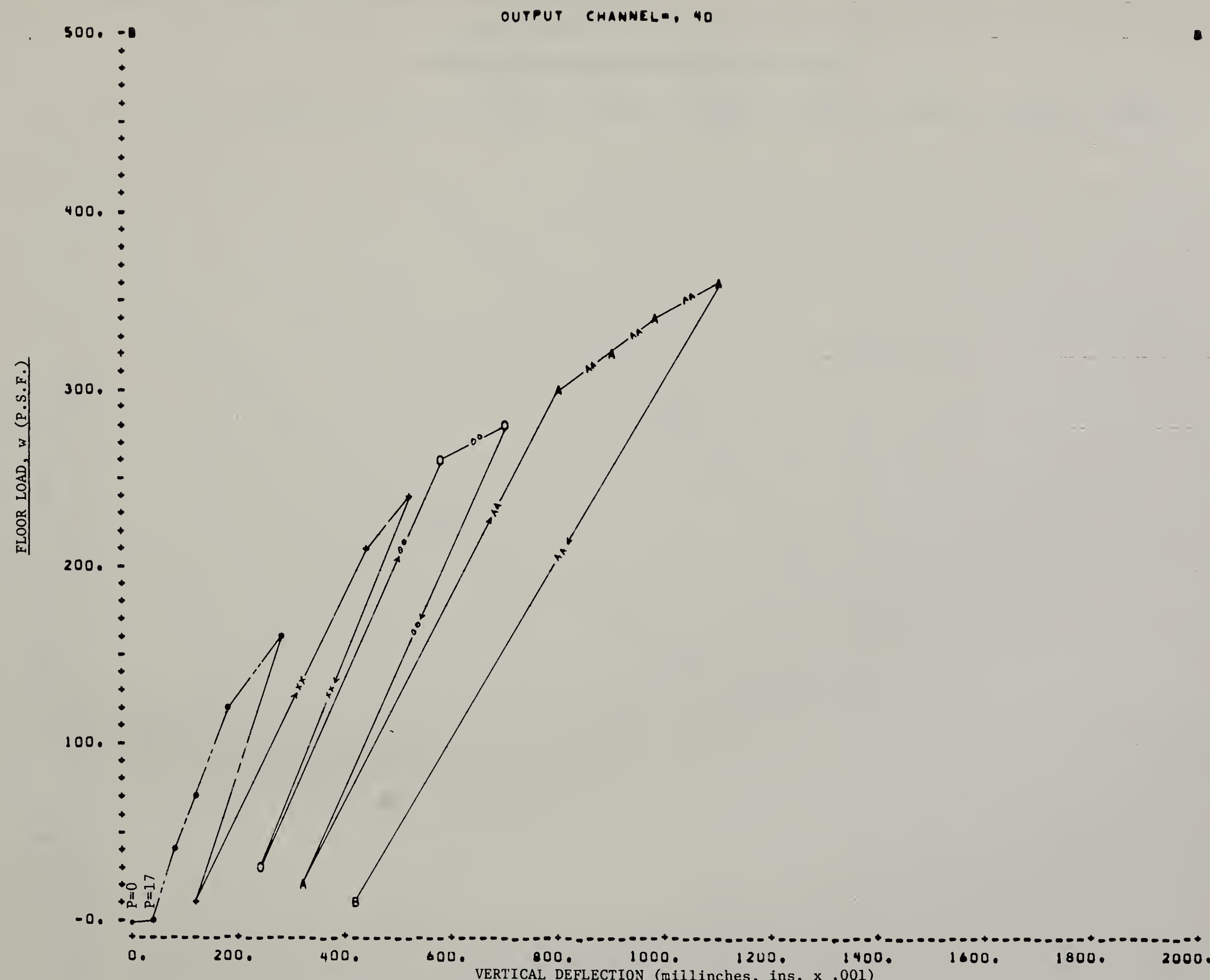


FIGURE NO. A.64 - TEST NO. 16, MAJOR FLOOR LOAD VS. BEAM DEFLECTION

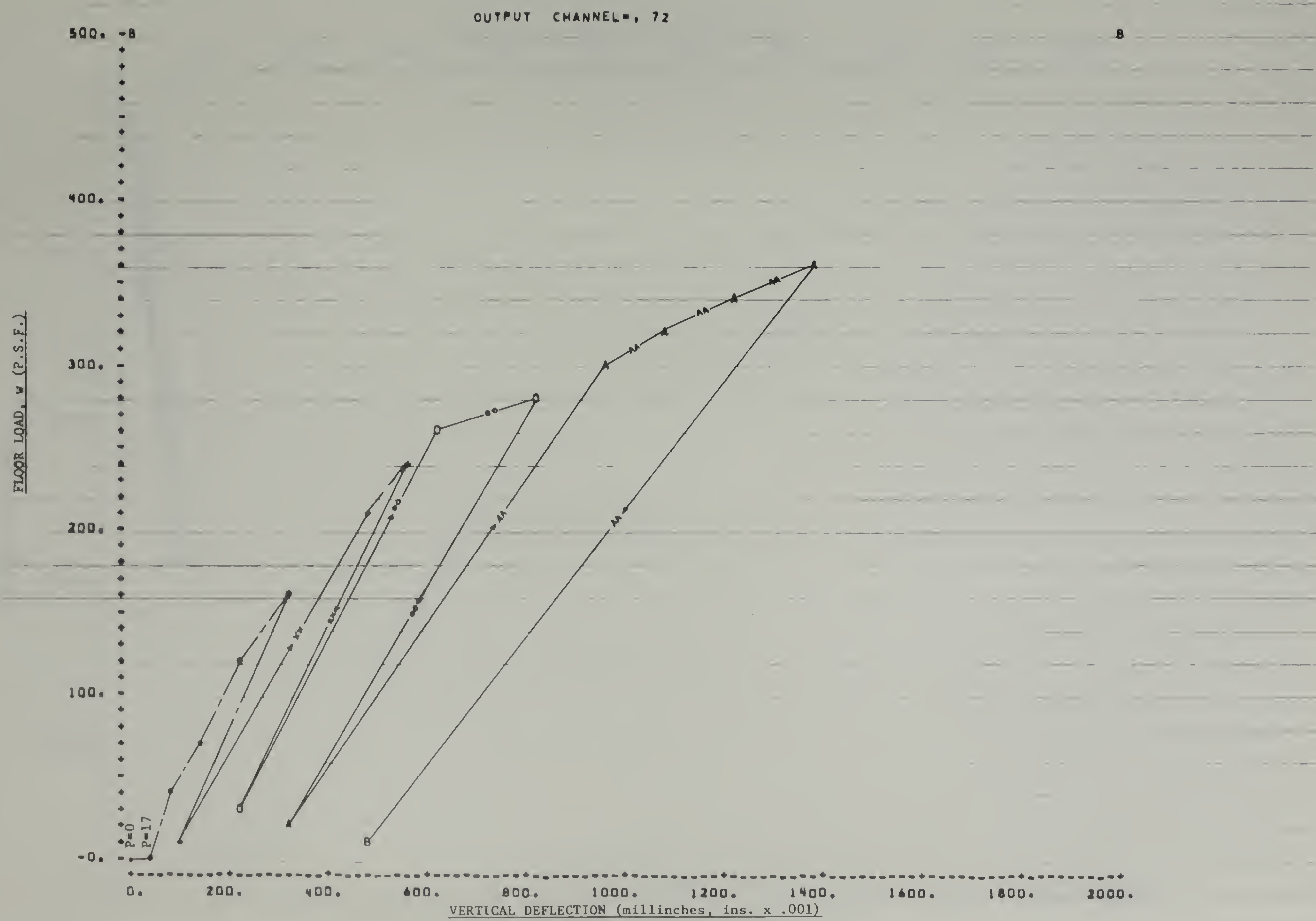


FIGURE NO. A.65 - TEST NO. 16, MAJOR FLOOR LOAD VS. BEAM DEFLECTION

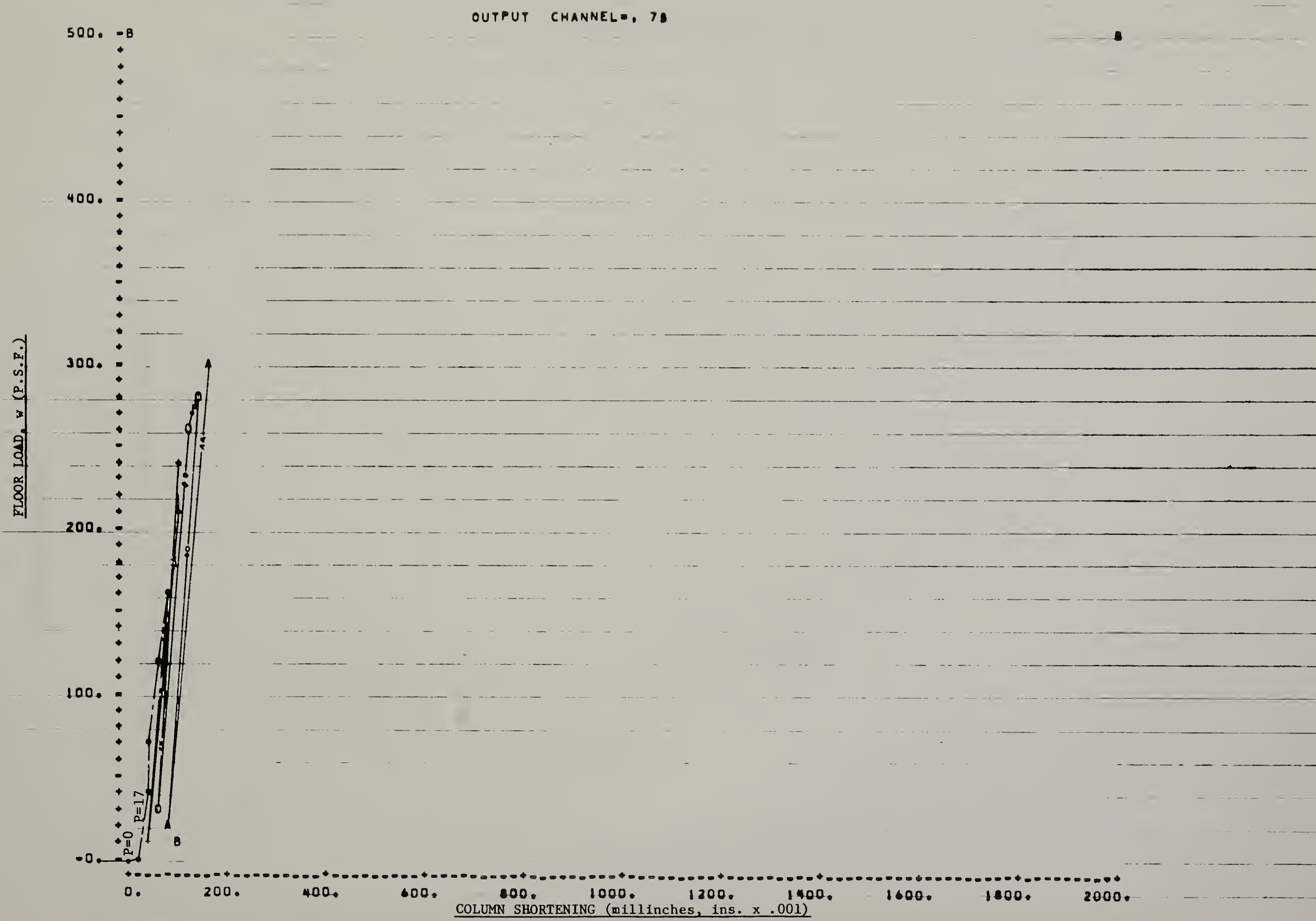


FIGURE NO. A.66 - TEST NO. 16, MAJOR FLOOR LOAD VS. COLUMN SHORTENING

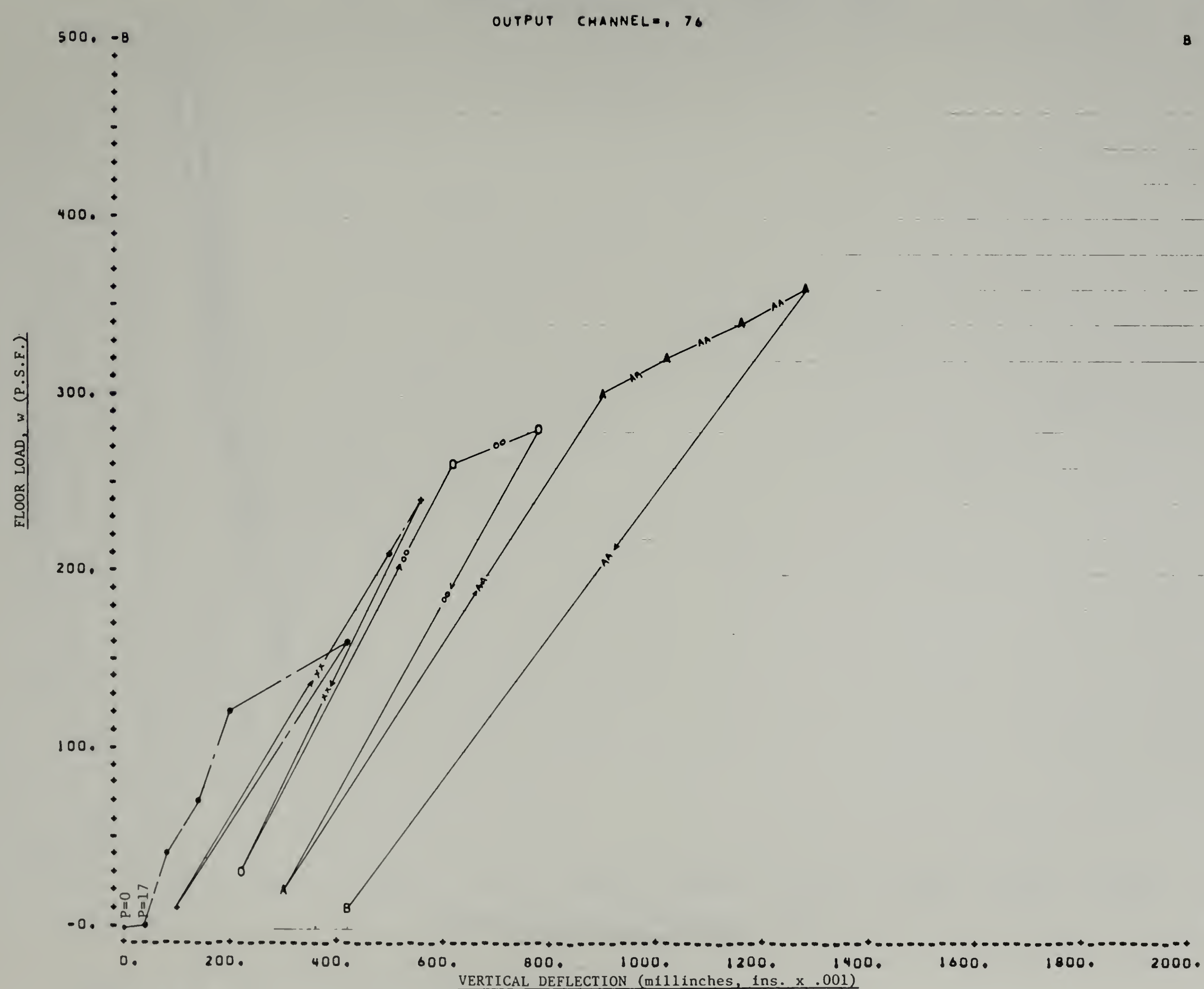


FIGURE NO. A.67 - TEST NO. 16, MAJOR FLOOR LOAD VS. SLAB DEFLECTION

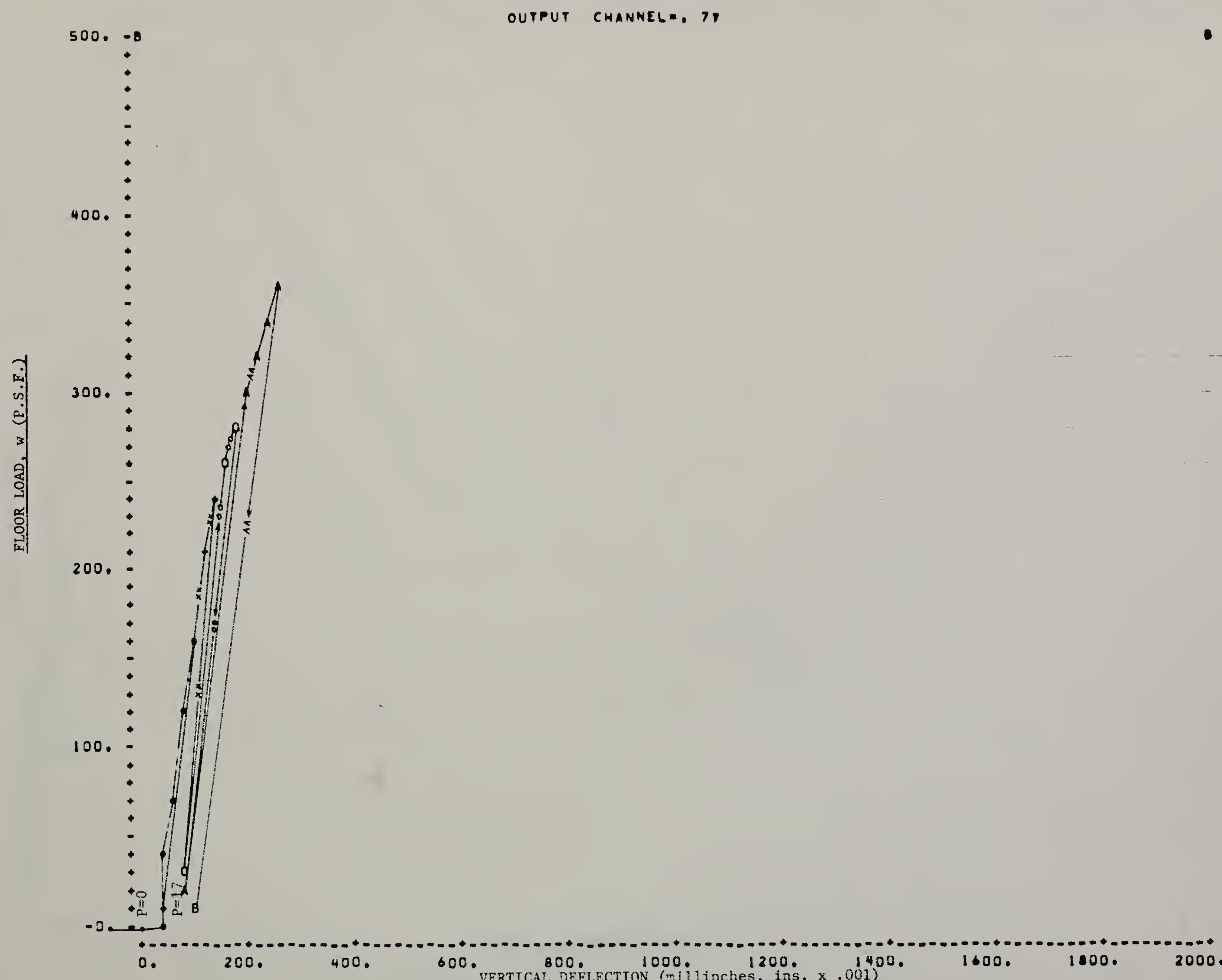


FIGURE NO. A.68 - TEST NO. 16, MAJOR FLOOR LOAD VS. SLAB DEFLECTION

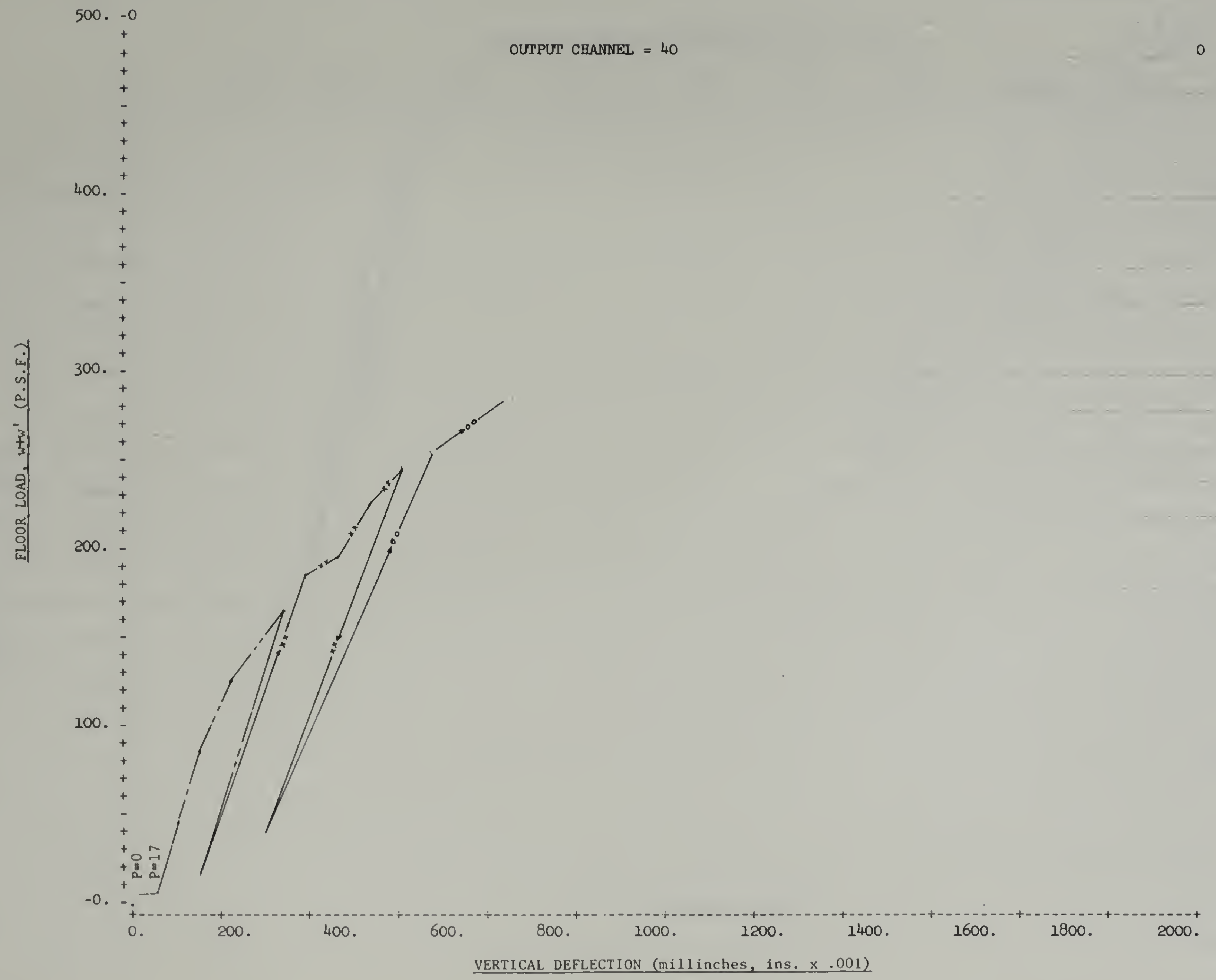


FIGURE NO. A.69 - TEST NO. 16A, FLOOR LOADS VS. BEAM DEFLECTION

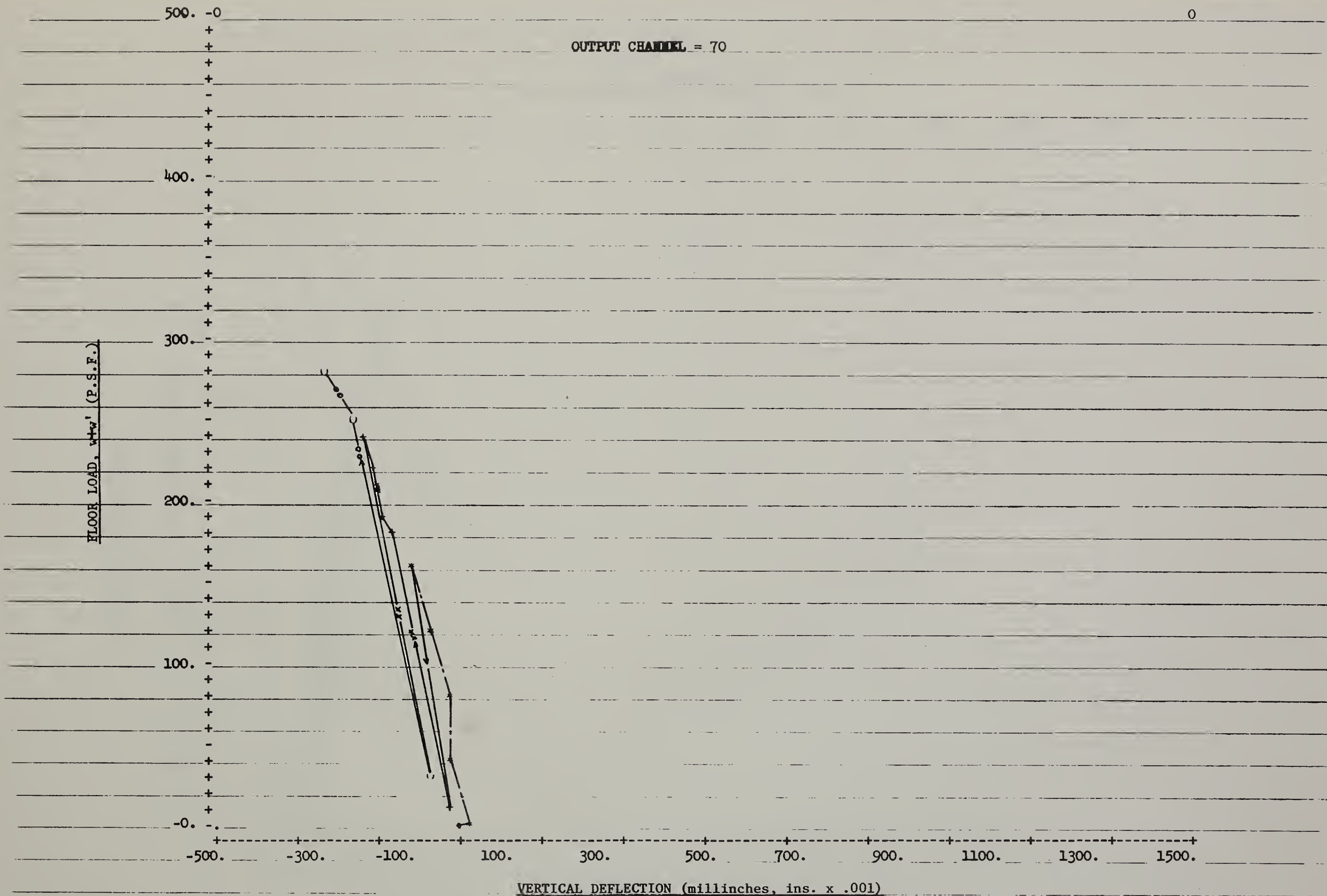
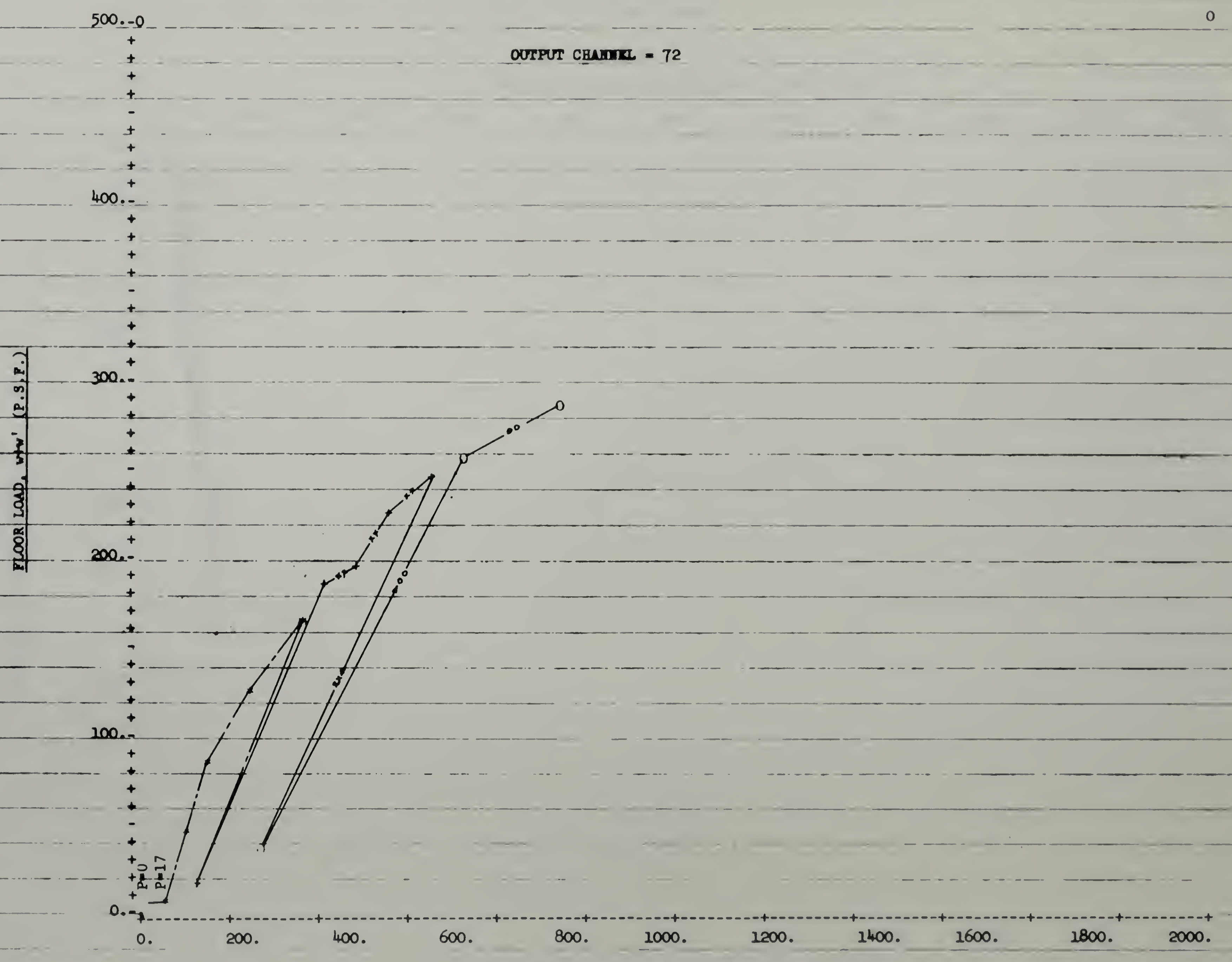


FIGURE NO. A.70 - TEST NO. 16A, FLOOR LOADS VS. BEAM DEFLECTION



OUTPUT CHANNEL = 72
FLOOR LOAD, w+w' (P.S.F.)
500.
400.
300.
200.
100.
0.
P=0
P=17
0.
200.
400.
600.
800.
1000.
1200.
1400.
1600.
1800.
2000.
VERTICAL DEFLECTION (millinches, ins. x .001)

OUTPUT CHANNEL = 73

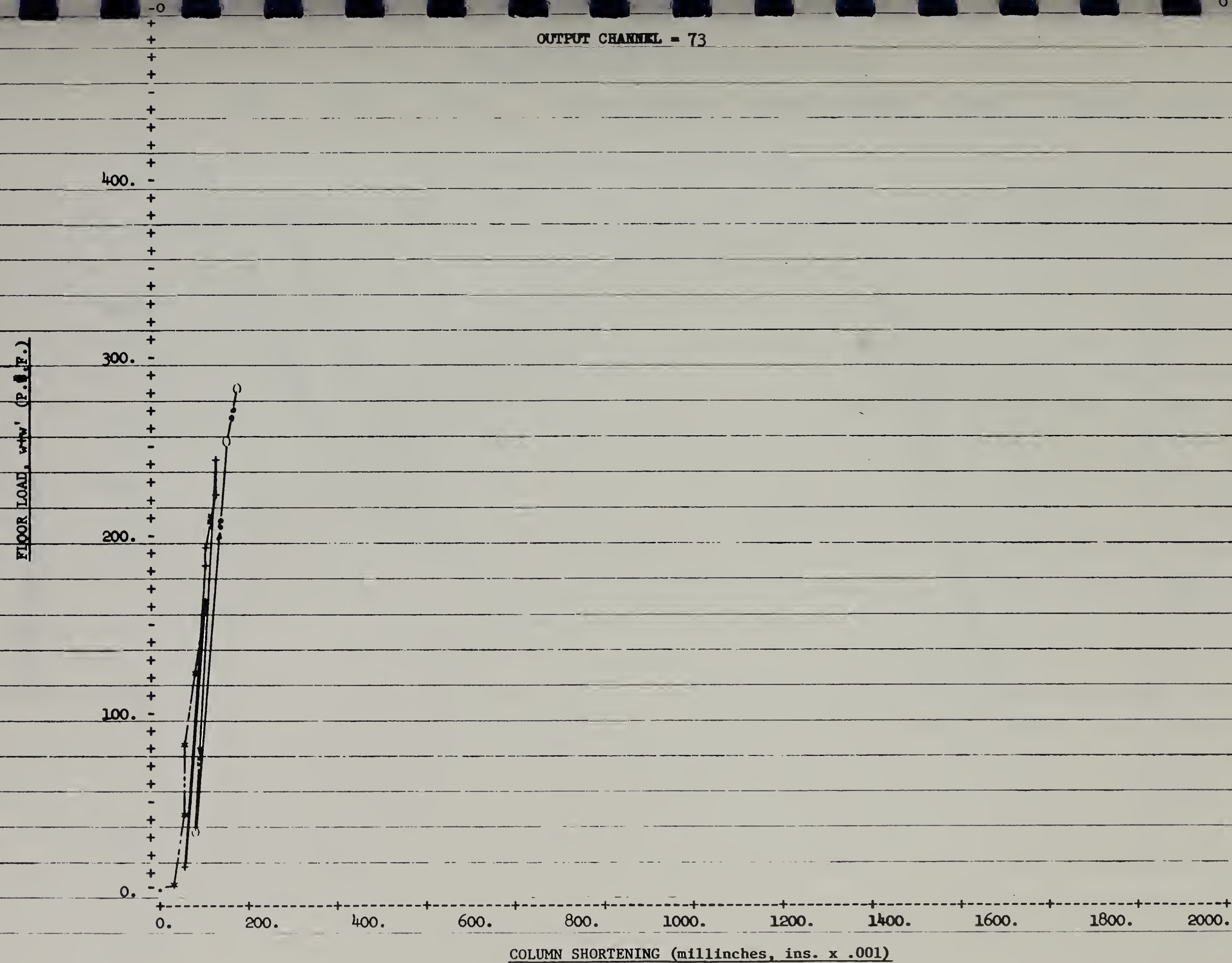
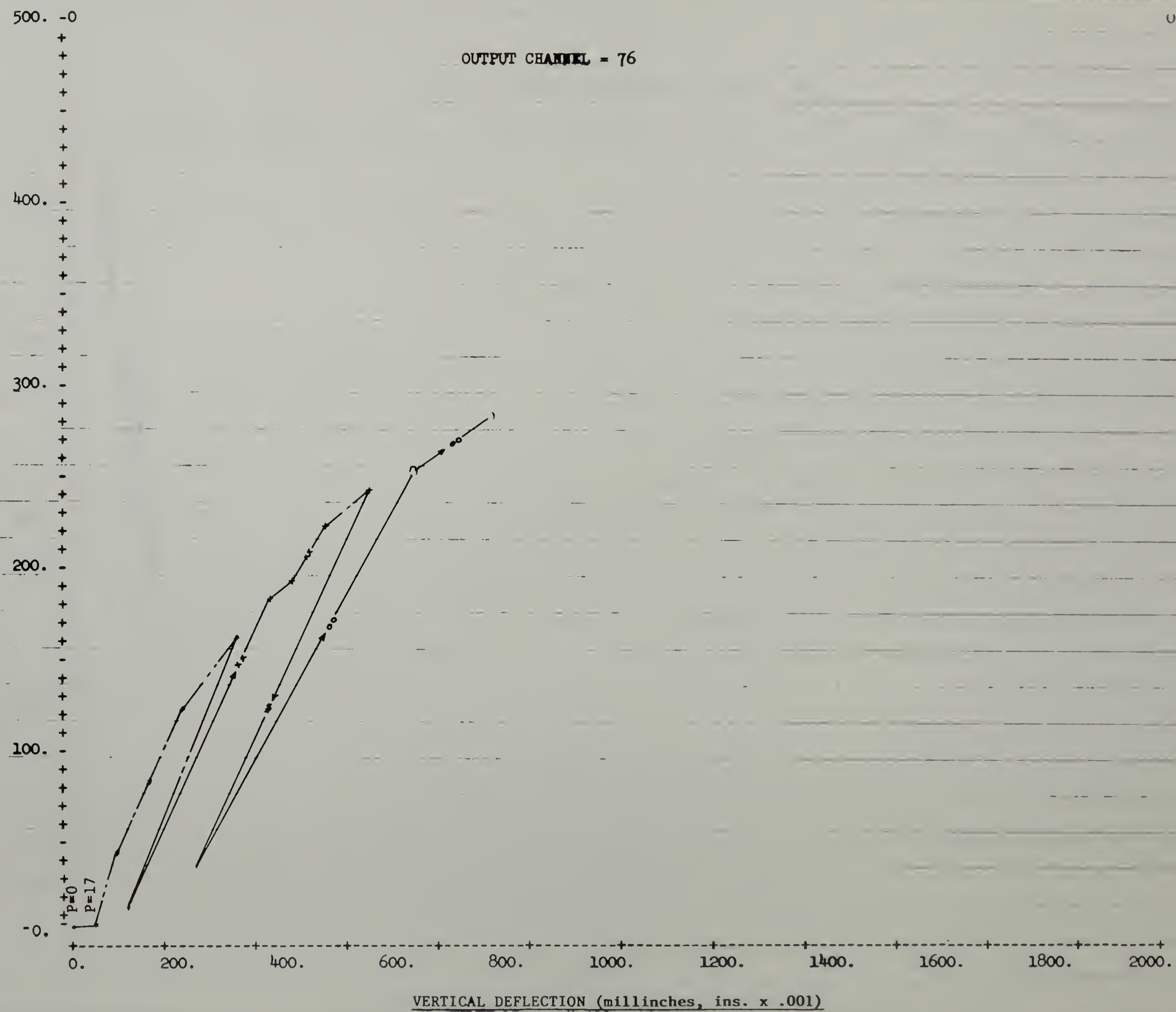


FIGURE NO. A.72 - TEST NO. 16A, FLOOR LOADS VS. COLUMN SHORTENING


OUTPUT CHANNEL = 76
FLOOR LOADS, w+w' (P.S.F.)
500. -0
400.
300.
200.
100.
-0.
P=0
P=17
0.
200.
400.
600.
800.
1000.
1200.
1400.
1600.
1800.
2000.
VERTICAL DEFLECTION (millinches, ins. x .001)

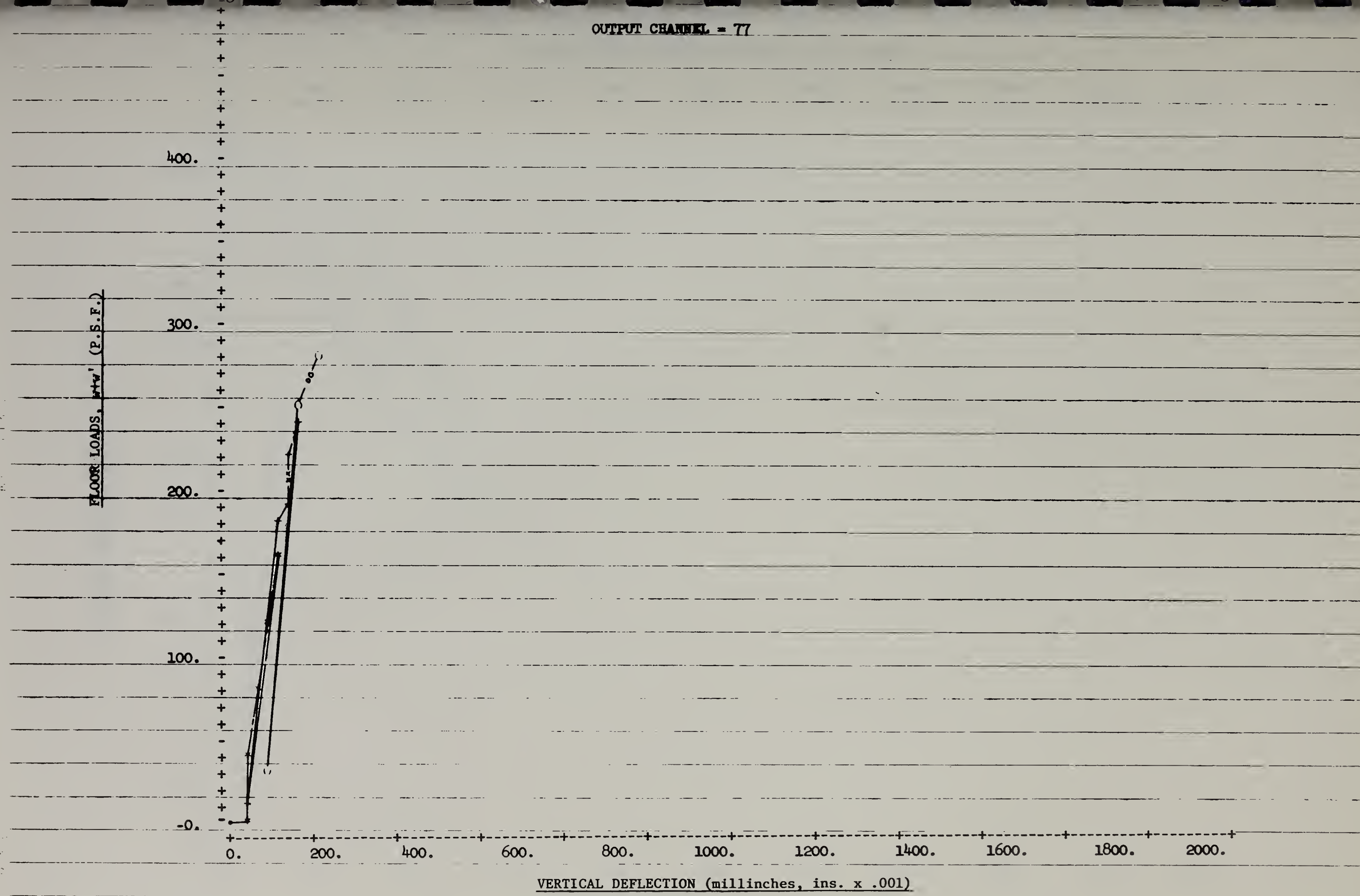
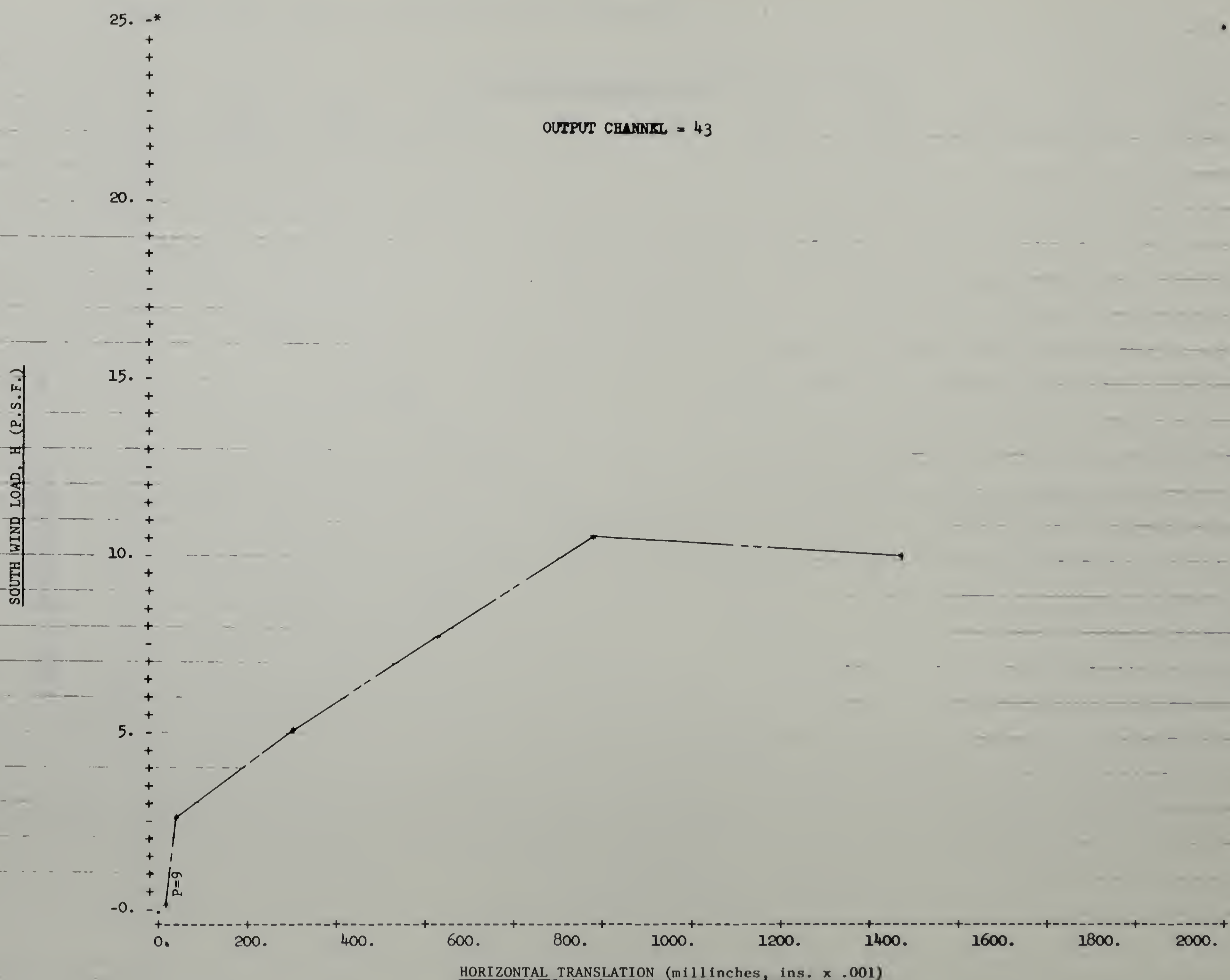


FIGURE NO. A.74 - TEST NO. 16A, FLOOR LOADS VS. BEAM DEFLECTION


OUTPUT CHANNEL = 43
SOUTH WIND LOAD, H (P.S.F.)
25.
20.
15.
10.
5.
-0.
P=9
0.
200.
400.
600.
800.
1000.
1200.
1400.
1600.
1800.
2000.
HORIZONTAL TRANSLATION (millinches, ins. x .001)

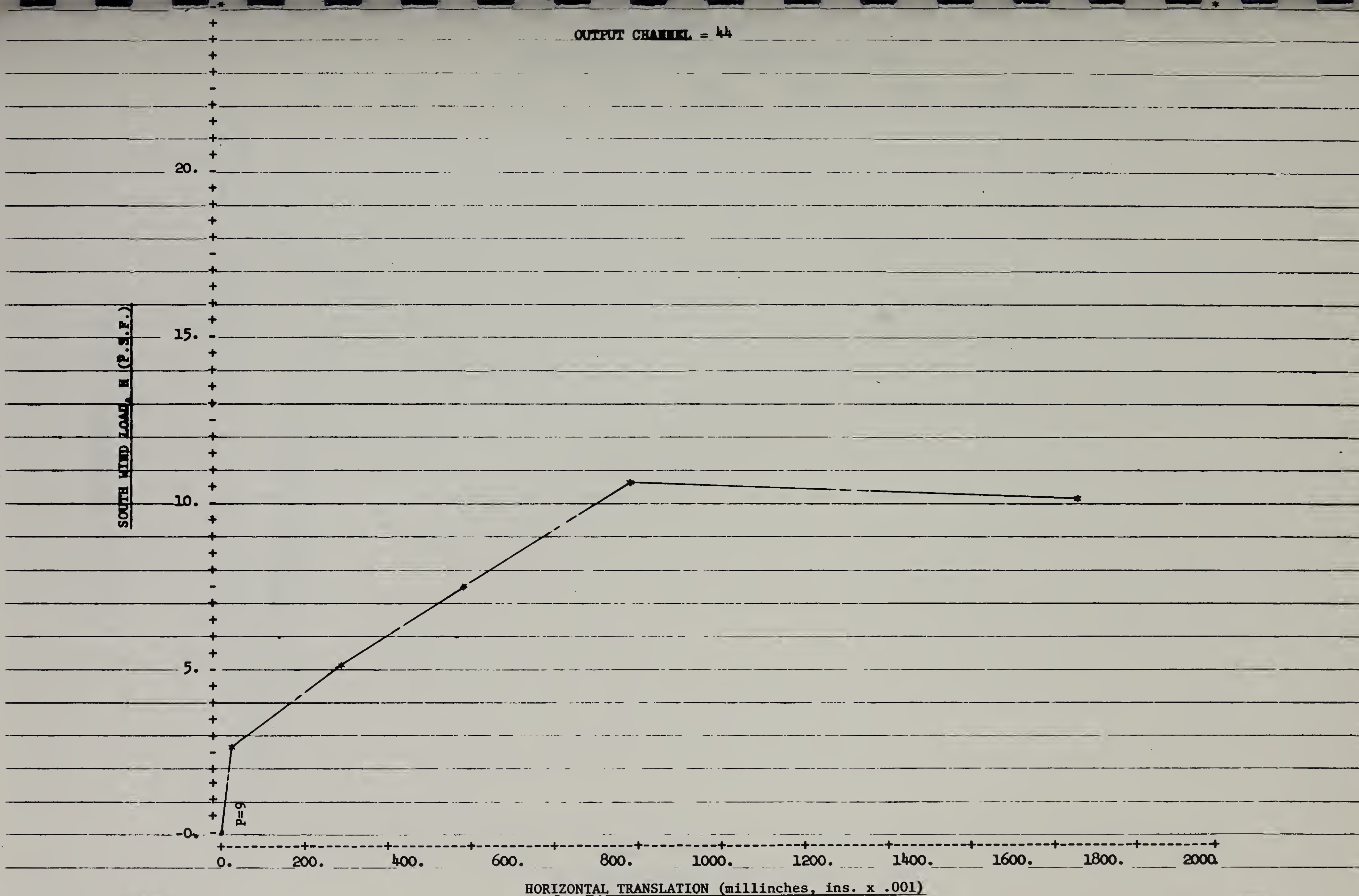
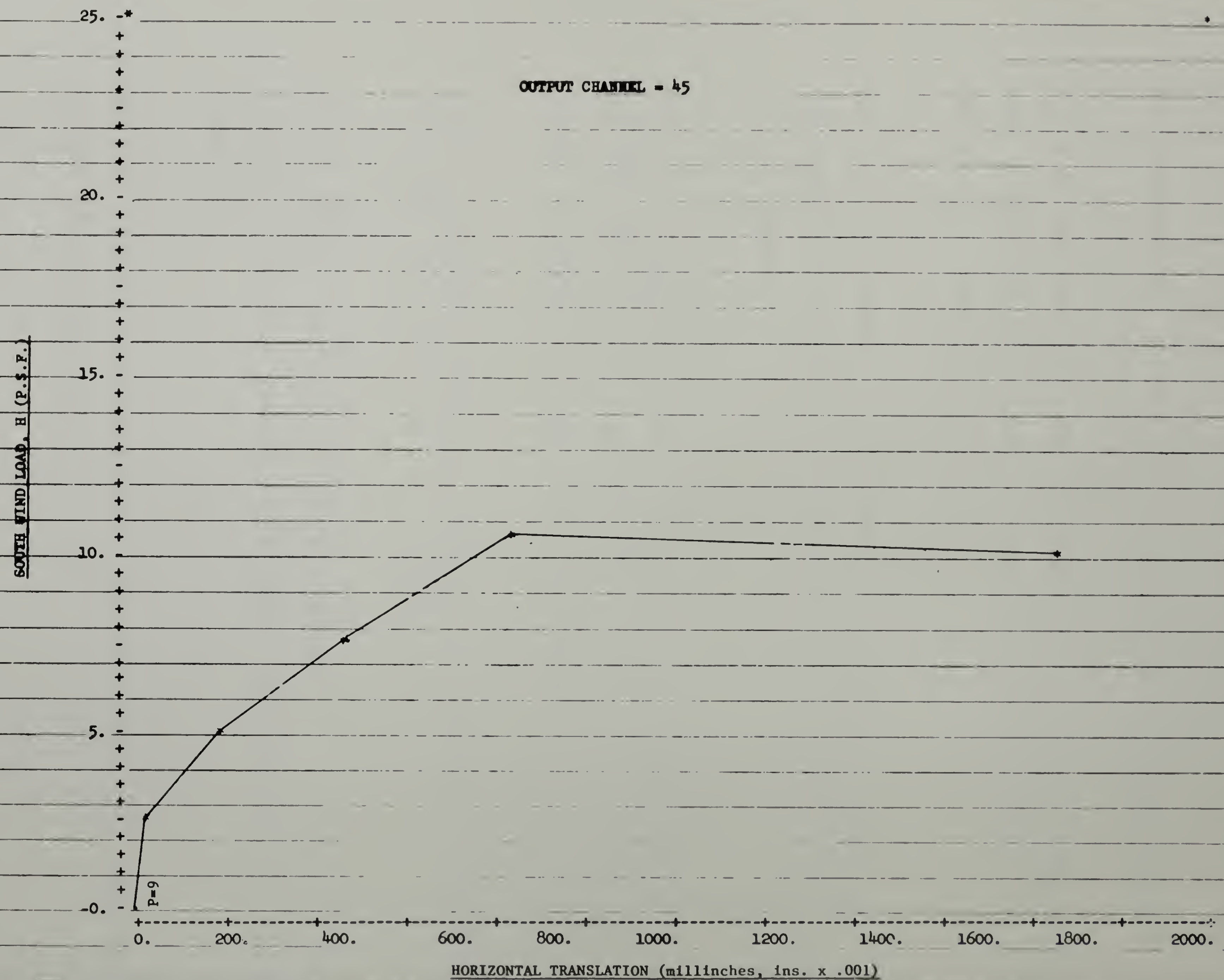


FIGURE NO. A.76 - TEST NO. 18, SOUTH WIND LOAD VS. TRANSLATION


OUTPUT CHANNEL = 45
SOUTH WIND LOAD, H (P.S.F.)
25.
20.
15.
10.
5.
-0.
P=9
0.
200.
400.
600.
800.
1000.
1200.
1400.
1600.
1800.
2000.
HORIZONTAL TRANSLATION (millinches, ins. x .001)

Acknowledgement

The contribution of the following persons is acknowledged:

Frank A. Rankin and James W. Raines, Engineering Technicians, were respectively in charge of erection and electronic instrumentation of the test structure.

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

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

Robert G. Mathey, Assistant Chief of the Structures Section, supervised and coordinated the testing effort.

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

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

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_3^1 in Fig. B.1).

To summarize, the upper story column load simulation, as applied, tends to increase the column moment, while the foundation simulation tends to decrease it. The net effect in the case of this test structure is illustrated in Figure B.2. Figure B.2 (a) shows the moment distribution in a three story

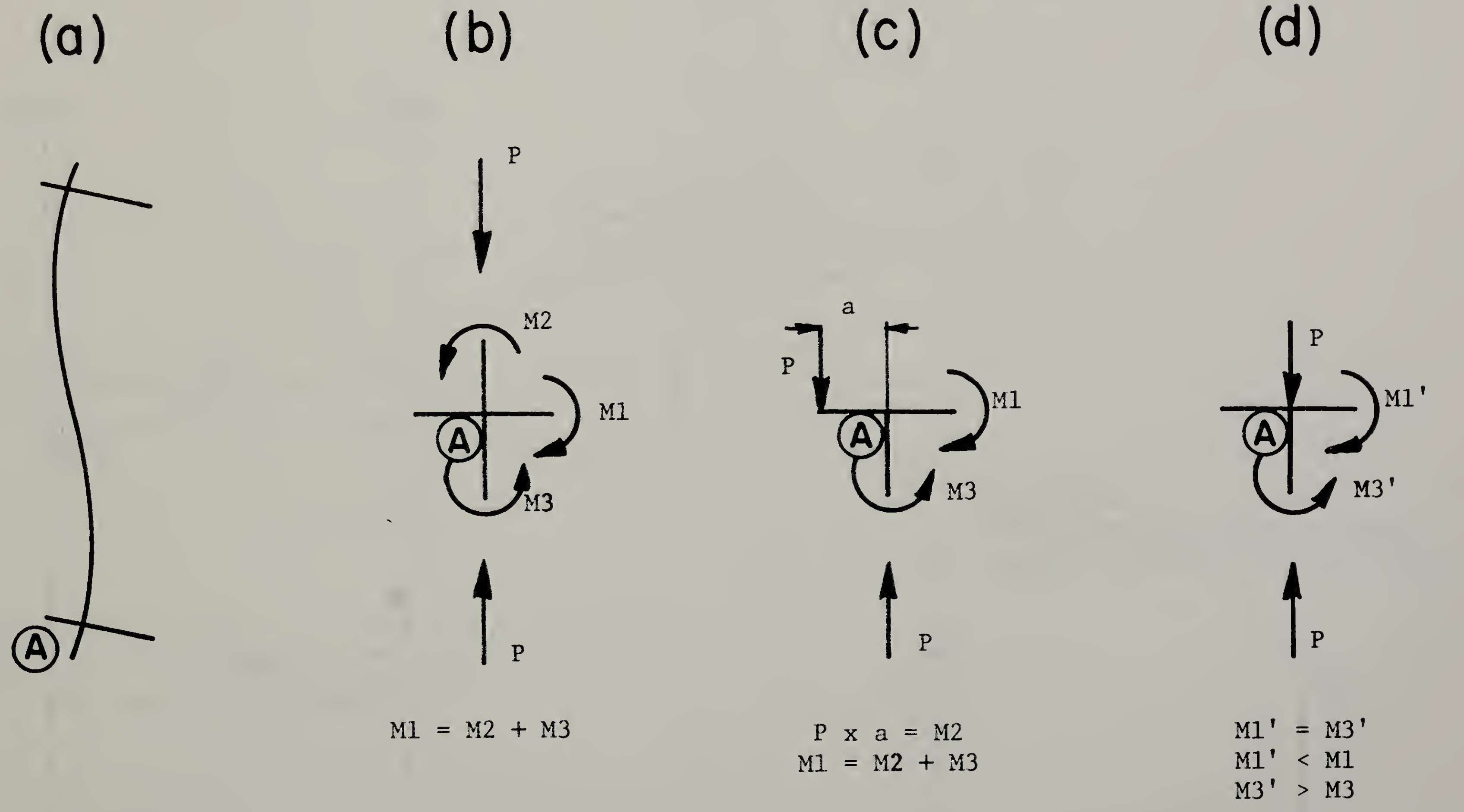


FIG. B.1 COLUMN SIMULATION EFFECT ON CONNECTION MOMENTS structure (comprising half of the real structure) caused by one (1) live load applied to the column strip. Figure B.2 (b) shows the moment distribution due to the same loading in the test structure.

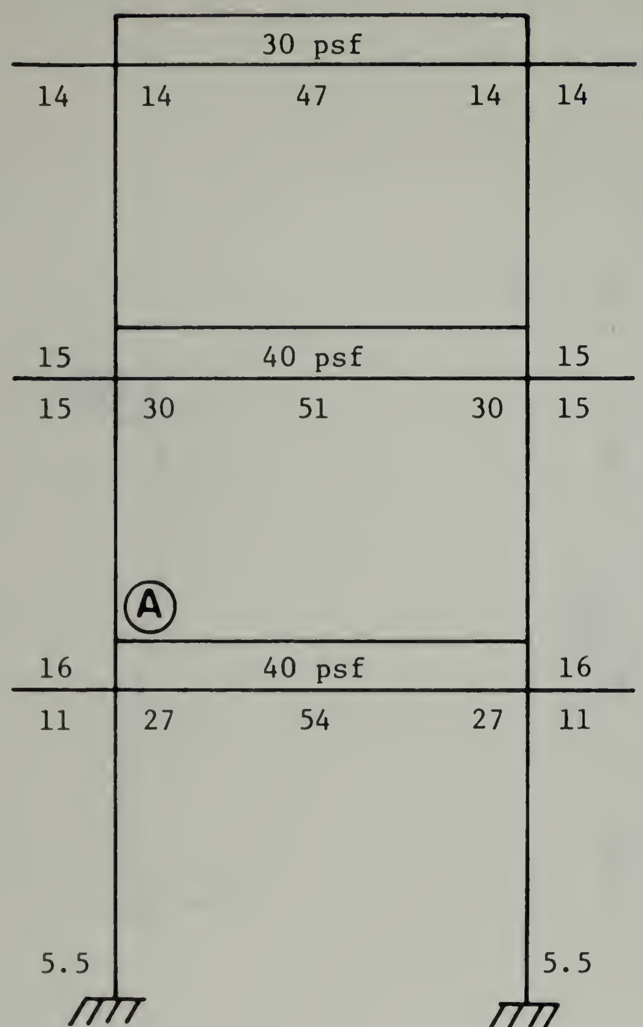
By comparing joint "A" in Figure B.2 (a) and Figure B.2 (b), it may be seen that at joint "A" in the test structure the column moment is <u>increased</u>. Also, the negative main beam moment is <u>decreased</u>, while the positive main beam moment at the center of the span is <u>increased</u>. Thus,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

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

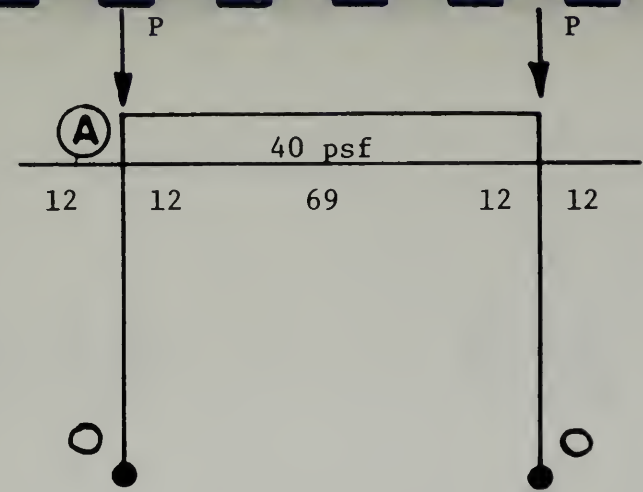
(2) Produced maximum dead and live load deflections and maximum positive beam moments, since midspan deflection and moment increases with decreasing negative moment at beam ends.

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

APPENDIX C



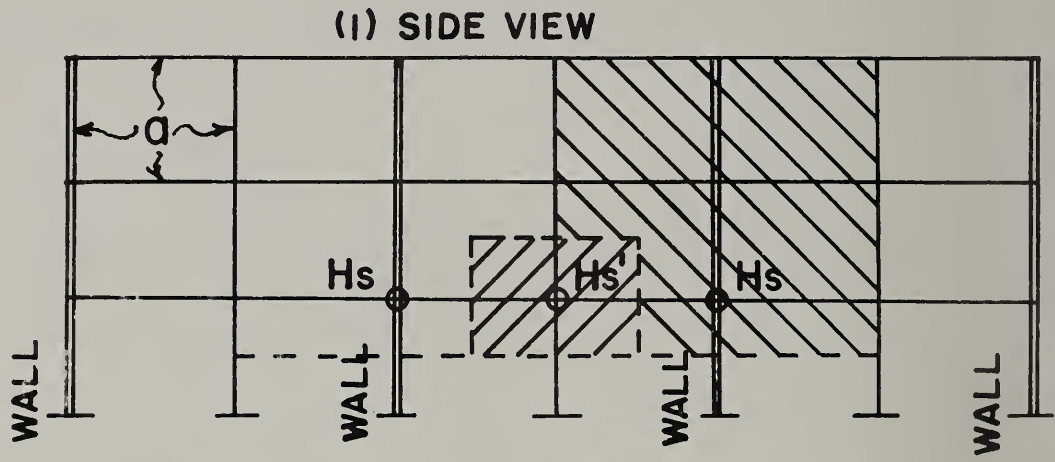
(a) Moment Distribution (kip-ins.) due to one live load on three story structure.



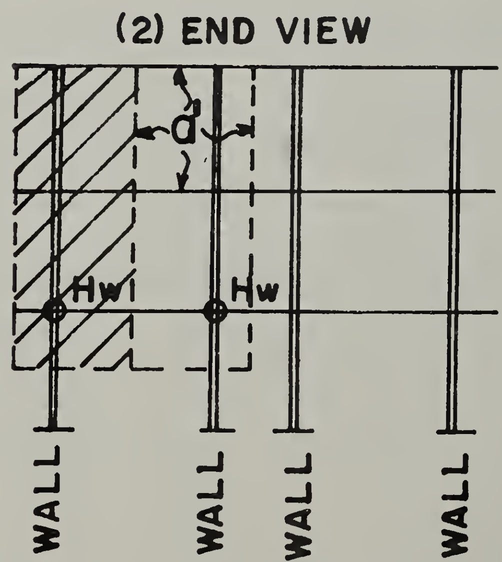
(b) Moment Distribution (kip-ins.) due to one live load on test structure.

FIG. B.2 MOMENT DISTRIBUTION

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



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



AREA TRIBUTARY TO Hw = 2.5a'

FIGURE C.1 — WIND LOADS

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 Structures

	D	L	.9D	H	.8H	1.1H	1.6H	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	4.5	5.7			
H_s'-kips				0.9	0.7	1	1.3			
Hw-kips				2.05	1.5	2.3	2.9			
w or w' - (psf)	9.3*	43						52	100**	145**

*Allowance for dead weight of partitions and fixtures.
**Includes dead weight of test structure (i.e., 43 x 1.07) multiplied by appropriate factor.

(b) Simulated Loads on Lower Column for Component Tests of Columns

	D+L	.9D+L	1.3D+1.7L	1.25 (1.5D+1.8L)	D	L
P-kips	25	24	37	51	14.4	11

(c) Simulated Load on Main Beam for Fatigue Test in Kips per Ram, See Figure 10.23

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

(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: $1D + 1L = 25^k$

$1.3D + 1.7L = 37^k$

$1.25(1.5D+1.8L) = 51^k$

*42.8 x 1.07 = 46

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: $2.25'' \times \frac{1'}{12''} \times 110 \ \#/ft^3$* = 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: $$\frac{(6\text{x}4.75)^{\text{in}^2}}{144 \frac{\text{in}^2}{\text{ft}^2}} \times 2 \times 7.83^{\text{ft}} \times 100^{\#/\text{ft}^2} \times \frac{1}{200} \text{sf} = 1.55 \text{ psf}$$

(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

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

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

$\underline{a'}$ = 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: $\underline{Hw} = 1/2 \times (2.5\text{x}82)^{sf}\text{x}20^{\#/sf} = 2{,}050\text{lb.} = \underline{2.05 \text{ kips}}$.

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, Tables 6.1 and 6.2, Appendix C(1).

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

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

Start of test:	5/10/68	9:09 A.M.
Completion:	5/10/68	9:41 A.M.
Loading:	$P = 0.9D - 9^{k}$	
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.

(7) Main beam loads for fatigue test.

(a) 1/4 point loading: (1/2 the load resting on the beam, to be applied by each ram)

Area: $1/2 \times 12.5^{ft} \times 10.385^{ft}$ = 65 sf

1 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 6.25 ft. x 9' = 920# See (2)

Walls, etc: 8.75 psf x 65 sf = 570 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

IV. Wind Loads

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

a = Story height x panel width = 8.62 x 10.39 = 90 sf

@ 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: $Hs = 1/2 \times (4.5\times90)ft^2 \times 20\#/ft^2 = 4{,}050$ 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 not too low, considering the fact that a service wind load of 20 psf is extremely conservative for a built up area.

(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.

(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.

(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	

(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 + 3.5L)

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

Notes: (1) Four cycles of loading and unloading were applied for w and w' at progressively higher loads.

(2) w and w' + w were applied 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.

(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	

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.

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.

DEPARTMENT OF COMMERCE ★ NATIONAL BUREAU OF STANDARDS ★