NBS BUILDING SCIENCE SERIES 95

U.S. DEPARTMENT OF COMMERCE / National Bureau of Standards

The Development of an Improved Compression Test Method for Wall Panels

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NBS Building Science Series

C.W.C. Yancey
L. E. Cattaneo

Center for Building Technology
National Engineering Laboratory
National Bureau of Standards
Washington, D.C. 20234

Prepared for
Division of Energy, Building Technology and Standards
Office of Policy Development and Research
Department of Housing and Urban Development
Washington, D.C. 20410

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Jordan J. Baruch, Assistant Secretary for Science and Technology
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SI Conversion Units

In view of the present accepted practice in this country for building technology, common U.S. units of measurement have been used throughout this publication. In recognition of the position of the United States as a signatory to the General Conference on Weights and Measures, which gave official status to the metric SI system of units in 1960, appropriate conversion factors have been provided in the table below. The reader interested in making further use of the coherent system of SI units is referred to:

NBS SP 330, 1972 Edition, "The International System of Units"

ASTM E380-75/IEEE Std. 268-1976 ASTM/IEEE Standard Metric Practice

Table of Conversion Factors to Metric (S.I.) Units

Physical Quantity	To convert from	to	multiply by
Length	inch	meter	$2.54^* \times 10^{-2}$
	foot	m	$3.048^* \times 10^{-1}$
Area	$inch^2$	m^2	$6.4516^* \times 10^{-4}$
	$foot^2$	m^2	9.290×10^{-2}
Volume	$inch^3$	m^3	1.639×10^{-5}
	$foot^3$	m^3	2.832×10^{-2}
Temperature	Fahrenheit	Celsius	$t_C = (t_F-32)/1.8$
Temperature difference	Fahrenheit	Kelvin	$\Delta t_K = (\Delta t_F)/1.8$
Pressure	inch Hg (60F)	$newton/m^2$	3.377×10^3
Mass	lbm	kg	4.536×10^{-1}
Mass/unit area	lbm/ft^2	kg/m^2	4.882
Moisture content rate	$lbm/ft^2 week$	$kg/m^2 s$	8.073×10^{-6}
Density	lbm/ft^3	kg/m^3	1.602×10
Thermal conductivity	$Btu/hr \cdot ft^2 \cdot (°F/inch)$	$\frac{W}{mK}$	1.442×10^{-1}
U-value	$Btu/hr \cdot ft^2 \cdot °F$	$\frac{W}{m^2K}$	5.678
Thermal resistance	$°F/(Btu/hr \cdot ft^2)$	$K/(W/m^2)$	1.761×10^{-1}
Heat flow	$Btu/hr \cdot ft^2$	W/m^2	3.155

*Exact value; others are rounded to four digits.

The Development of an Improved Compression Test Method for Wall Panels

by

C. W. C. Yancey and L. E. Cattaneo

Abstract

An experimental and analytical investigation of the primary factors involved in the testing of prototype wall panels under axial compression loading is reported. The objective of the investigation was to develop a method of testing wall specimens that incorporates the best features of ASTM Standard Method E 72 while at the same time incorporating improvements in the areas of deficiency in the Standard. Twenty-five laboratory tests were conducted on samples composed of five types of wall panel construction. The panels were tested to failure under either of two different eccentricities of load, while being supported with one of two types of idealized end conditions. Selected test results and detailed descriptions of the laboratory procedures used are presented. A computer-aided analytical study of the variables affecting the degree of uniformity of loading was conducted. Equations based on the analogy of beams supported on elastic foundations were used in the analysis. A study of the statistical parameters commonly used to interpret test results was conducted to establish useful guidelines for predicting structural performance on the basis of small sample test results. A compression test method applicable to traditional and innovative wall constructions is presented. The principal additions in the revised test method are as follows: (a) a provision for variable eccentricity, (b) a procedure for selecting a load distribution assembly which will be compatible with the test panel.

Keywords: Compression; eccentric loading; flat-end; kern; loading rate; pin-end; test method; wall panels; walls.

1. INTRODUCTION

1.1 Background - This study was conducted as a part of an applied research program, sponsored by the U. S. Department of Housing and Urban Development (HUD), to develop improved structural test methods for evaluating the performance of housing components. As an initial step in the program, a comprehensive survey of the literature relating to structural test methods used in evaluating walls, floors, roofs and complete buildings was conducted. A written survey was conducted among the membership of Committee E-6, On Performance of Building Constructions, American Society for Testing and Materials (ASTM) to help in identifying test method areas in need of basic research. A state-of-the-art report, Building Science Series (BSS) 58 [1][1/], was published as a result of the literature survey and written questionnaire. The information obtained from this initial effort provided the basis for several specific recommendations on fundamental studies needed to develop improved standard test methods. One of the recommendations focused on the need for a combined laboratory and analytical investigation of the section of the existing ASTM Standard Method E72 that pertains to bearing wall prototypes tested by in-plane compressive loading.

1/ Numbers in brackets refer to literature references listed in Section 8 of this report.

Much of the research that served as the basis for ASTM E72 was conducted at the National Bureau of Standards (NBS) in the early 1930's. In light of the fact that one of its primary functions is the development and improvement of methods of testing materials and structures, NBS was sponsored to study the principal features of several test methods with the objective of recommending structual test methods applicable to both traditional and innovative building materials and concepts.

As an aid to establishing the investigation plan, contact was made with the members of the ASTM Committee E6 Task Group that is concerned with the testing of vertical structures and with several members of the NBS staff who have had experience in testing innovative wall panels during HUD's Operation BREAKTHROUGH. With the help of written replies from and direct conversations with several of these individuals, it was possible to identify several test factors which were of common concern. A summary of the opinions expressed on the subject of wall compression testing is as follows:

(1) The ASTM E72 Test Method doesn't consider what is often the weakest link - the details at the top and bottom.

(2) The use of an eccentric loading at the top of the wall while using a flat bottom condition introduces an unknown eccentricity.

(3) The measurement of end crushing is necessary only if concentric loading is applied.

(4) The specific reference to a fine wire and a mirror for measuring lateral deflection should be expanded to incorporate modern methods of measurements. More desirably, a general description of the gage lengths and graduations of the measuring instruments should be included.

(5) Lateral deflections should be made at the level of maximum deflection, which may not always be at midheight.

(6) The selection of a specific eccentricity for standardization is arbitrary. An improvement to the standard could be made by removing the reference to a specific eccentricity and writing the standard method in a general manner that permits the selection of a specific eccentricity dependent on the nature and type system to be tested.

(7) In positioning the instruments for measuring either axial or lateral deformation, consideration must be given to the geometry and composition of the specimen as well as to the method of attachment of the instruments to the specimen.

1.2 Objective-The objective of this investigation was to develop an improved method of testing bearing wall prototypes under in-plane compressive loading. As it was intended that the improved compression test method would be recommended for adoption as a national voluntary standard, two limitations were placed on the investigation: (1) the recommended test method must be generally applicable both to traditional and innovative concepts in wall construction, and (2) the recommended test method must be simple enough in detail to be practicable for the majority of testing facilities. The objective was approached by first identifying the principal test factors and then studying them experimentally or analytically.

1.3 Scope - Five types of wall panels were selected as being a representative sample of both traditional (e.g., wood-frame) and innovative wall constructions. The sample walls, consisting of 4-ft wide by 8-ft high (1.2-m by 2.4-m) specimens, were constructed as follows:

(1) light-gage steel framing faced with single layers of gypsum wallboard and insulating fiberboard,
(2) wood framing faced with single layers of gypsum wallboard and plywood,
(3) sandwich panels consisting of a paper honeycomb core and woven roving fiberglass laminate skins; the panels were faced with single layers of gypsum wallboard,
(4) sandwich panels consisting of a corrugated fiber glass-reinforced polyester laminate core and skins made of the same material as the core, and
(5) sandwich panels consisting of a foamed-in-place urethane core and skins of single layers of asbestos cement and plywood.

The use of masonry wall sections was considered, but the literature revealed that considerable research has been devoted to investigating procedures for testing masonry walls with in-plane compressive loads.

A total of 25 compression tests was conducted in the laboratory, using the specimens whose constructions were described above. For a more detailed description of the test specimens and the prevailing conditions for each test, refer to table 1 in section 2.1. In each experiment, the load was applied by hydraulic rams and the corresponding deformations were measured with electromechanical gages. For monitoring the deformation response of the specimens, visual observation and still photography were used.

The test factors considered in this investigation are listed below along with a description of the scope of the investigation relative to each factor.

(1) eccentricity of loading - An end loading fixture was designed to permit the application of load at several eccentricities. Two eccentricities of loading were used in the laboratory investigation to assess the feasibility of using the fixture and to observe the effect of different eccentricities on the behavior of the wall panels.
(2) load distribution - An analytical study was conducted to investigate the problem of selecting load distribution hardware that is stiff enough to effect a uniform distribution along the top of a test specimen. An algorithm was derived, based on the theory of beams on elastic foundations, for determining the degree of load uniformity after selection of a load distribution assembly and the number of discrete loading points. A computer program was written in Fortran language, based on the derived algorithm. The results enable the user to simply calculate the maximum allowable axial stiffness of wall specimens for various combinations of number of loading points and distribution assembly stiffnesses to ensure approximate uniform loading, within an acceptable tolerance.

(3) end conditions - Two idealized end conditions were used in the testing program: (a) pinned at the top and pinned at the bottom and (b) pinned at the top and flat at the bottom. Also, the amount of end restraint against lateral movement of the specimens was varied through the use of end fixtures with adjustable angles. End restraint variations were introduced to determine the effect, if any, of the variations on the load capacity and mode of failure of identically constructed walls.

(4) monitoring and measuring deformation - In each test electromechanical gages were placed to measure deformation as outlined in the Standard ASTM E72-74. Visible signs of structural distress and the mode and location of failure were noted and related to the established gage locations.

(5) significance of results and number of specimens - A study of statistical parameters was conducted with the objective of determining what parameters would most effectively describe the significance of test results generated by testing wall prototypes. After establishing the desired statistical parameter, the study was concluded with a recommendation for the minimum number of specimens needed to make reliable predictions.

1.4 History - On August 6, 1937, the National Bureau of Standards published a letter circular (LC-502A) [2], in which were described test procedures to be used in determining the structural strength and stiffness of walls, partitions, floors and roofs. The stated intention was to develop standardized test procedures that would provide a reasonable basis for evaluating the merits of new construction. The circular established the number and size of specimens, the test conditions and the measurements to be recorded for a series of structural tests. In the compression test procedure, triplicate specimens, 4-ft wide x 8-ft high, (1.2-m by 2.4-m) were to be tested as columns having a flat-end bottom support, with load applied to a steel plate covering the top of the specimens. The load was to be applied uniformly along a line parallel to the inside face and one-third the thickness of the wall from the inside face. The loading was to be applied in equal increments until maximum load was reached; at each increment of load a reading of axial shortening was to be recorded. After recording the shortening, the load was to be removed and the unrecovered deformation or set to be recorded. Because the letter circular was prepared as a result of a research program focused on low-cost construction, it established minimum requirements for what appeared to be the most important structural properties. These minimum requirements provided performance limits for deciding if a construction prototype warranted further study.

On August 10, 1938, NBS published the first formal report that described methods used in the NBS structures laboratory for measuring the strength, stiffness and resistance to local damage of various types of building construction. The publication, Building Materials and Structures (BMS) 2 [3], contained details of the measuring apparatus and recommendations for reporting test results. It also cited the requirements for loading, size and number of specimens. In reference to the recommended height (8 ft) of specimens for compression testing, an explanation was offered as to how that dimension was established:

> "A height of 8 feet from floor to ceiling has been widely recognized as satisfactory for a low-cost house. Therefore, a height of 8 feet was selected for the wall specimens for compressive load. This is about the least height that can be used in a low-cost house. The actual height of a wall may be somewhat greater than 8 feet depending upon connections between wall and floor, roof, etc. For a wall, as for any column, the higher the wall, the lower the compressive strength. Most walls for low-cost houses are short sturdy columns; therefore, the strength is about the same for any height used in a house. For some constructions, particularly thin walls, the strength may be much less if the height is greater than 8 feet; it may be necessary to determine the relation between height and strength of these constructions by making additional tests. It appears probable that the strength of a wall, having a height greater than 8 feet can be estimated by the usual engineering methods with sufficient accuracy for practical purposes."

The limiting of the floor-to-ceiling height to 8 feet appears to be applicable to houses in general, irrespective of cost. It is not clear whether the authors of BMS 2 were actually referring to "small houses" in the statement about height limitations, as opposed to "low-cost houses".

The description of the loading included in BMS 2 is identical to that presented in Letter Circular LC-502A. No rationale is offered in either publication for the selection of the designated end conditions. Also, there is no explanation given for stipulating the eccentricity as one-third of the wall's thickness measured from the inside face. Apparently, the required eccentricity reflected a conclusion to apply the compressive load at the outer limit of the theoretical "kern area" described in theoretical texts on column behavior. The kern area is defined as that portion of the cross section within which an axial force can be applied without causing a stress of opposite sign at any point. At this point in the development of the compression test method, there were no recommendations for rate of loading and hold, or waiting, time at a given level of loading.

The compression test method incorporated the measurement of two modes of deformation: 1) shortening of the specimen and 2) lateral deflection at mid-height. In order to measure the shortening at various load increments, four compressometers were specified, two on each of the opposite faces of the specimen. As described in the report, a compressometer consisted of a metal rod, two brackets and a dial micrometer. The metal rod was supported near the top of the specimen by a bracket and at its lower end it was seated in a hole in the spindle of the micrometer. The micrometer was supported, with the spindle up, by the other bracket. The dial was graduated to 0.001 in (0.025 mm).

In order to measure the lateral deflections two deflectometers were used. To each edge of the specimen was attached a deflectometer, which consisted of a small diameter wire, two clamps, rubber bands and a mirror. Half of the mirror was covered by a graduated paper scale. The wire was attached at its upper end to one of the clamps and at its lower end to the rubber bands. The rubber bands were stretched to make the wire taut and were connected to a clamp located near the bottom of the specimen. The mirror was mounted horizontally to the edge of the specimen at midheight. The scale was graduated to 0.1 inch (2.54 mm). As the wall deflected laterally, the image of the wire, reflected on the mirror, marked the point to be read and recorded.

The test results were to be reported in tabular form. The deformations recorded by each deflectometer and each compressometer at the various load increments were to be reported along with the set observed after removal of each load increment. The maximum load for each specimen was also included as necessary information. Even though axial shortening and lateral deflection were measured in this method, the method was applied essentially as a strength test. As there had not been established the degree of correlation between the test-induced deformations and corresponding behavior in actual structures, the test results had minimal value as a measure of stiffness. The test results so obtained by this method were beneficial, however, in establishing the relative performance of different types of construction when subjected to an identical set of loading and boundary conditions.

On May 7, 1947, a set of test methods for evaluating the structural properties of building components was accepted by the American Society for Testing Materials (ASTM) with a "tentative" status. These tentative methods, ASTM E72-47T [4], were virtually identical in content and presentation to the recommended test methods cited in BMS 2. In the section pertaining to the reporting of test results, ASTM E72 went a step further than BMS 2 by requiring a graphical presentation of load, lateral deflection, shortening and set data. The section in ASTM E72 which related to the size of the specimens contained a note of explanation for the selection of the 8-ft (2.4-m) height with one notable word change from the paragraph included in LC-502A. The reference to a satisfactory floor-to-ceiling height for "low-cost houses" was replaced by a reference to "small houses".

In 1954, ASTM Method E72 was adopted as a standard entitled, <u>Standard Methods of Conducting Strength Tests of Panels for Building Construction</u>. The standard underwent some revision in 1955; however ASTM E72-55 did not present any changes in the sections relating to in-plane compression testing of wall panels.

In 1961, ASTM E72-55 was revised as a standard and was given a new designation, ASTM E72-61 [6], reflecting the year of latest adoption. ASTM E72-61 [7] underwent extensive editorial changes in June 1965; the major changes affecting the sections on compression testing of wall panels are as follows:

Test Specimens - Instead of designating the specimen height to be 8 feet (2.4 m), the revised edition specified that each specimen was to have a height equal to the height of the element.

Apparatus - The figures illustrating the apparatus were improved by replacing the original schematic drawing and photographic view with two detailed schematic views of the specimen as equipped with the compressometers and deflectometers.

Loading - For the first time the schematic diagram depicting the manner of loading included an illustration of a steel wide-flange load distribution beam located atop a steel plate which covered the upper end of the specimen. Also, the adopted rate of loading (corresponding to a movement of the testing machine crosshead of 0.03 in/min (0.013 mm/sec) for wood construction was specifically cited in the loading paragraph. No explanation is given for the selection of this particular rate of loading except to say that it has been found to be satisfactory. Furthermore, no reason was given for the fact that no recommendations were made as to the loading rate for other types of construction.

The latest edition of the ASTM Standard, E72-74 [8] was approved September 19, 1974. The sections related to compression loading on walls remained unchanged from the corresponding sections in the revised version of E72-61 [7].

2. LABORATORY EXPERIMENTS

Compressive load tests were performed on specimens of traditional wood-frame wall construction as well as on specimens representing proprietary wall panel systems. Testing was conducted as a means of studying procedures in an effort to develop an improved test method for recommended use. Therefore, the numerical results were end products gained from exploring various techniques and are not to be considered as evaluations of the specimens for reference. A summary of the tests performed is given in table 1.

2.1 Description of Specimens

2.1.1 Wood-Frame Construction - The wood-frame wall specimens were 4 ft by 8 ft (1.2 m by 2.4 m) and were constructed with a single bottom plate and a double top plate (figure 2.1). Studs were spaced nominally at 16 in (0.4 m) on centers; some specimens contained 3 studs (designated WFTH) and others contained 4 studs (WFFR) placed as shown in figure 2.1. Studs and plates were nominal 2 by 4 members of Construction grade Hem-fir and were fastened with 16-penny (16d) common nails. Single sheets of 4 ft by 8 ft (1.2 m by 2.4 m) gypsum board, 1/2 in (12.7 mm) thick, and 24/0 CD grade plywood, 3/8 in (9.64 mm) thick, were used as interior facing and exterior sheathing, respectively. These sheets were fastened to the framework with 6d common nails spaced 6 in (0.152 m) center-to-center along the plates and at 12 in (0.305 m) center-to-center along the studs. Facing edges of the 3-stud panels (WFTH) were braced by wood spacers as shown in fig. 2.1. The overall thickness of the specimens was 4 3/8 in (0.111 m).

2.1.2 Steel-Frame Construction - These panels (designated STLS) were nominally 4 ft by 8 ft and contained a frame fabricated of galvanized light-gage steel members which had a 0.05-in (1.3-mm) thick "Z" cross section (figure 2.2). The frame contained three studs, one at the middle of the 4-ft width and one at each of the two vertical 8-ft edges of the panel. The same "Z" sections were used to form the two horizontal members, one at the top edge and one at the bottom edge. The "Z" section members had a 3 5/8-in (92.1-mm) web, one flange 1 1/2 in (38.1 mm) wide and the other 1 1/8 in (28.6 mm) wide. Pointed barbs, which were 3/8 in (9.6 mm) wide and 1 in (25.4 mm) long, were punch-formed along the length of the 1 1/2 -in(38.1-mm) wide flange. The web at both ends of the horizontal members protruded beyond the flanges, was bent 90° over the ends of the stud webs and was fastened to them with one sheet metal screw at each connection. Both of the flanges of all three studs protruded at both ends (as tabs) and were drilled for single-nail-fastening to the top and bottom wood plates (nominal 2 by 4) that were added later as part of the wall panel assembly. The flanges of the top and bottom horizontal "Z" section members were notched out where needed to allow the extended stud flange tabs to pass through. There was no fastening of the center stud to either top or bottom horizontal "Z" member. The steel members of the frame were so oriented that

Table 1. Summary of Research Compression Tests on Wall Panels

Run No.	Test Description	Specimen Description (Code)	End Fittings (Top) (Bot.)	End Conditions (Top-Bot.)	Loading Eccentricity[a]	Instrumentation for Deflection	Instrumentation for Shortening	Maximum Load (lb/ft)	Deflection at Max. Load (in)	Mode of Failure
1	Specimen #1 Test 1	Wood-Frame (4-studs) (WFFR)	(angles in contact) (angles in contact)	Pin-Pin	t/6 [b]	10-LVDT's (5 at each wall edge)	4-LVDT's (1 near each face corner)	1600 [d]	0.23 [d]	Not loaded to failure
2	Specimen #1 Test 2	"	" "	"	"	"	"	2250 [d]	0.28 [d]	"
3	Specimen #1 Test 3	"	" "	"	"	"	"	3000 [d,e] 4000 [d,e]	0.36 [d,e] 0.56 [d,e]	"
4	Specimen #1 Test 4	"	" "	"	"	none	none	7500	[f]	Crushing & rotation of top & bot. wood plates; et al.
5	Specimen #2 Test 1	"	" "	"	t/2 [b]	"	"	3100 [d]	[f]	"
6	Specimen #3 Test 1	"	" "	"	t/2 [c]	"	"	4800	[f]	"
7	Specimen #4 Test 1	"	(angles in contact) (bearing plate)	Pin-Flat	t/6 [b]	3-LVDT's at one wall edge	"	4000 [d,e] 7200	0.15 [d,e] 0.88	"
8	Specimen #5 Test 1	"	(angles in contact) (angles in contact)	Pin-Pin	t/6 [c]	10-LVDT's (5 at each wall edge)	4-LVDT's (1 near each face corner)	4850 [d,e]	0.80	"
9	Specimen #5 Test 2	"	(one angle) (one angle)	"	"	"	"	4700 [d,e]	0.91	"
10	Specimen #6 Test 1	Steel-frame panel (STLS)	(angles in contact) (angles in contact)	"	"	"	"	3800	0.23	Web buckling of stud at ends
11	Specimen #7 Test 1	"	(one angle) (one angle)	"	"	"	"	2300	0.10	"
12	Specimen #8 Test 1	Reinf.-plastic sheet panel (GFRP)	(angles in contact) (angles in contact)	"	t/6 [b]	"	none	5400	1.31	Bond failure between skin and core
13	Specimen #9 Test 1	Foamed-plastic core panel (ACUP)	" "	"	"	"	"	11500	1.28	Separation of alum.-extrusion frame
14	Specimen #10 Test 1	Paper honeycomb panel (GPHC)	" "	"	"	"	"	6850	0.43	Bond failure between facing and core
15	Specimen #11 Test 1	Steel frame panel (STLS)	(angles in contact) (bearing plate)	Pin-Flat	t/6 [c]	"	"	3150	-0.03	Web buckling of stud at ends

Table 1 - Summary of research compression tests on wall panels.

Table 1 (cont'd). Summary of Research Compression Tests on Wall Panels

16	Specimen #12 Test 1	Paper honeycomb panel (GPHC)	" "	"	t/6 [(b)]	"	"	6900	0.24	Bond failure between top plywood block & facing
17	Specimen #13 Test 1	Reinf.-plastic sheet panel (GFRP)	" "	"	"	"	"	5700	0.65	**Skin buckling** & bond failure
18	Specimen #14 Test 1	Foamed-plastic core panel (ACUP)	" "	"	"	"	"	10950	0.95	Separation of alum. extrusion frame
19	Specimen #15 Test 1	Reinf.-plastic sheet panel (GFRP)	" "	"	t/2 [(b)]	"	"	4750	0.83	Skin buckling & bond failure
20	Specimen #16 Test 1	Wood frame (3-studs) (WFTH)	(angles-space) (angles-space)	Pin-Pin	t/6 [(b)]	4-LVDT's (1 at each mid-edge & mid-face)	6-LVDT's (as above, plus 1 at each mid-face)	5700	0.70	Crushing and rotation of top & bot. wood plates, et al.
21	Specimen #17 Test 1	"	" "	"	"	"	"	5100	0.46	"
22	Specimen #18 Test 1	"	" "	"	"	"	"	5900	1.35	"
23	Specimen #19 Test 1	"	" "	"	"	"	"	4300	1.60	"
24	Specimen #20 Test 1	"	" "	"	"	"	"	4270	1.30	"
25	Specimen #21 Test 1	"	" "	"	"	"	"	4700	>2.00	"

(a) measured from ℄ toward interior surface
(b) t = gross wall thickness
(c) t = frame stud thickness
(d) not loaded to failure
(e) unloaded to zero
(f) deflection not measured

Table 1 (cont.) - Summary of research compression tests on wall panels.

all barbs were on the same side of the frame. The barbs were used to fasten, by piercing and clinching, a 4-ft by 8-ft (1.2-m by 2.4-m) sheet of 1/2-in (12.7-mm) asphalt-impregnated insulating fiber board to one side of the frame as the exterior sheathing of the wall panel. The opposite, or interior, surface of the panel was a 1/2-in (12.7-mm) thick, 4-ft x 8-ft (1.2-m by 2.4-m) sheet of gypsum board. This was fastened to the steel frame by one plastic rivet at each of the 4 corners and by adhesive applied to the flanges of all frame members. The wall panel cavity was filled with glass wool insulation.

2.1.3 Fiber Glass-Reinforced Polyester Panels - Wall panels which were fabricated of sheets of polyester reinforced with chopped glass fibers were designated (GFRP). As shown in figure 2.3, the basic panel, nominally 4 ft x 8 ft by 3 5/8 in, (1.2 m x 2.4 m x 92.1 mm) consisted of a facing, approximately 0.08 in (2.0 mm) thick, bonded to each side of a corrugated core, made of a similar sheet 0.05 in (1.3 mm) thick, with a polyester adhesive. All four edges of the panel were closed off by U-shaped sections (similar to the corrugation material) adhesive-bonded to the faces in a re-entrant position. The exterior surface of the panel was sprayed with a polymer-aggregate coating for architectural effect. Rock wool insulation filled the vertically oriented voids formed by the corrugated cores. In preparation for testing, top and bottom wood plates (nominal 2 x 4) were epoxy-bonded within the existing top and bottom edge U-sections to simulate field attachments of bearing members.

2.1.4 Foamed Plastic Core Sandwich Panels - These test specimens which were nominally 4 ft x 8 ft x 3 in (1.2 m x 2.4 m x 76.2 mm) were marked (ACUP). Their exterior face was a 1/8-in (3.2-mm) thick asbestos cement board covered with an epoxy adhesive and decorative stone. The core was filled with foamed-in-place polyurethane of 2 pcf (3.2 kg/m^3) density and the interior facing was a sheet of 1/4-in (6.35-mm) thick plywood. All of these components were contained by an aluminum-extrusion frame shown in figure 2.4. The section of the frame was shaped for splining to matching extrusions, for the attachment of panels to adjacent panels, roof and floor.

2.1.5 Paper Honeycomb Sandwich Panels - Specimens which were designated (GPHC), contained composite facings of gypsum board and fiber glass woven roving cloth bonded with polyester resin to both sides of a paper honeycomb core. Figure 2.5 shows an exploded view of the basic sandwich panel composition used in the wall construction specimens tested. The glass fiber reinforced plastic laminates formed the structural (load resisting) component of the composite facings. The wall specimens were nominally 4 ft x 8 ft x 4 3/8 in (1.2 m x 2.4 m x 0.111 m). The upper and lower 3 in(76.2 mm) of the core (top and bottom plates) in each panel, consisted of laminated plywood bearing blocks bonded to the fiber glass cloth and the honeycomb core. The design of the wall panels required the application of compressive loads to the walls through the wood bearing blocks without any load bearing on the gypsum board facings.

2.2 Loading Procedures

2.2.1 Test Load Reaction Frame - Compression tests were performed within an adjustable steel frame (figure 2.6) composed of bolt-connected beam and column sections. The frame was bolted to a structural test laboratory tie-down floor. It should be noted that the same purpose can be accomplished by using a self-contained, closed, 4-sided frame of sufficient strength and rigidity without need for a tie-down floor.

2.2.2 Loading Fixtures - Compressive test loads were applied to the top end of all the walls by the knife-edge fulcrum fixture shown in figure 2.7. The fixture was designed to make possible the application of line loads having various eccentricities on test walls of different thicknesses. Instead of making the fixtures continuously adjustable to produce eccentricities (relative to the centerline of the specimen) ranging from zero to one-half the wall thickness (i.e.,$0 \rightarrow t/2$), the bearing plates were notched to provide for three specific eccentricities, of 0, t/6 and t/2 on a centrally placed specimen of 4-in (0.10-m) thickness. The lateral adjustability of the slotted, containing-angles provided for modifying the three specific eccentricities, in addition to accommodating specimens greater than and less than 4 in (0.10 m) in thickness. At the bottom end of the wall specimens, two alternative support conditions were used: (1) pin-end and (2) flat-end. Where a pin-end bottom was called for the specimen was supported on a fulcrum assembly identical but inverted, to the one used at the top end (figure 2.8). For the flat-end-bottom condition the bottom fixture consisted of a flat steel bearing plate. The specimen rested on the bottom fixture. Above the top knife-edge base plate, at each of four points of load application located along the specimen width, there were arranged in tandem, a load cell and a hydraulic ram. The interconnection of these elements is shown in the photograph of figure 2.9. The 30-ton (267-kN) capacity, rams were rigidly mounted on the box beam of the reaction frame and were hydraulically operated from a single manifold by a flow-controllable motorized pump. A schematic diagram of the hydraulic loading system is shown in figure 2.10. An analytical method for determining the number of discrete load points needed to achieve a satisfactory distribution of load and the theory behind the method are discussed in section 3.1.

2.2.3 Rate of Loading - Since, in undertaking these tests, rate of loading was a subject open for discussion, no firmly established rate was adopted. Such a selection is largely influenced by the strength and stiffness of the specimen and the need to obtain accurate data at enough load levels to determine a meaningful load-deformation performance curve. In choosing a loading rate, it is necessary for the rate to be slow enough so that the rate of accompanying deformation does not lag behind the test load development, but extremely slow rates must be avoided in order to conduct a test in a reasonably short time. After some preliminary consideration of previous testing experience and of anticipated maximum loads, it was decided that for the purpose of these experiments a suitable loading rate would be one which permitted a test to be completed in approximately one hour (with modification as needed). For the width of specimens, 4 ft (1.2 m), and number of rams involved (4), this resulted in a basic rate of approximately 80 lb/ft·min (19.5 N/m·sec) and was altered for some circumstances (e.g., during accelerated deformation accompanying incipient failure). It was noted that at this approximate rate of load application, an X-Y recorder used for plotting the deflection as a function of load, did not indicate any deformation lag at the incremental measurement stops; and that, for monitoring load application by a load cell connected to a digital voltmeter, the rate of digit change which confronted the pump operator was equal to one digit (i.e., 0.01 mV or 14 lb/ram) every 10 seconds, a rate not difficult to regulate.

2.3 Instrumentation and Measurements

2.3.1 Load - At each location of load application (i.e., at each ram) the measurement of forces was made by means of an electrical resistance type load cell of hollow-steel-cylinder design. Each load cell had a calibrated capacity of 25 kips (111.2 kN). Loads were applied through these load cells and measured in increments of 250 or 500 lb (1.11 or 2.22 kN) per ram, depending on the anticipated strength and stiffness of the specimen. The electrical signals from all of the load cells, as well as other transducers described below, were fed into the data logging equipment described below. The signal from one representative load cell was also simultaneously fed to a digital voltmeter and to the vertical axis of an X-Y recorder for monitoring of the test progress by equipment operators and observers.

2.3.2 Deformation - Deformations or displacements of the specimens were measured by linear variable differential transformers (LVDT'S). These transducers measured flexural transverse deflection and longitudinal shortening of the wall panels subjected to vertical compressive loads. The LVDT's were of three different calibrated ranges ($\pm$ 3 in, $\pm$ 1 in, $\pm$ 0.1 in) depending on the anticipated displacement to be measured; all transducers had a total range output signal of about $\pm$ 4.5 volts.

In order to measure vertical shortening of a wall, compressometers were made by mounting an LVDT in one end of an aluminum tube to achieve long gage lengths. As shown in figure 2.11, the empty end of the tube was pin-connected to the panel at its top. At the bottom end, the sliding core of the LVDT contacted a reference bracket that was attached to the wall. Also, close to the bottom end, the tube was connected by rubber bands to a V-notched guide bracket that was attached to the wall. The bracket guided the sliding of the tube as the wall shortened under load.

Lateral deflection of the panel was measured by LVDT's which were mounted transversely on similarly constructed aluminum tube supports (figure 2.11). The LVDT's were oriented in a direction parallel to the expected deflection and were attached to the tubes at the several levels at which deflection measurements were desired. It should be noted that in attaching the tube suspension pins to the wall, regardless of whether a face or an edge surface is involved, the orientation of the axes of the pins in the horizontal plane must be parallel to the neutral surface of the wall in order to permit unrestrained pivoting of the tubes as the wall ends are rotating. All of the LVDT output signals were fed to the data logging system. In addition, one LVDT which detected mid-height deflection of the wall was simultaneously connected to the horizontal axis of the monitoring X-Y recorder.

2.3.3 Recording - All load cells and displacement transducers were calibrated, using NBS traceable standards, before use in the tests. Load cell readouts were accurate to at least within $\pm$ 1% and LVDT readouts were accurate to within $\pm$ 0.5%. The data for all tests were acquired by use of a computer-controlled automatic scanning data acquisition system which had a capacity of 200 signal channels. The voltage signals from all load cells and LVDT's were recorded on magnetic tape by the data logging system at each load increment and as digital printout by a teletypewriter upon demand. The data recorded on magnetic tape were subsequently processed by an electronic computer which converted them to engineering units printed out in tabular and graphic forms.

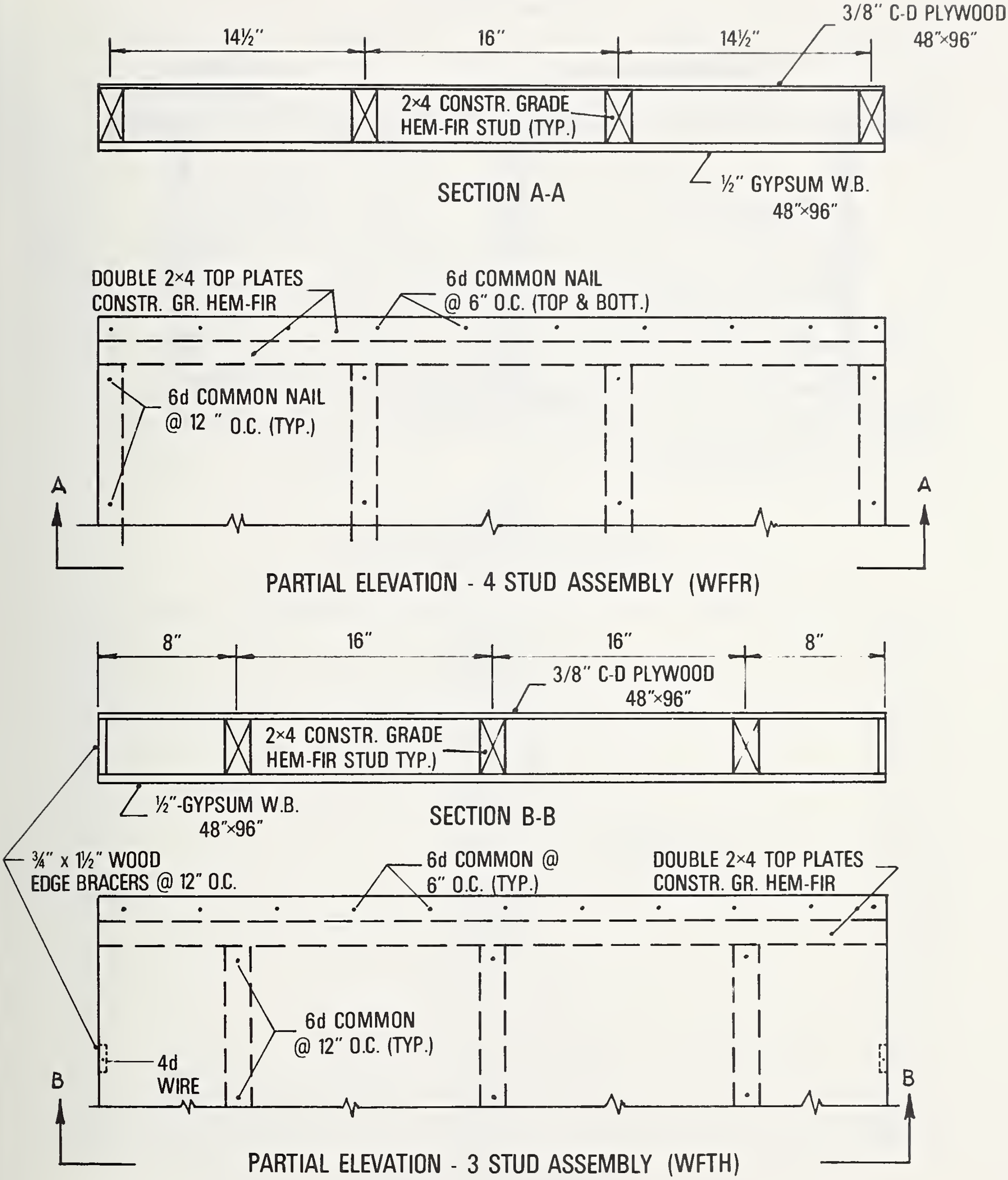
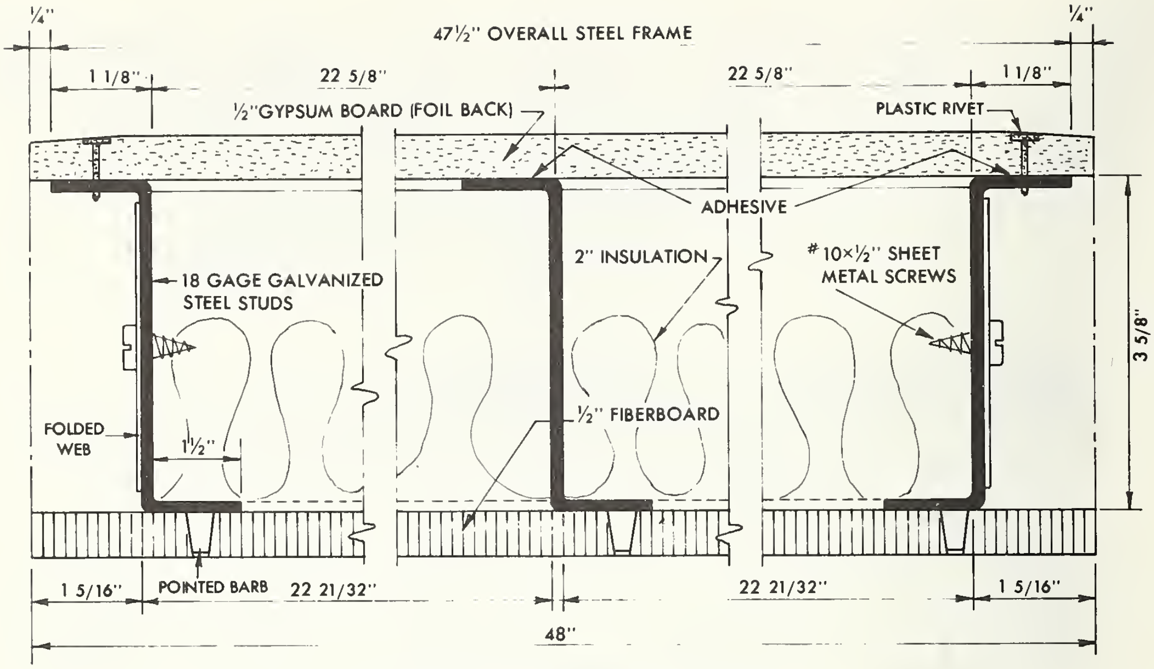
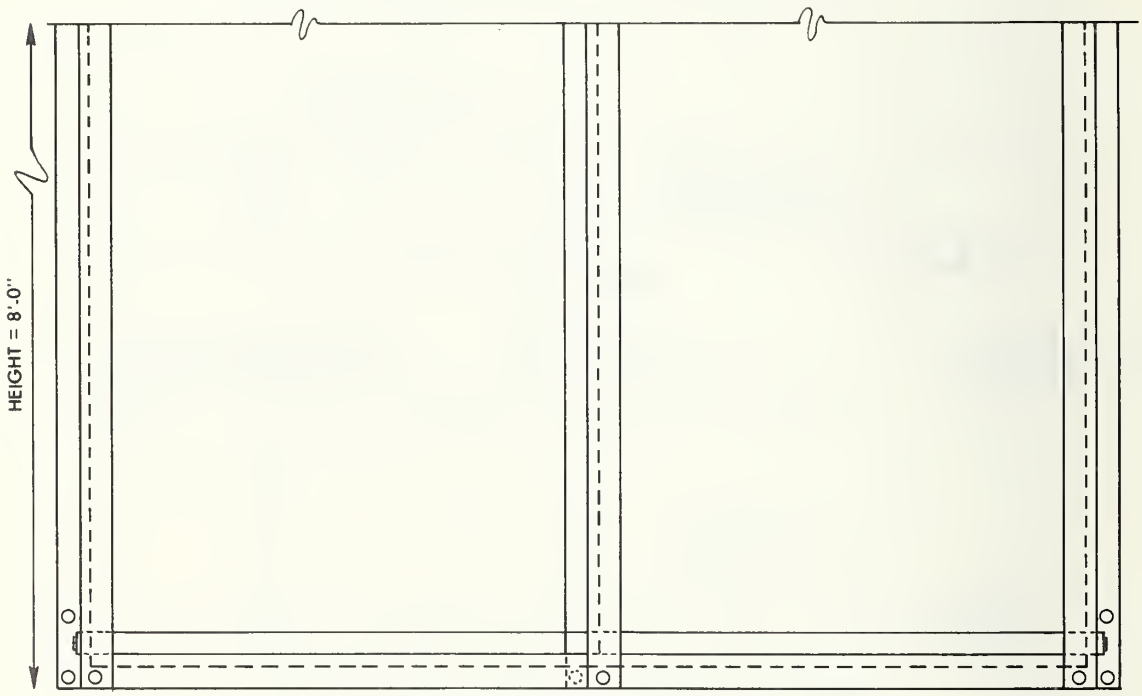


Figure 2.1 - Wood-frame construction test panels (WFFR, WFTH).



HORIZONTAL SECTION THRU PANEL



PARTIAL EXTERIOR ELEVATION
STEEL PANEL FRAME
w/o FACINGS

Figure 2.2 - Steel-frame Construction test panel (STLS).

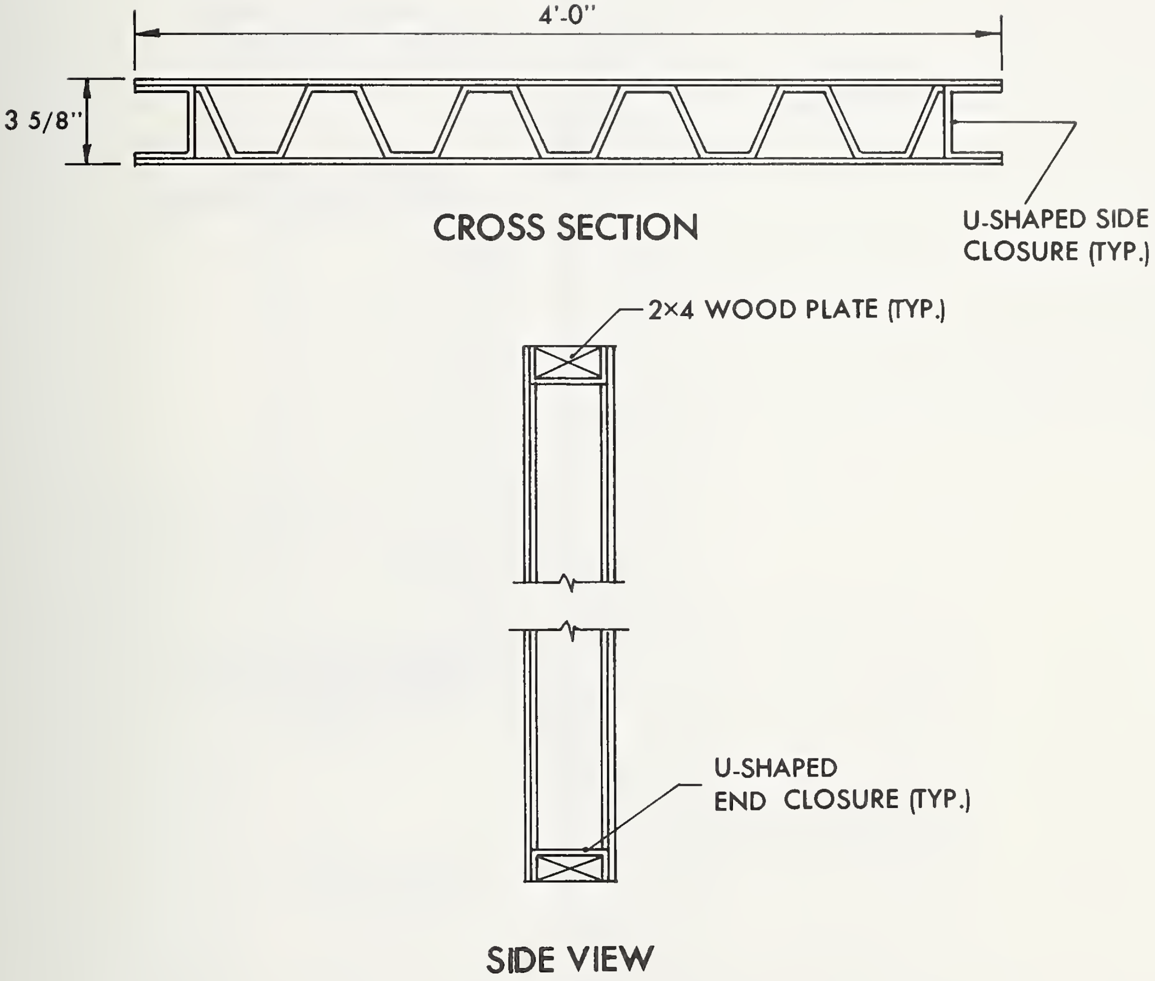


Figure 2.3 - Glass-fiber reinforced plastic test panel (GFRP).

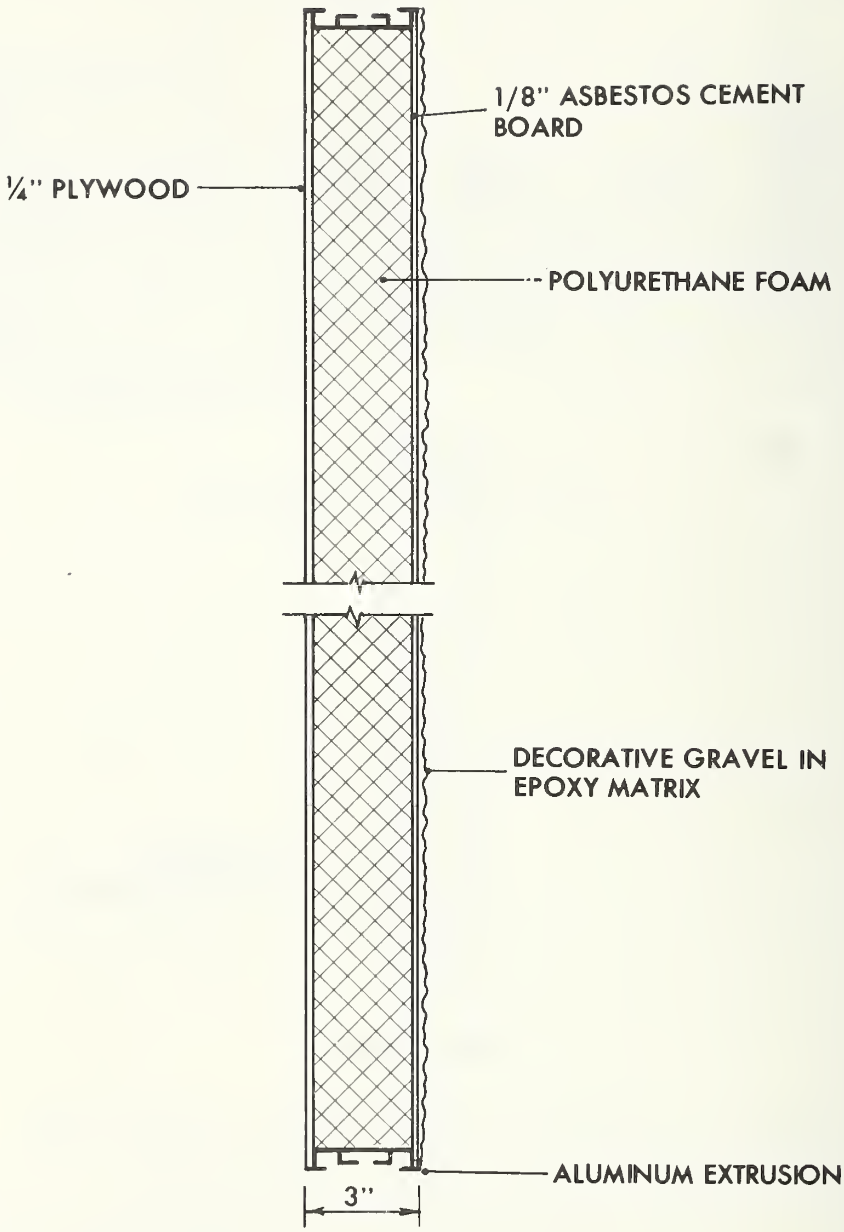


Figure 2.4 - Foamed plastic core sandwich test panel (ACUP).

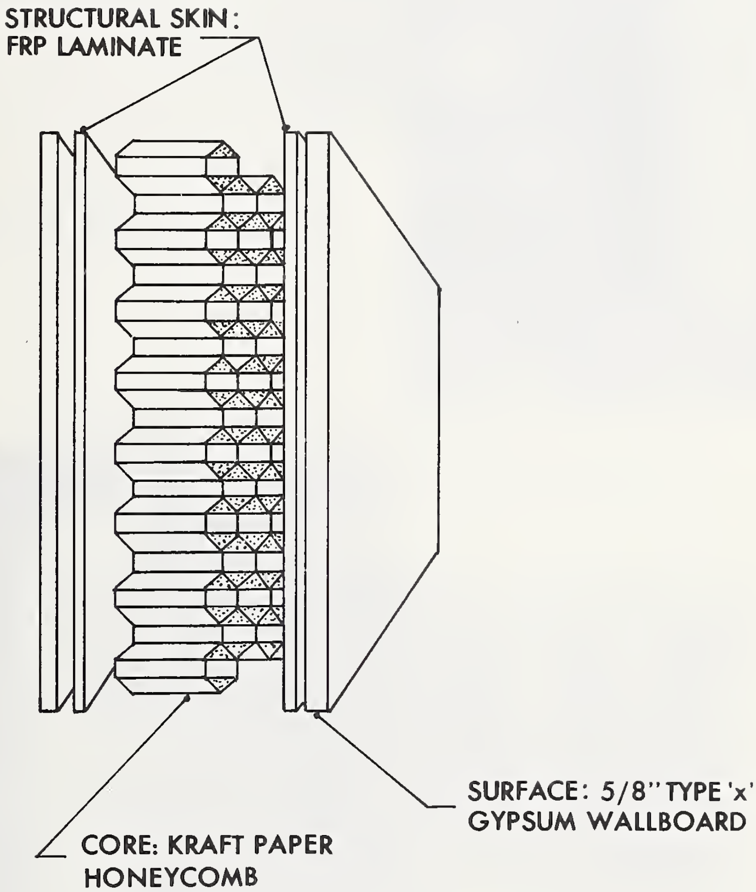


Figure 2.5 - Construction of paper honeycomb sandwich test panel (GPHC).

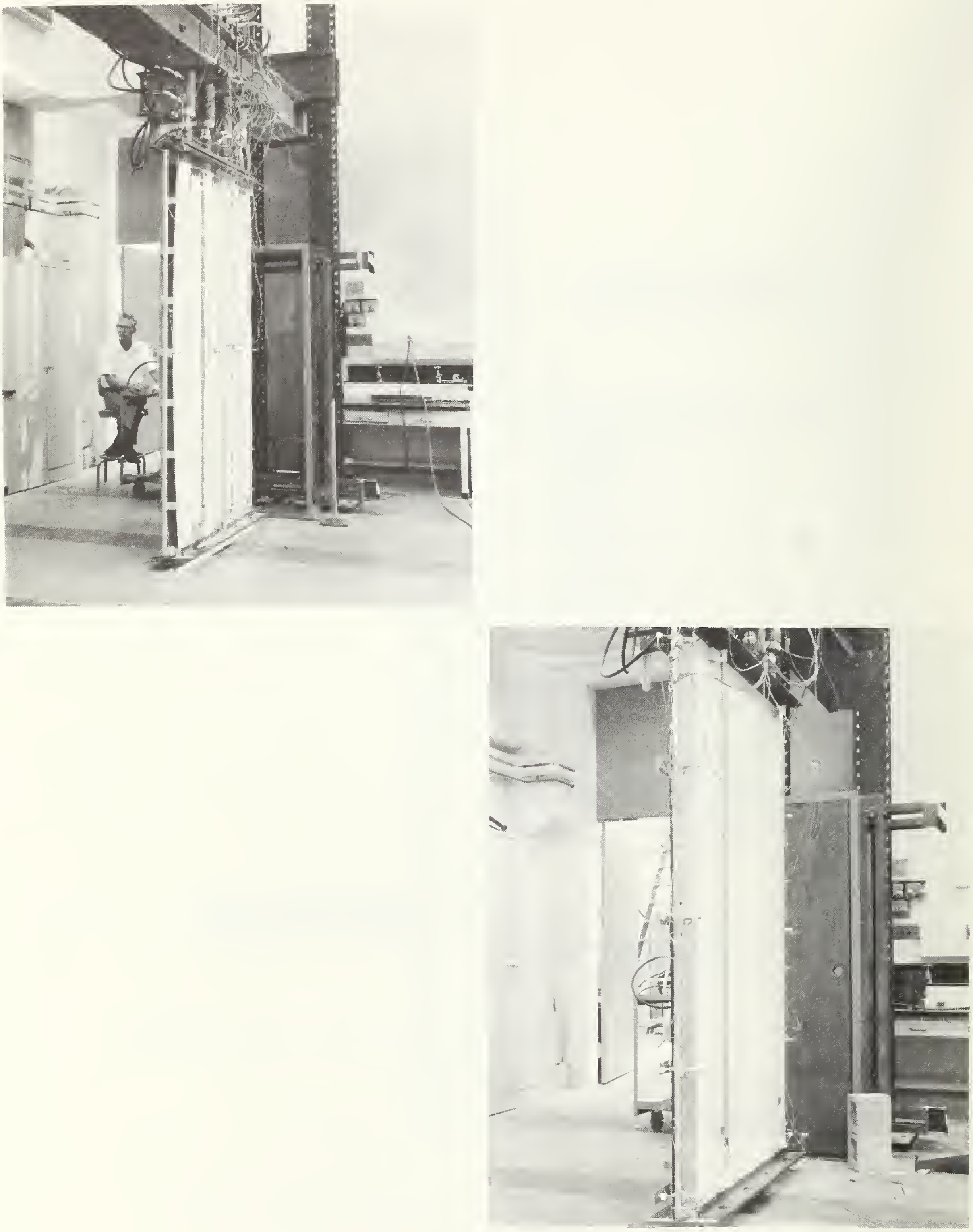


Figure 2.6 - Typical compression test setups.

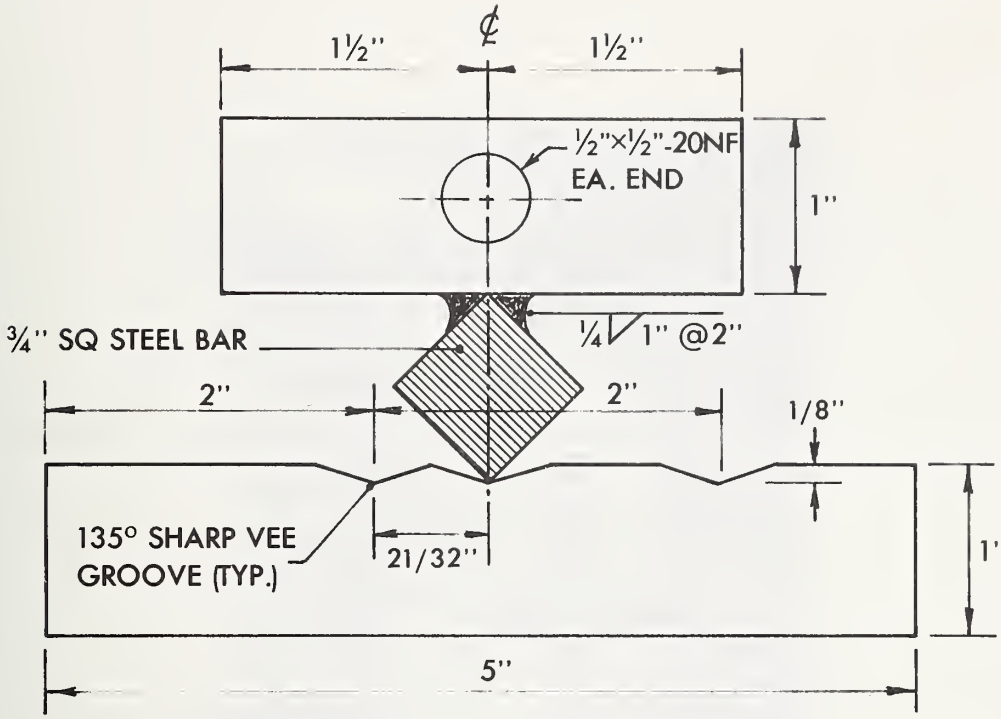


Figure 2.7 - Fulcrum loading fixture for pin-end condition.

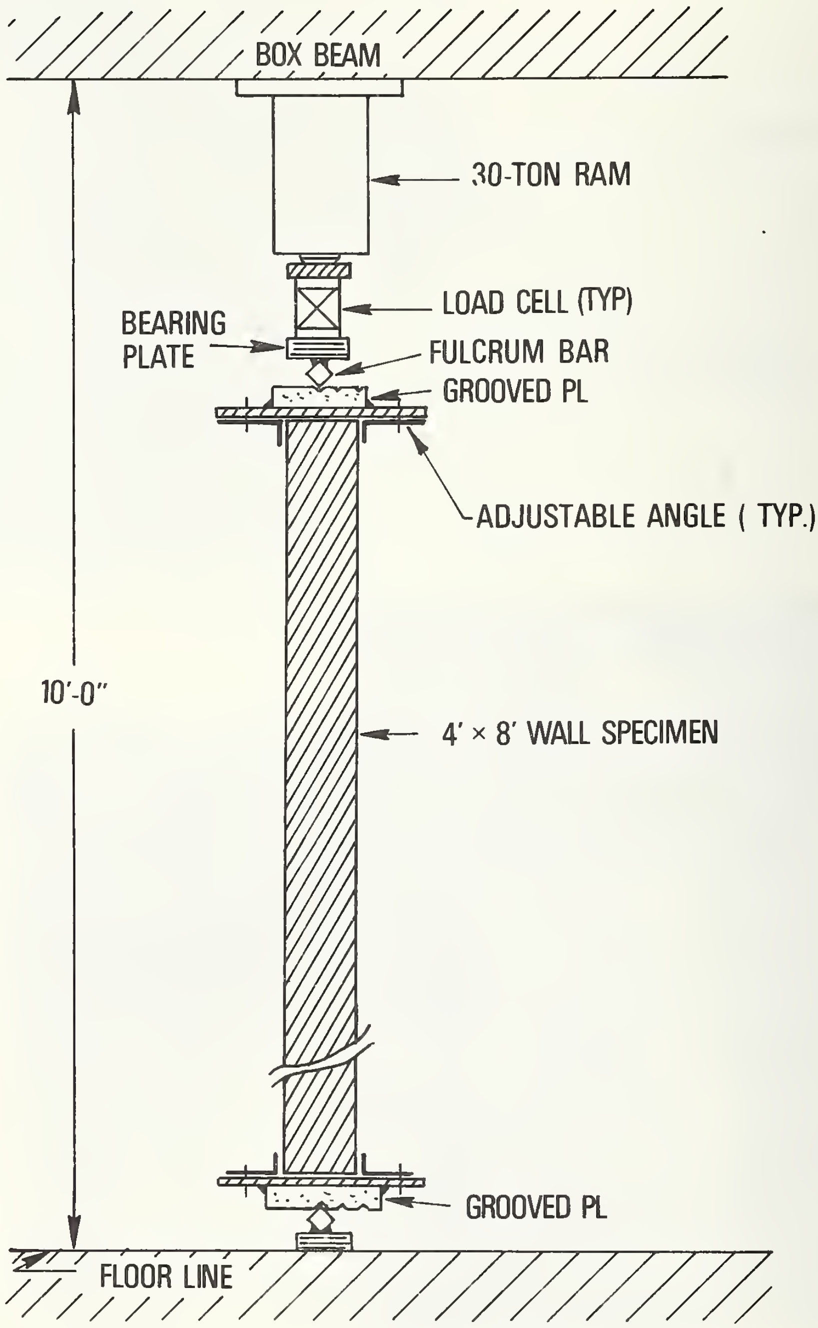


Figure 2.8 - Profile of typical pin-pin loading setup.

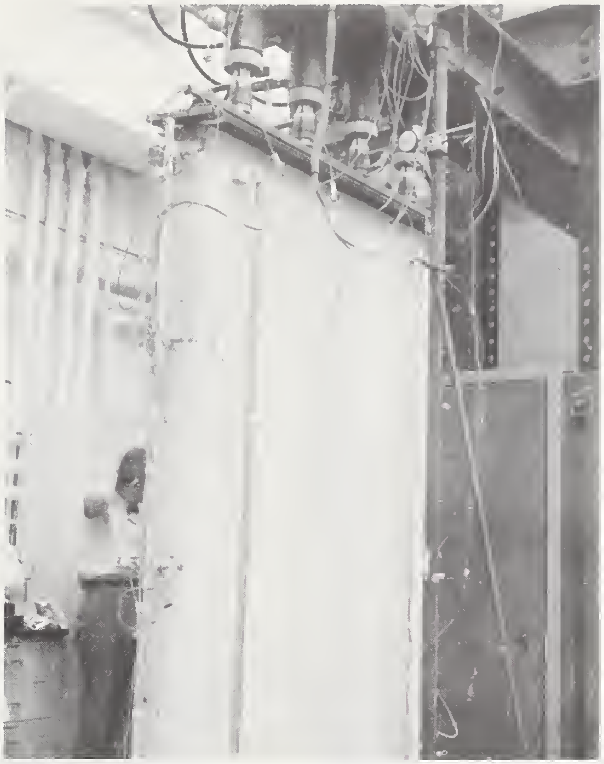


Figure 2.9 - Interconnection of upper test-apparatus elements.

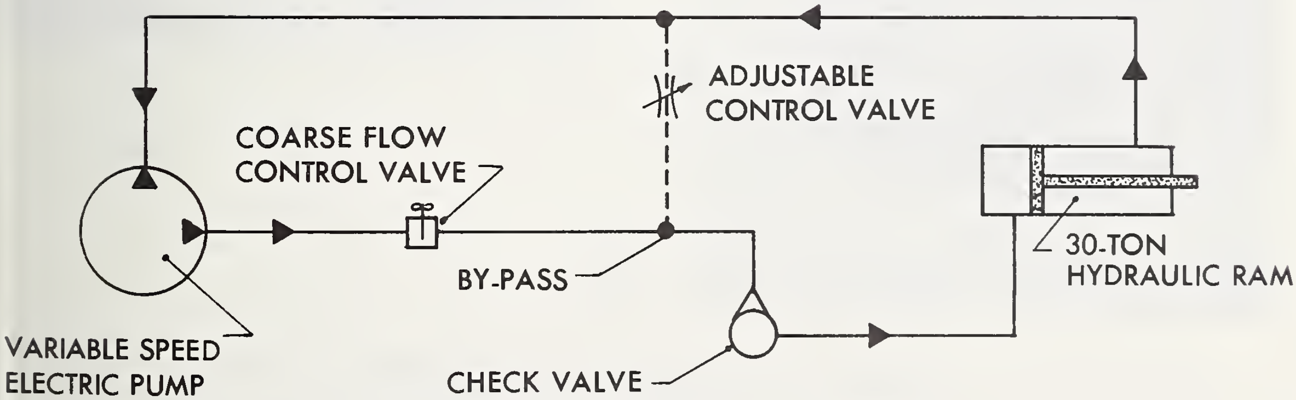


Figure 2.10 - Schematic of hydraulic loading system.

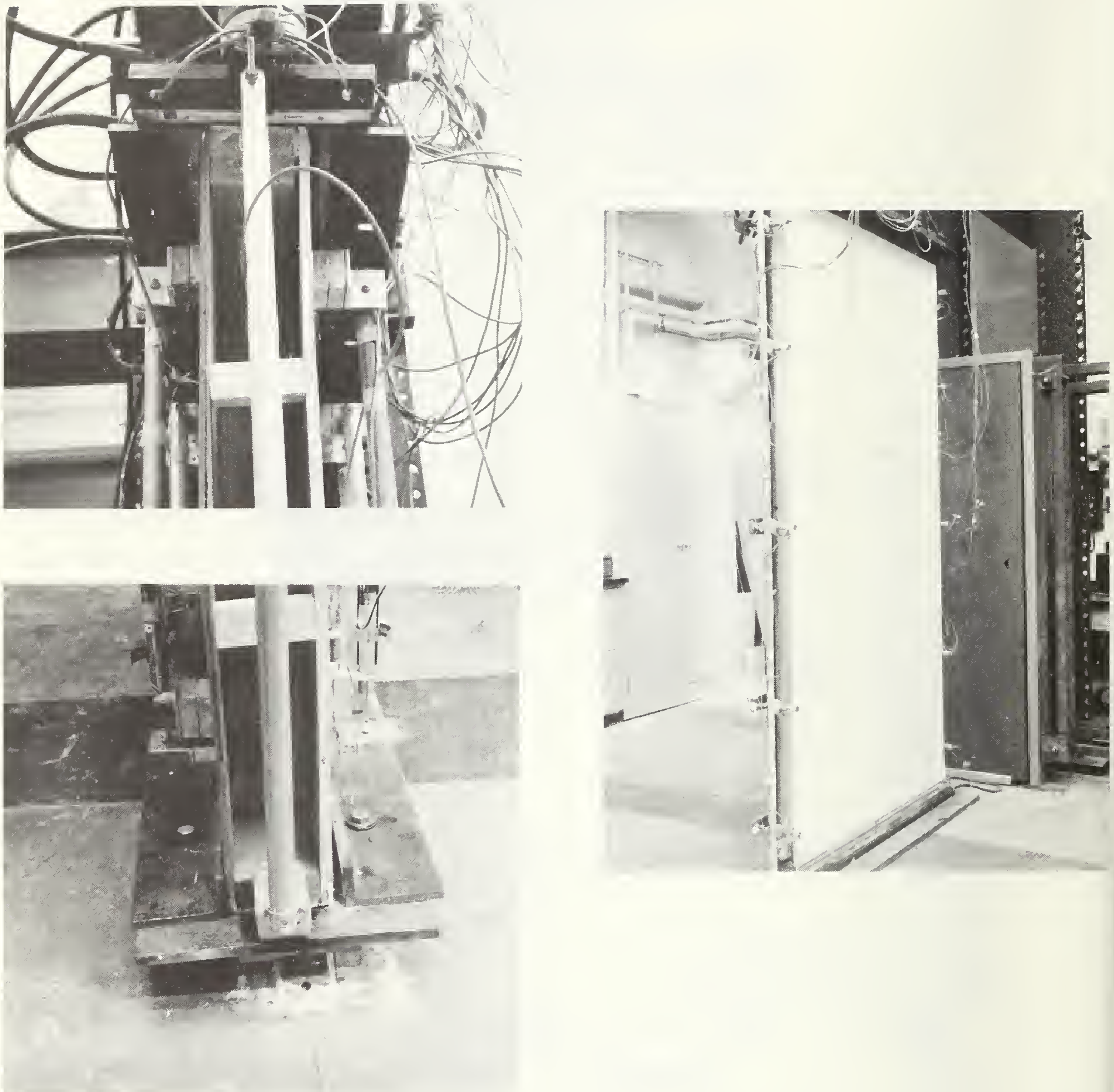


Figure 2.11 - Mounting details of LVDT's.

3. ANALYTICAL INVESTIGATIONS

3.1 Load Distribution - A very important practical consideration in the development of a test method for evaluating the compressive resistance of walls is the procedure for applying the load to the specimens. The usual practice is to use several hydraulic rams to apply concentrated loads to a distribution beam which in turn distributes the total load to the top of the wall specimen. The technical problem lies in the determination of the design of the load distribution assembly so as to effect a good approximation of uniform loading. The variables under the control of the evaluator are the number and position of the loading rams and the stiffness (EI) of the distribution beam. The greater the number of rams and the stiffer the distribution beam - relative to the axial stiffness of the wall specimen - the closer one comes to achieving uniform loading. This fact not withstanding, the economics of conducting performance tests dictate that one attempt to optimize the distribution system. The problem then, is to derive a method for designing the load distribution system that leaves the decisions as to the maximum tolerable deviation from uniformity and the allocation of equipment to the evaluator.

The conditions of this technical problem were found to have a theoretical analogy in the theory of elastically supported beams. In defining the conditions of support, Hetenyi [10] states that "elastic support is provided here by a load-bearing medium, referred to as the 'foundation,' distributed uniformly along the length of the beams." The fundamental assumption in this theory is that the magnitude of resistance provided by the foundation at any point is proportional to the vertical deflection, y, of the beam at that point. Assuming the supporting material obeys Hooke's law, its distributed reaction, p, can be expressed as follows:

$$p = ky$$

where k is called the modulus of the foundation or foundation constant, expressed in units of lbf per inch of wall per inch of deflection (newton per meter per meter). The deflection curve of the beam can be expressed in terms of a fourth-order differential equation:

$$\frac{EId^4y}{dx^4} = -ky + q$$

where q represents the intensity of distributed loading on the beam. The particular case in the theory which relates to the problem is that of beams of finite length with free ends. Taking the associated boundary conditions into account, Hetenyi has provided solutions for various selected loading cases. To obtain a solution for a number of concentrated loads, spaced equally along the length of the beam, it is only necessary to apply the fundamental principle of superposition. Thus, the basic equations are available to enable the computation of the deflection - and, hence, the intensity of foundation reaction - at any point along the beam, for a combination of loadings.

On the basis of the above mentioned analytical tools, an algorithm was developed to optimize the selection of the load distribution assembly. The initial step of the method is to compute the magnitude of the foundation (i.e., wall) reaction, as a function of the deflection and an initial trial foundation constant - at several key locations along the distribution beam. These discrete values of foundation reaction are then averaged and their respective deviations from the average are computed. This procedure requires the input of several variables: (1) EI of the beam assembly, (2) the number and location of the concentrated loads and (3) the absolute value of maximum variation from uniform load distribution that will be allowed. The output from this procedure is a value of the maximum stiffness of wall specimen that can be tested for the selected EI and load input without exceeding the maximum allowable load variation specified. The process for obtaining the desired output is an iterative one and a computer program was written to facilitate the tedious computations. The details of the computer program are discussed in Appendix A. A practical application of the analysis is illustrated in the proposed standard test method which is presented in section 6. Approximate foundation constant values were computed for some typical wall constructions and they are presented in tabular form (table 6.A.1) in the same section.

3.2 Eccentricity of Loading - Because the horizontal components (such as floors, roofs ceilings) of buildings connect to exterior walls unsymmetrically it is customary to think of testing bearing wall prototypes by applying an eccentric axial, compressive load. In fact, ASTM Standard E72 has standardized the eccentricity by specifying that it shall be applied along a line located one-third the thickness of the specimen from the surface of the inner facing material. The rationale for this specific eccentricity requirement was not determined during this study, but the one-third fraction brings to mind the postulate of a core or kern radius. The core is defined by Shanley [9] (e.g.) as "that portion of the cross section within which an axial force can be applied without causing a stress of opposite sign at any point."

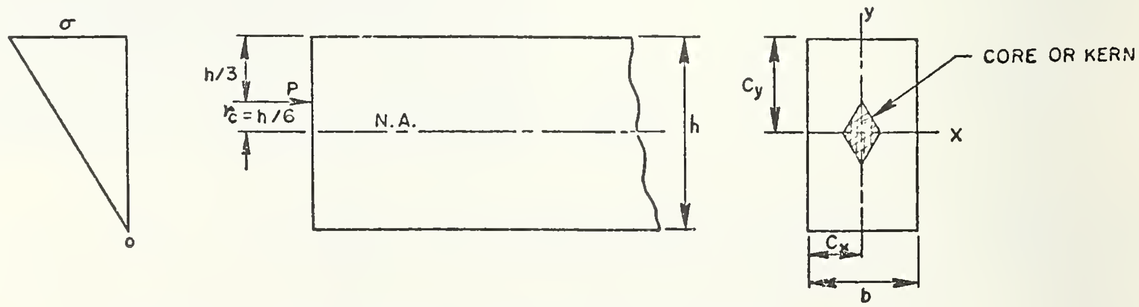


Figure 3.1 - Kern geometry of a rectangular cross section.

There is implicit value in knowing the boundaries of this area in types of construction in which tensile stresses are to be avoided. Figure 3.1 illustrates the concept of the core and core radius for a homogeneous material of rectangular cross section.

In figure 3.1 the symbols are as follows:

P - axial load

σ - extreme fiber stress

h - height of cross section

b - width of cross section

C_y - distance parallel to the y-axis from the edge to the centroid of the area

C_x - distance parallel to the xaxis from the edge to the centroid of the area

x,y - principal axes

r_c - core radius

N.A. - neutral axis or axis of zero stress for pure elastic bending

The concept of the core as pictured above is not generally applicable to walls because the majority of wall types are heterogeneous. Most types of wall construction consist of at least two different materials that are joined together to form a structural unit. The kern radius of composite walls can be located if their cross sections are "transformed" into an equivalent cross section of uniform properties. One of the materials is taken as a base and then the other materials are transformed, in proportion to the ratio of their respective moduli of elasticity, relative to that of the base material. In Appendix B a procedure is illustrated for calculating the cross-sectional properties of a transformed area. The procedure includes a step for calculating the core radius. Once the value of the core radius is determined, it can then be compared to the eccentricity described in ASTM E72. Examples are included to illustrate the use of the procedure for three commonly used wall constructions. The purpose of performing the calculations was to establish if the basis for stating the single value of eccentricity, e , was the concept of kern radius. The conclusion drawn from this exercise is presented in section 5 of this report.

3.3 Statistical Parameters Needed to Describe Results - The type of calculations required to report the test results and the kind of predictions that are expected is dependent upon the type of test that is conducted. On the subject of types of tests, Dorey and Schriever [11] have distinguished between the objectives of an Acceptance (or Proof) Test and those of a Rating Test.

The Acceptance Test is conducted to evaluate the structural properties of the unit under test. Simulation of actual boundary conditions is very important in this case. On the other hand, a Rating Test is conducted to establish a maximum test load - or some other performance attribute - for a prototype specimen. Specimens are nominally identical to the structural components which they represent. In order to project the performance of the actual, built components, consideration must be given to the variability in strength and stiffness of the samples. In view of the fact that most performance evaluation testing of bearing wall construction is conducted on prototypes of the actual, built components, it is concluded that the test results should be reported in a format consistent with the objectives of a Rating Test. In fulfulling these objectives, the question arises as to which statistical parameters are germane. Before attempting to answer this question it was necessary to define the kind of prediction that one desires to make after testing a sample of wall panels in compression. It is premised that the evaluator would like to use the performance test results to predict, with a reasonably high level of confidence, that a high proportion of the population of walls represented by the test sample will possess a strength or stiffness in excess of a certain limit. A study of the application of the statistical parameters commonly used to describe test results revealed that the use of one-sided statistical tolerance limits was appropriate for this purpose. It is important to note that the application of one-sided tolerance limits constitutes a very significant extension of the current general practice of simply estimating the mean or variance of the population distribution on the basis of testing three or more specimens. The concept of one-sided tolerance limits can be expressed in the following formulation: the probability is γ (e.g., 0.95) that at least a certain proportion, p (e.g., 0.90), of a population will be greater than the limit $\bar{x} - ks$, where $\bar{x}$ is the sample estimate of the population mean and s is the positive root of the sample estimate of the population variance (i.e., the sample estimate of the standard deviation). The value of the factor, k, can be obtained from a table of factors for one-sided tolerance limits for a normal distribution function. Table A-7 in reference [12] presents a listing of k factors for selected sample sizes and values of γ and p. The reader is referred to section 6.9.3 for the step-by-step procedure of calculating the test results as recommended in the proposed standard test method.

4. DISCUSSIONS

The discussions of the individual test runs which follow correlate with the abbreviated summary of research tests given in table 1.

4.1 Test Run No. 1 (WFFR) - This run and the next three (run Nos. 2, 3, 4) were made on the same specimen for initial exploratory tests in trying out the equipment of the test setup. The 4-stud wood-frame specimen was of the type shown in figure 2.1 with the plywood considered as the exterior face and the gypsum board, the interior surface. The top and bottom ends of the facings were trimmed back 1/4 in (6.3 mm) from the respective bearing surfaces to prevent direct bearing of the steel loading plates on the edges of the facings. Steel angles, used as end fittings for lateral containment of the ends of the wall, were bolted in position on the steel bearing plates in contact with the wall but without lateral pressure. The fulcrums and bearing plate notches were positioned to provide a loading eccentricity of t/6 toward the interior (gypsum) face based on the gross thickness of the wall section. Arrangement of the LVDT's for measuring transverse deflection (10 positions) and shortening (4 positions) corresponded to the instrumentation setup shown in figure 4.1.

In this test run the wall was loaded in increments of 250 lb/ft (3.6 kN/m) to a maximum of 1600 lb/ft (23.4 kN/m). Load and deformation measurements were logged after adding each increment of load. From the maximum load level the wall was unloaded continuously at approximately the loading rate of 80 lb/ft/min (19.5 N/m/sec). The taped data were considered unreliable because of electronics difficulties which were detected during the test. However, the x-y recorder provided a graph of the test as represented by the monitored load-cell and midspan-deflection transducers. The graph exhibited much "staircasing" or "stepping" as opposed to being a smooth curve record of a load-deflection relationship. At the time of testing the phenomenon was considered attributable to erratic specimen behavior but observations in later tests (e.g., c.f. No. 2) indicated that the staircasing might be corrected by improvements in LVDT mounting.

4.2 Test Run No. 2 (WFFR) - The setup and procedure for this test were the same as run No. 1 except that loading was carried to a maximum of 2250 lb/ft (32.8 kN/m). The electronics circuitry had been modified but the data output was still suspect. Staircasing of the x-y record was again evident. However, observation during the test seemed to indicate that the fault was in the manner of mounting the LVDT's used for measuring transverse deflection of the wall. As shown in figure 4.2 (a), the bullet-nose end of the LVDT core extension had been glued (hot-melt) to the wall surface reference plate. While this arrangement provided stability for the LVDT, it did not allow for vertical movement of the LVDT relative to the reference plate. The relative vertical movement was caused by the shortening of the wall which carried the supporting aluminum tube. The result was a jamming of the core in the coil by frictional force which was overcome intermittently (thus producing a "staircase" graph) during horizontal movement of the core produced by deflection of the wall. It was decided to provide the necessary stability for the core by replacing the gluing procedure with compression-spring loading of the core as shown in figure 4.2 (b). This permitted vertical sliding of the bullet-nose on the reference plate.

4.3 Test Run No. 3 (WFFR) - Run No. 3 also used the same specimen, setup and procedure as Run No. 1, except that the LVDT cores were spring-loaded as described in 4.2. The first cycle of loading was carried to 3000 lb/ft (43.8 kN/m) and returned to zero. No staircasing was observed on the x-y graph. The second cycle of loading was taken to 4000 lb/ft (58.4 kN/m) and returned to zero. From about 3500 lb/ft (51.1 kN/m) up, staircasing was apparent. This time, the jamming of the cores occurred because the deflection LVDT's were mounted in such a direction (i.e., with cores referenced to the wall surface which become concave) that the core springs were compressed further as deflection increased. The result was an increased pressure of the bullet-nose against the reference plate, causing difficulty in sliding between the two, and as before, consequential friction-jamming of the core in the coil. Upon unloading the wall in the second cycle, the staircasing disappeared at about the same value of deflection of the wall as that at which it had started. In future tests, the deflection LVDT's were oriented so that the core end was referenced to the convex face of the wall. In this way, wall deflection caused extraction of the LVDT core from its coil and the stabilizing spring (sufficiently compressed) was allowed to relax, yet perform its function.

During the remainder of the second unloading cycle other aberrations appeared on the x-y graph. Close observation of the test setup and the graph being generated revealed that these deviations were caused by irregular "unload-recover" behavior due to ram piston friction being developed by the lateral thrust of the flexed wall which was tending to recover. The electronics circuit was still not trouble-free during this test run.

4.4 Test Run No. 4 (WFFR) - This test setup and specimen were the same as for run No. 1 except that no LVDT's were used and load cells were read only by the teletype; one load cell was also used to monitor the load using only the vertical axis readout on the x-y recorder. The test was intended to provide information on the mode of failure without risking damage to instruments. The wall was loaded in increments of 250 lb/ft (3.6 kN/m) to a maximum of 7500 lb/ft (109.4 kN/m). Complete failure of the specimen occurred at maximum load by splitting of an outer stud near a knot (fig. 4.3). This had been preceded by crushing and rotation of the top 2 x 4 wood plates caused by eccentric loading of the wood plates against the stud ends (a relatively concentrated loading condition). Also apparent was evidence of nail heads being pulled through the gypsum board. Cracking of the gypsum board seen in figure 4.3 was considered secondary, following the stud failure; quadrilling of the gypsum board had been performed to assist detection of surface distortions visually and photographically. Both the plywood and gypsum board were cracked near the upper steel containing angles. The gypsum board was also crushed because of rotation of the top wood plates, resulting in closure of the 1/4-in (6.3-mm) space between the end of the gypsum board and the steel bearing plate. The corresponding 1/4-in (6.3-mm) space on the plywood side had become enlarged. Electronics difficulties were still present during this test run.

4.5 Test Run No. 5 (WFFR) - The preceding test (run No. 4) was duplicated by this run with the following two exceptions. Instead of trimming the ends of the facings, an aluminum shim plate, 1/4 x 3 1/2 x 48 in (6.3 x 88.9 x 1219.2 mm) was fastened with screws to the top and the bottom wood plates of the new specimen to provide clearance from bearing for the facing edges. The loading eccentricity was 1/2 the gross thickness of the wall so that the load was in line with the exposed surface of the gypsum board. The wood plates were beginning to show evidence of crushing-rotation when, at 3100 lb/ft (45.2 kN/m), the bottom end hardware slipped off the end of the wall and pivoted about the edge of the aluminum shim plate (figure 4.4). Considering that the load eccentricity was large (t/2) and that it extended beyond the edge of the shim, where there was no wall bearing, the result was not unexpected. Nevertheless, the test was instructive with regard to hardware performance relative to the development of specimen failure. Data readouts were still questionable in this test because of continued trouble with the electronics.

4.6 Test Run No. 6 (WFFR) - Two modifications of the preceding test run (No. 5) provided the setup for this run. The loading eccentricity (t/2), was based on stud width, thus putting the load line at the interface of studs and gypsum board, and the aluminum shim plates were not attached to wood plates but, rather, tack-glued to the steel bearing plates. Otherwise, the test proceeded as before, primarily to observe specimen behavior under load but without LVDT instrumentation. This new wood-frame specimen was loaded to 4800 lb/ft (70.0 kN/m) at which time the excessive crushing-rotation of the wood plates was considered to have constituted failure (figure 4.5). This was accompanied by cracking of the gypsum board at the level of the containing steel angles, as well as by its buckling near the top. The clearance for the gypsum board edges, provided by the aluminum shim plates, had been closed up by crushing and rotation of the end plates. However, the end fittings did not break loose from the wall. The circuitry problem continued to exist.

4.7 Test Run No. 7 (WFFR) - For this test a new wood-frame specimen was used. Only 3 LVDT deflectometers were mounted at one edge of the wall (one at mid-height and at the upper and lower 1/4-points). Inadvertently, these three spring-loaded LVDT's were again mounted with reference plates attached to the interior wall surface (concave), resulting in staircasing of the x-y recorder plot (c.f., test run No. 3). The upper end of the wall was fitted with the same hardware (angles, plates and fulcrum) used in preceding tests, but the bottom end was provided with only a flat steel bearing plate; aluminum shim plates (attached to the steel plates) were used at top and bottom to provide the facing edges with clearance from bearing. Loading eccentricity was set at t/6, based on gross wall thickness. The wall was loaded to 4000 lb/ft (58.4 kN/m) in 500-lb/ft (7.3-kN/m) increments and then unloaded. The top of the wall showed evidence (visual and x-y record) of permanent deformation by crushing and rotation of the wood plates. The wall was then loaded incrementally to 7200 lb/ft (105.1 kN/m) at which load the test was halted because of excessive top end-rotation (figure 4.6). The opposite ends of the wall showed great contrast in behavior as influenced by the respective load bearing conditions.

With the execution of this test the difficulties which had existed in the electronics circuit were resolved. For test run No. 7, and all following tests, the electronics instrumentation data were considered reliable.

4.8 Test Run No. 8 (WFFR) - In this test a fresh 4-stud wood-frame wall panel was set up with fulcrums, angles and unattached aluminum shim plates at top and bottom. The loading eccentricity, t/6, was based on the stud depth of 3 1/2 in (88.9 mm). The specimen was instrumented with 14 LVDT's attached, generally as in test run Nos. 1-3, for measurement of shortening and deflection. However, the reference plates for the deflectometers were mounted on the exterior face of the wall (which became convex), and each of the LVDT bullet ends was held in contact against the reference plate, by a lightly stretched rubber band. The smooth x-y recorder graph which was obtained in this test proved this core holding technique to be satisfactory. Several rack and pinion type dial gages were supported at the wall mid-height by a floor stand to obtain comparative readings of lateral displacement under load. Comparison of the deflection data from the dial gages with those obtained from the specimen-mounted LVDT's confirmed that the specimen-mounted deflectometers provided a more reliable and definitive measurement of the specimen's deflection. The wall panel was held in place before testing with a small initial load, loaded to 4850 lb/ft (70.8 kN/m) in 500-lb/ft (7.3-kN/m) increments and then unloaded gradually, stopping at 3 load levels to make measurements. The main purpose of this test run was to acquire a representive set of data to be processed by a digital computer into engineering values for tabular and graphic printout. In addition to simple conversions by use of factors, the program also contained subroutines for obtaining averages of designated transducers and for plotting multiple graphs on one set of multiply-labeled axes.

4.9 Test Run No. 9 (WFFR) - This run was a second test on the specimen used in the preceding run. It was conducted primarily to observe the performance of a specimen (with fulcrum loading at both ends) when the containing steel angle was not used at top or bottom on the interior (concave) side of the wall panel (figure 4.7). Other aspects of the test setup were the same as for run No. 8. In addition, two floor-stand-mounted displacement dials were used to detect lateral movement of the test frame box beam and the top loading plate (figure 4.7). The lateral movements of both of these parts were very small. At the maximum test load of 4700 lb/ft (68.6 kN/m) with accompanying specimen deflection of 0.91 in (23.1 mm) the test frame box beam had shifted laterally less than 0.004 in (0.1 mm) and the loading plate with fulcrum, less than 0.05 in (1.3 mm). The accomodation shift of the box beam was obviously insignificant. The load plate shift; though minimal insofar as causing an inclined (instead of vertical) load is concerned, does illustrate the desirability of mounting a deflectometer directly on the specimen rather than on a floorstand to measure net deflection of the specimen.

The removal of the "inner" steel containing angle at top and bottom (figure 4.7) was an improvement on the end loading conditions. The remaining two "outer" angles (one each at top and bottom) were sufficient for containing the ends of the wall panel up to the maximum test load. The absence of the two angles on the concave side eliminated cracking of the gypsum board interior face when the wood plates experienced crushing-rotation (figure 4.7). The edge cracking at the top of the gypsum board (seen in the same figure) was a consequence of the closure of the 1/4-in (6.3-mm) gap, provided by the shim plates to avoid edge bearing of the facings when that space was closed by crushing-rotation of the wood plates.

4.10 Test Run No. 10 (STLS) - This was the first test to make use of a light-gage steel stud frame wall panel described in section 2.1.2. The instrumentation and loading procedure were the same as those used for test run No. 1. Hardware at both the top and bottom included inner and outer containing angles and fulcrum load points. The loading eccentricity was t/6 based on the 3 5/8-in (92.1-mm) depth of the Z studs (figure 2.2). There was no need for shim plates to prevent facing edge bearing as in previous tests because the same purpose was served by each of the 2 x 4's attached flat against the webs of the top and bottom horizontal Z plate members, (fig. 4.8). It can be seen in the same figure that a stud bottom tab was close to bearing (a condition which would also occur in use); the protruding 1 1/2-in (38.1-mm) flange at the exterior (fiberboard) side of the bottom horizontal Z member also approached this condition. However, this same condition did not exist at the top since the protruding (interior) flange of the horizontal Z section is the shorter (1 1/8 in) (28.6 mm) one. Some of the deflection measurements at the first few increments of applied load indicated negative deformation of the wall (i.e., deflected toward the eccentric load). Apparently the negative deflection was the result of a relative movement of the LVDT coil-supporting tube (with respect to the wall-mounted core) caused by loading accomodations of the horizontal 2 x 4's to which the tubes were attached (figure 4.8).

The x-y record showed some load drop-off at incremental, observation stops (but without increase in deflection); this was thought to be due to slip of screw connections between sheet metal members. At 3000 lb/ft (43.8 kN/m) there was evidence of crushing of the bottom 2 x 4 directly under the web-edge of an outer Z stud (i.e., at the folded end of the web of the horizontal Z member). This folded end of the web of the horizontal Z member was also being bent outwardly. In loading from 3500 to 3800 lb/ft, (51.1 to 55.5 kN/m) there was web buckling of the Z studs visible at the top edges of the wall panel. The corresponding erratic x-y deflection record was an indication of the influence of local deformation at the ends of the wall on apparent deflection measurements. This was confirmed by the computer load-deflection plot.

4.11 Test Run No. 11 (STLS) - A new steel Z stud frame wall panel was used in this run, the procedure for which duplicated the preceding run (No. 10), except for two features: (a) one deflectometer supporting tube was mounted over a shorter portion of the wall height; and (b) the upper and lower interior (gypsum board) face containing angles were removed after applying the first load increment.

In an effort to observe any benefit derived from avoiding the effect of local deformation (at the ends of the specimen) on the suspension points of the deflectometer support tube, the ends of one such tube were supported on the Z stud web at 3 1/2 in (88.9 mm) from each wall end instead of the previous 3/4 in (19.1 mm) (i.e., the mid-thickness of the horizontal 2 x 4's).

The specimen was loaded incrementally to a maximum of 2300 lb/ft (33.6 kN/m). At 1750 lb/ft (25.5 kN/m) the ends of the outer Z studs, at all 4 corners of the wall were crushing the webs of the horizontal Z members into the 2 x 4's on the interior side of the wall. At 2300 lb/ft (33.6 kN/m) there was general web buckling near the top and bottom ends of the Z studs. Contrary to the usually expected behavior, the deflectometers referenced to end points located farther away from the top and bottom 2 x 4's indicated significantly more transverse deflection (approx. 3 times) than those referenced over the greater wall height. The most likely explanation for this anomaly is that the latter deflectometers were influenced by local crushing at the points of suspension. The remaining outer angles at the ends of the wall provided satisfactory containment of the specimen throughout the test (see figure 4.9).

4.12 Test Run No. 12 (GFRP) - The specimen for this test was a fiberglass-reinforced plastic panel with bonded 2 x 4 wood plates at top and bottom as described in section 2.1.3. Before testing, the plastic facings and U-section legs were cut back 1/4 in (6.3 mm) from the bearing surfaces of the top and bottom wood plates to ensure that the facings were not in direct load bearing (figure 4.10). Fulcrum loading fittings and containing angles were used at the top and bottom of the panel. A 1/16-in (1.6-mm) sheet of rubber was inserted between the angle legs and the specimen to provide better surface contact (especially on the exterior sprayed-aggregate surface). The loading eccentricity was set at t/6 (referenced to the full thickness) toward the interior wall surface. The interior face of the panel was quadrilled to facilitate observation of dimpling on the surface. Deflection of the specimen was measured by 5 LVDT's located along each edge at the following elevations: mid-height, quarter-points and 4 in (0.10 m) from each end. The span of the LVDT pivoted supporting tubes was 93 in (2.4 m). Load was applied in 500-lb/ft (7.3-kN/m) increments to a maximum of 5400 lb/ft (78.8 kN/m) where failure occurred by sudden loss of adhesive bond between the interior facing and the core corrugations, followed by wall buckling (figure 4.10). Although relatively large magnitudes of deflection (1.3 in) (33 mm) and end rotation occurred, the end-fittings functioned satisfactorily. The fixtures contained the wall ends and the fulcrums remained in the notches.

4.13 Test Run No. 13 (ACUP) - A foamed urethane core sandwich panel, of the type described in section 2.1.4, served as the specimen for this test run. Fulcrum plate and angle fittings were used at both top and bottom of the wall panel specimen. To achieve better contact surfaces, the aggregate on the exterior surface of the panel was chipped away at the locations of the containing angles (as well as at the locations of the deflectometer reference plates for better adhesive bonding) and a double thickness of 1/16-in (1.6-mm) sheet rubber was used as an interface. Although the wall height was less than the nominal 8 ft - 94 1/2 in (2.4 m) - the 5 LVDT's at each wall edge were located at the mid-height, 2 ft (0.60 m) above and below mid-height, and at 4 in (0.10 m) from the ends; the length of the deflectometer supporting tubes was 90 in (2.3 m). The loading eccentricity was t/6 toward the interior wall surface (plywood) and was based on a nominal wall thickness of 3 in (76.2 mm) -- the depth of the surrounding extruded aluminum frame. No compressometers were used.

The specimen was loaded in increments of 500 lb/ft (7.3 kN/m) to a maximum of 11,500 lb/ft (167.8 kN/m) with an accompanying mid-height deflection of more than 1 1/4 in (31.8 mm). At that point a twisting separation of the aluminum frame from the body of the wall at mid-height was followed by explosive tensile cracking of the wall. The sudden release of energy caused the failed specimen to spring out from under the loading apparatus (figure 4.11). The same figure includes a view of the wall after recovery.

It was observed during the test that it is important to accurately orient the deflectometer LVDT's parallel to the direction of deflection to avoid binding of the cores in the coils when the core ends are held in place against the reference plates by rubber bands and to avoid an oblique measurement of the deflection.

4.14 Test Run No. 14 (GPHC) - This test run used a paper honeycomb sandwich panel of the type described in section 2.1.5. However, this particular specimen had a thinner exterior surface of a proprietary material similar to, but more dense than gymsum board; the overall thickness of this wall panel was 4 in (0.10 m). The test setup included fulcrum load point assemblies, containing angles and 1/4-in (6.3-mm) face-bearing relief shim plates (the facing boards were virtually flush with the bearing surfaces of the laminated wood blocks) both at top and bottom of the specimen. The loading eccentricity was t/6 based on the full 4-in thickness of the panel. Only transverse deflections were measured, with LVDT's located along both panel edges at the mid-height, 1/4-points and 4 in (0.10 m) from top and bottom (figure 4.12). The length of the LVDT support tubes was 92 3/4 in (2.4 m). Loading of the specimen in 250-lb/ft (3.6-kN/m) increments, with short stops for measurement scans, provided a relatively trouble-free, smooth x-y load-deflection record and corresponding digital data. The failure of the panel was sudden at a load of 6850 lb/ft (99.9 kN/m) by delamination of the gypsum board from the woven roving and by wall buckling, at a location just above the bottom plywood block, on the compression side (figure 4.12). The failure permitted abrupt additional rotation of the bottom fixture with consequent secondary cracking of the panel faces near the vertical legs of the containing angles.

4.15 Test Run No. 15 (STLS) - Beginning with this test run a group of five tests (Nos. 15--19) were conducted using specimens of the various proprietary types tested previously but employing flat bearing conditions at the bottom, without either containing angle. This bearing condition was described in section 2.2.2. Fulcrum loading, including use of both containing angles in contact with outer and inner wall surfaces, was used at the top. All 5 tests made use of only deflectometer-LVDT's (no compressometers) located along both panel edges at mid-height, 2 ft (0.60 m) above and below mid-height, and at small distances (given in each test discussion) from the top and bottom ends.

Run No. 15 (STLS) was made on a panel of steel-frame construction described in section 2.1.2. The loading eccentricity was t/6 toward the inner (gypsum board) face, based on the 3 5/8-in (92.1-mm) depth of the Z stud (figure 2.2). The LVDT supporting poles were pivoted at 3 1/2 in (88.9 mm) from the top of the wall and rested in V-guides at 5 in (0.13 m) from the bottom; tneir 87 1/2-in (2.2-m) span was totally within the upper and lower wood plates (i.e., on the Z-stud web). The outermost deflectometers were 7 in (0.18 m) from the bearing ends of the panel. The specimen was loaded in increments of 250 lb/ft (3.6 kN/m) to a maximum of 3150 lb/ft (46.0 kN/m). The maximum lateral displacement reading was a negative value, -0.03 in (-0.76 mm), indicating movement toward the gypsum board face. This apparent negative translation is opposite to that expected for the direction of load eccentricity. While it was observed that the studs experienced very little bowing, it was concluded that the sign reversal was the result of the local distortion observed at the top of the studs. The localized deformation caused top plate rotation (fig. 4.13) and probable shifting of the pivots at the end of the LVDT support poles. This explanation was supported by the erratic nature of the x-y recorder plot and of the digital data.

4.16 Test Run No. 16 (GPHC) - The specimen for this test was a paper honeycomb sandwich panel described in section 2.1.5. The loading eccentricity for the pin-flat setup was set at t/6 - with respect to the full thickness, 4 3/8 in (0.11 m), of the panel - toward the interior face. The pivots for the deflectometer LVDT support poles were attached 1 1/4 in (31.8 mm) from the top and the V-guides, 2 1/4 in (57.1 mm) from the bottom thus making the poles span from upper to lower wood blocks. The LVDT's which were farthest from the mid-height, were at 4 in (0.10m) from the top and bottom ends. Bearing of the gypsum board facings was prevented by use of 1/4-in shim plates over the wood block bearing areas. Comparative measurements of lateral displacement of the middle of the interior face were made with a rack and pinion dial mounted on a floor stand in 250-lb/ft (3.6 -kN/m) load increments up to a maximum of 6900 lb/ft (100.7 kN/m); the maximum deflection was approximately 1/4 in (6.3 mm). Failure occurred at the top wood block by separation from the facing (figure 4.14). Subsequent rotation of the top end caused the exterior gypsum board to crack. The displacement dial readings generally agreed with the average of the mid-height LVDT's for the first few load increments but for the remainder of the test, they were repeatedly, but not systematically, higher. At several scanning stops the waiting period was deliberately extended to two minutes to observe the effect on the test setup. Within the range of linear response (i.e., up to 5000 lb/ft) (73.0 kN/m), load drop-off ranging from 100 to 300 lb/ft (1.5 to 4.4 kN/m) was observed. This load drop-off was a function of the hydraulic system and was not accompanied by creep deflection. Beyond the linear range, load drop-off during extended waiting was accompanied by creep-deflection.

4.17 Test Run No. 17 (GFRP) - This test run was made on a fiber glass-reinforced plastic panel described in section 2.1.3. The pin top-flat bottom arrangement, which included bearing shim plates, had a loading eccentricity of t/6 toward the smooth interior face which was quadrilled for better observation of surface deformations (figure 4.15). The upper-and lower-most LVDT's were at 4 in (0.10 m) from the ends of the wall; the LVDT-supporting poles extended from 1 in (25.4 mm) below the top to 2 1/2 in (63.5 mm) above the bottom, the wall height being 96 1/2 in (2.5 m). Loading was in 500-lb/ft (7.3-kN/m) increments. Figure 4.15 shows the panel during testing at a load of 4900 lb/ft (71.5 kN/m) after the test when failure had caused the bottom end to suddenly shift laterally. Failure occurred by buckling bond-separation of the exterior face from the corrugated core at about 1 ft (0.30 m) up from the bottom. It should be noted that, being a flat-end bearing, the bottom end was partially restrained from rotation. The bottom portion of the wall where the failure developed was probably well below the inflection point of a column member under such loading. The result was that the exterior face was quite apparently under compression. Buckling in this direction caused the bottom of the wall to suddenly translate in the direction of the interior surface, carrying with it the bottom shim plate which is seen up-ended in figure 4.15. Shim plates were used with this type specimen, in this test to avoid cutting back the facings near the bearing ends.

4.18 Test Run No 18 (ACUP) - A foamed urethane core sandwich panel was set up as seen in figure 4.16. A load level of 10,800 lb/ft (157.6 kN/m) was recorded just prior to failure. Based on the experience with a similar specimen in test run No. 13, safety bars were installed (figure 4.16) to prevent a possible complete overturning of the panel at failure. The loading eccentricity used was 1/6 the wall thickness of 3 in (76.2 mm). Other details of the test setup duplicated run No. 13 where applicable; as in run No. 13, load bearing was directly on the surrounding aluminum frame. In summary, the major difference between this run and run No. 13 was the use of flat load bearing at the bottom as opposed to fulcrum load bearing in run No. 13. In progressing by load increments of 500 lb/ft (7.3 kN.m), popping noises suggestive of incipient failure were heard at 8750 and 9800 lb/ft (127.7 and 143.0 kN/m). Failure occurred at 10,950 lb/ft (159.8 kN/m) by a twisting separation of the aluminum frame from one edge, near the bottom of the wall, followed by a tensile crack and spalling of a large wall section (fig. 4.16). Also visible in fig. 4.16 is the faulty void in the foamed urethane core which caused the failure at that location. Consequent displacement of the wall as a whole was limited to a lateral shift of the bottom end (fig. 4.16). Maximum deflection of the panel at failure was nearly 1 in (25.4 mm).

4.19 Test Run No. 19 (GFRP) - The specimen for this test was a fiber glass-reinforced panel which is described in section 2.1.3. For this test the loading eccentricity was adjusted to one-half the wall thickness, placing the load line in the plane of the interior surface. Except for the greater eccentricity the test setup was the same as that for run No. 17. During the test, oblique flood-lighting was used to give prominence to surface deformations (fig. 4.17). The panel was loaded in increments of 250 lb/ft (3.6 kN/m) to a maximum of 4750 lb/ft (69.3 kN/m). Maximum deflection at failure took place gradually by bond separation of the facing on the compression side near the top. The three photographs in figure 4.17 show the panel as maximum load was reached and upon subsequent failure.

4.20 Test Run Nos. 20-25 (WFTH) - In performing the next 6 tests, additional objectives included observing the effect of a difference in the method of testing on the general performance and mode of failure of a given type of construction, and observing the reproducibility of results in replicate tests. To do this, 2 groups of 3 test runs were performed. Test run Nos. 20-22 permitted direct load bearing on the wall panel facing elements; test run Nos. 23-25 prevented it. These last 6 test runs made use of wood-frame construction panels described in section 2.1.1. The panels, designated WFTH, contained 3 studs spaced symmetrically about the centerline of the wall at nominal 16-in centers. The 3-stud design was chosen over the 4-stud (WFFR) arrangement as being more representative of a 4-ft (1.2-m) width of this type of construction. In such a 3-stud specimen the outermost 8 in (0.20 m), of the facings at both edges of the panel were left unsupported. To compensate for the lack of continuity afforded the facings between studs, the vertical edges were braced by wood spacers. The spacers, 3/4 x 1 1/2 in (19.1 x 38.1 mm), were cut to match the stud depth, 3 1/2 in (88.9 mm), and attached by single, centered 4d wire nails through each facing (figure 4.18). The specimens used in the 6 tests were alike in all respects except one. The 3 panels for test run Nos. 20-22 had facings which were flush with the bearing surfaces of the top and bottom wood plates while the panels for the remaining 3 test runs (Nos. 23-25) had their facings cut back 3/8 in (9.5 mm) from the bearing surfaces at top and bottom.

The loading procedure, using a loading eccentricity of t/6 based on the gross wall thickness, was the same for all 6 tests and was as described in section 2.2, except that in the following tests the upper and lower containing angles on the interior side (which became concave) were spaced 1/2 in (12.7 mm) from the wall surface (figure 4.19) to prevent unrealistic localized damage, yet contain the wall-end if necessary. In some of these tests, it was necessary to increase the 1/2-in (12.7-mm) space during the test by repositioning the angles to avoid contacting the bowed wall. The exterior angles tended to become tangent to the convex wall surface at the bearing ends as curvature developed and did not create a problem. The load increment, at which loading was stopped for making measurements, was 500 lb/ft (7.3 kN/m).

The instrumentation was the same for all 6 tests and was of the same general arrangement described in section 2.3 but with the following modifications (see figure 4.18). In addition to the 4 compressometers used in earlier tests (e.g., test run No. 1), 2 others were used--one at the middle of each face, offset 3 in (76.2 mm) from the vertical centerline in opposite directions. Lateral deflection measurements were limited to mid-height locations. In addition to the 2 edge-deflectometers, 2 others were used, one next to each mid-face compressometer, but 3 in (76.2 mm) to the opposite side of the vertical centerline. The mid-face compressometers, as well as the deflectometers, on opposite faces of the panel were located diagonally with respect to each other. The gage length of all face-mounted LVDT support poles was 88 in (2.2 m) and that of the 2 edge-mounted deflectometers, was 94 inches (2.4 m).

Test run nos. 20, 21, 22 further ensured full bearing of the already flush facing edges by having the ends of the panels capped across the gross thickness with a thin embedment of high-strength gypsum (figure 4.19). Capping was accomplished by gravity-bedding each end of a panel, alternatively, against a floor-based, oiled steel plate. Performance of the specimens in these three tests followed reasonably similar patterns. Evidence of failure in test run Nos. 20, 21 and 22 is illustrated respectively by after-test figures 4.19, 4.20 and 4.21. As seen in table 1, maximum load capacities were similar but maximum deflections were not. Compression failures of the gypsum wall board at the tops of the specimens in test run Nos. 20 and 21 are seen in figures 4.19 and 4.20. The beginning of a similar failure was observed at the bottom in test run No. 22, but testing was terminated by splitting of a stud seen in the close-up of figure 4.21. All three tests demonstrated crushing and rotation of the horizontal wood plates. This type of damage is typical of such frame construction. The uniformly distributed, but eccentric, load is concentrated on the ends of the studs which crush the contact surfaces of the horizontal plates off-center, thus resulting in rotation of the top and bottom plates.

Test run nos. 23, 24, 25 were performed with deliberate easement of facing bearing as described earlier in this section. However, to make a fair comparison of the 2 groups of triplicate tests the bearing ends of these panels (i.e., wood-frame plates only) were also capped with gypsum to achieve comparable bearing (figure 4.22). Results of these three test runs are illustrated by figures 4.22, 4.23 and 4.24 respectively. The general performances of these 3 specimens were more similar, within a group, than were those of the facing bearing specimens as is illustrated by the superimposed load-deflection curves in figure 4.25. The curves are the x-y monitor records for one ram and one edge mid-height deflectometer for the earlier stages of both groups of tests before localized damage excessively influenced performance. Table 1 shows the similarity in maximum load capacities of tests 23, 24 and 25. Wall panels in these three tests all experienced closure of gypsum board bearing-relief cut-backs as a result of crushing rotation of the wood plates (see figs. 4.22, 4.23 & 4.24). Test run No. 23 was terminated after load bearing of the gypsum board was followed by cracking of an outer stud at mid-height (figure 4.22). The mid-height crack in the gypsum board was secondary. Extra spacer blocks, visible at mid-height, had been installed to bridge handling damage present at one edge. The damage was not considered contributory to failure. The cracks which are visible in the top wood plate in figure 4.22 are shrinkage cracks. Test run No. 24 repeated closely the performance of No. 23. Figure 4.23 shows the interior stud which split, as well as other evidence of similar behavior. Test run No. 25 also repeated, generally, the performance of the two preceding tests; however, no sudden failure occurred. The load reached a maximum value beyond which it gradually decreased. The test was terminated because the deflection exceeded the translation capacity of the LVDT's and because of the forced displacement of the face-mounted support tubes. This forced displacement was caused by the bowed exterior face at mid-height (figure 4.24).

As an example of the type of load-deflection plot which can be obtained by computer programming, figure 4.26 presents data obtained in test run No. 24. East and west sides refer to panel edges; south face refers to the interior or concave surface.

Digital data provided by the computer processing program were also examined for the last 6 tests (Nos. 20-25) to determine the feasibility of reducing the number of instrumented measurement locations on wall test specimens. Average values of the 4 "edge"-compressometers were compared with the 2 mid-face compressometers and average values of the 2 edge-deflectometers were compared with each of the 2 mid-face deflectometers. Conclusions based on these comparisons are given in the following section.

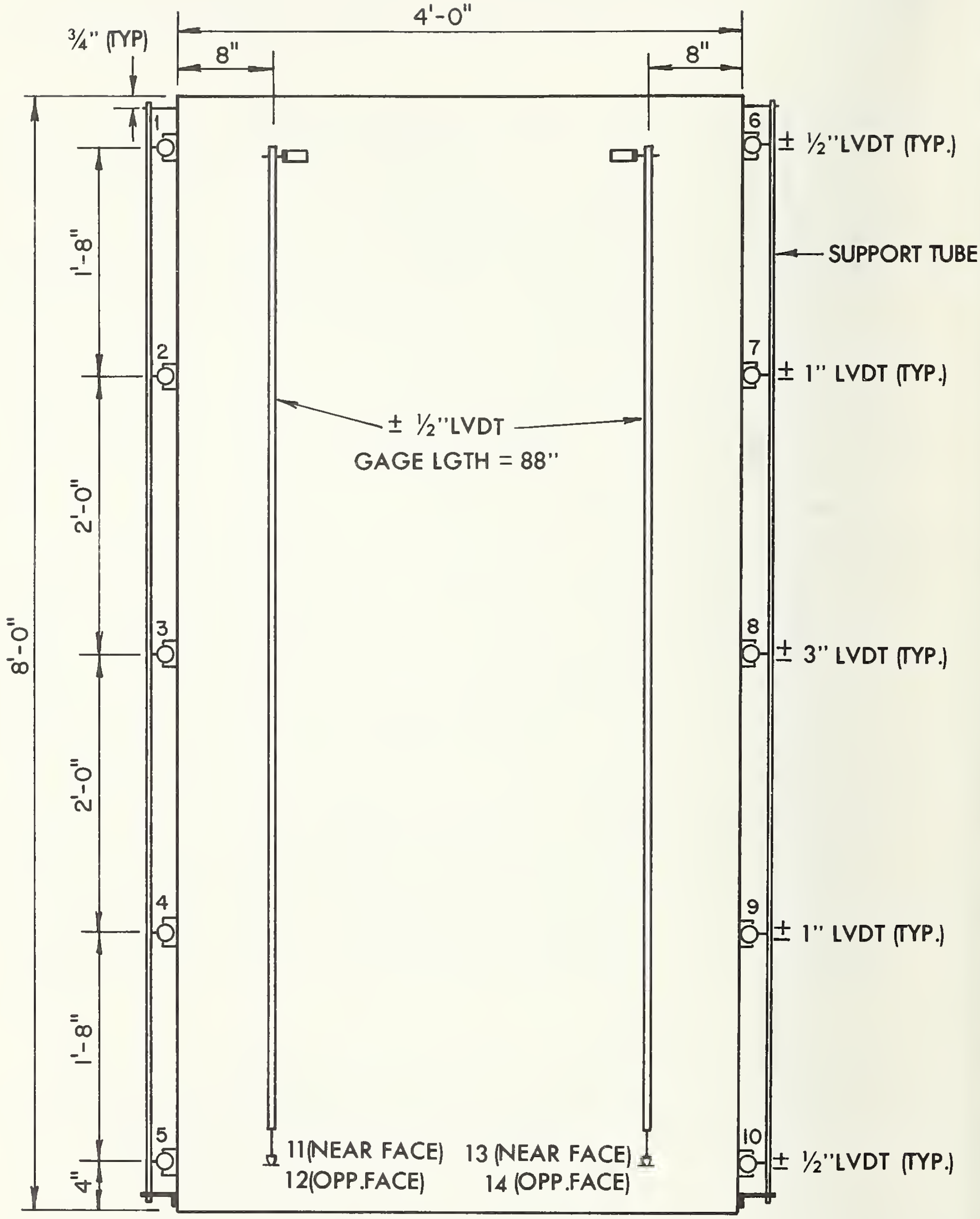


Figure 4.1 - Instrumentation setup for measuring wall shortening and deflection.

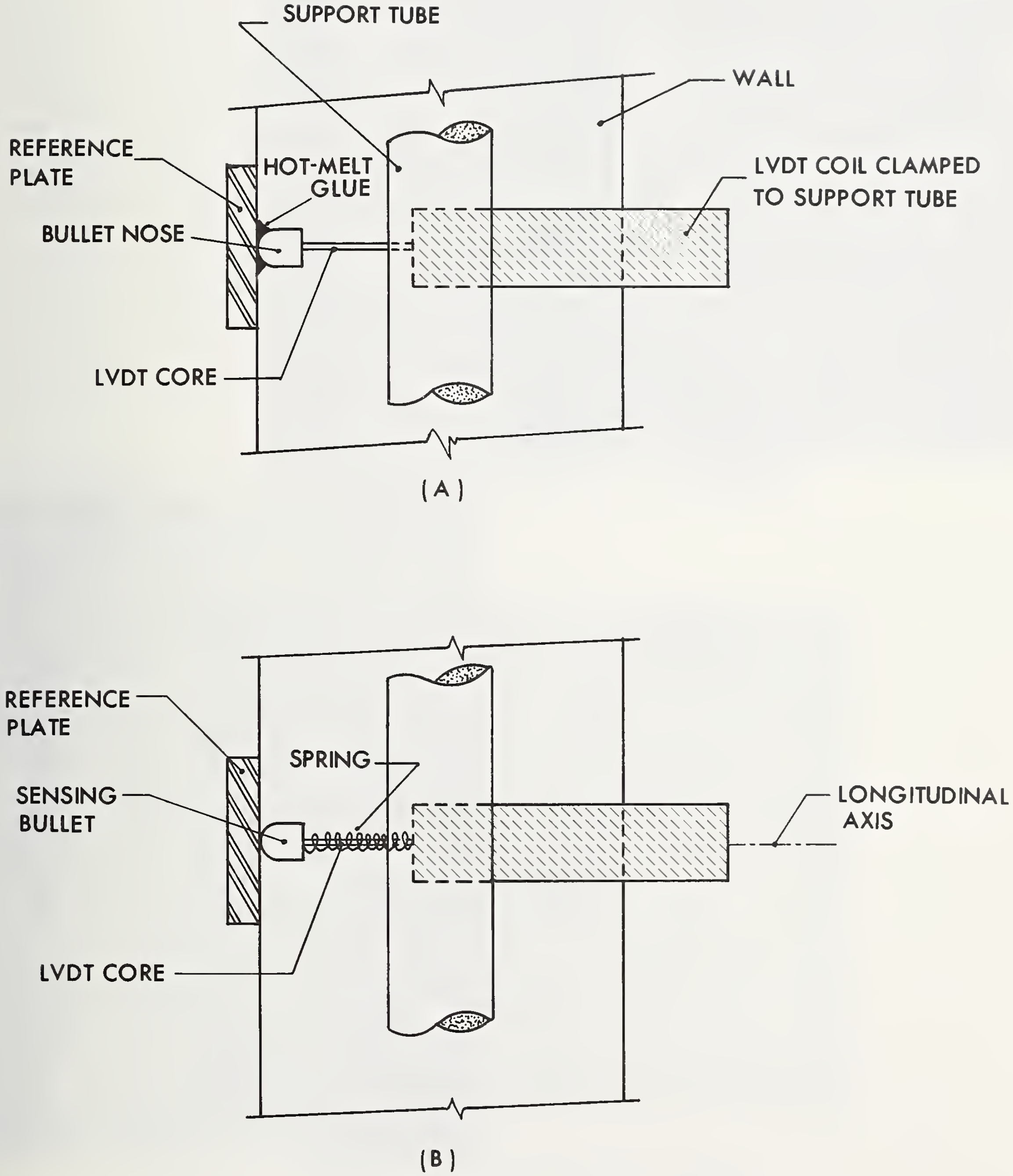


Figure 4.2 - Details of LVDT-core contacting wall-mounted reference plate.

Figure 4.3 - Test run No. 4 (WFFR).

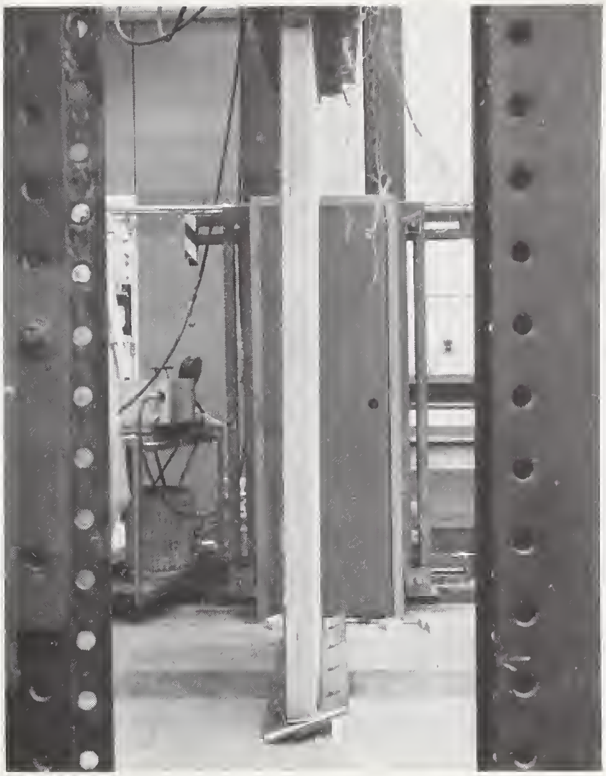
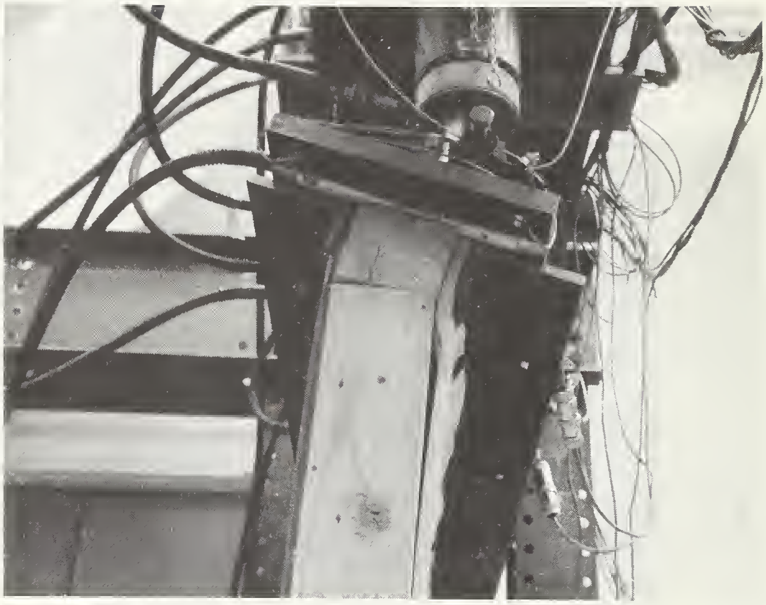


Figure 4.4 - Test run No. 5 (WFFR).



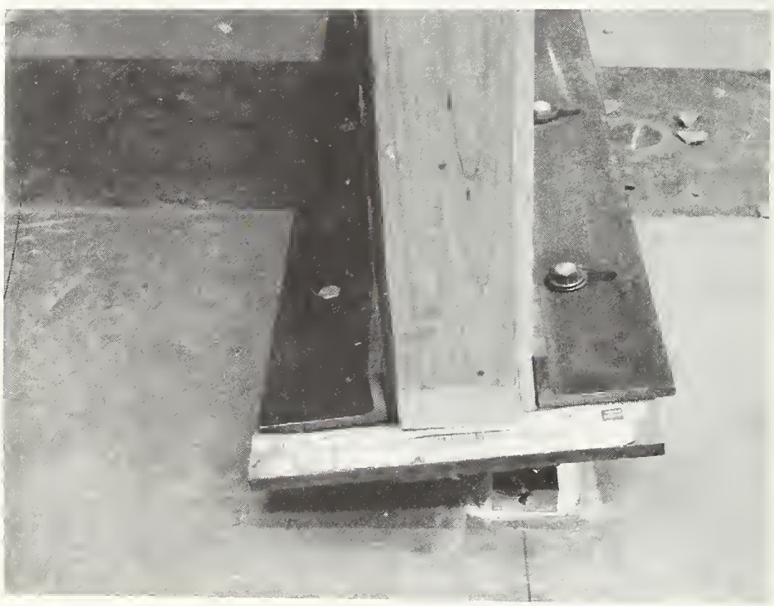


Figure 4.5 - Test run No. 6 (WFFR).

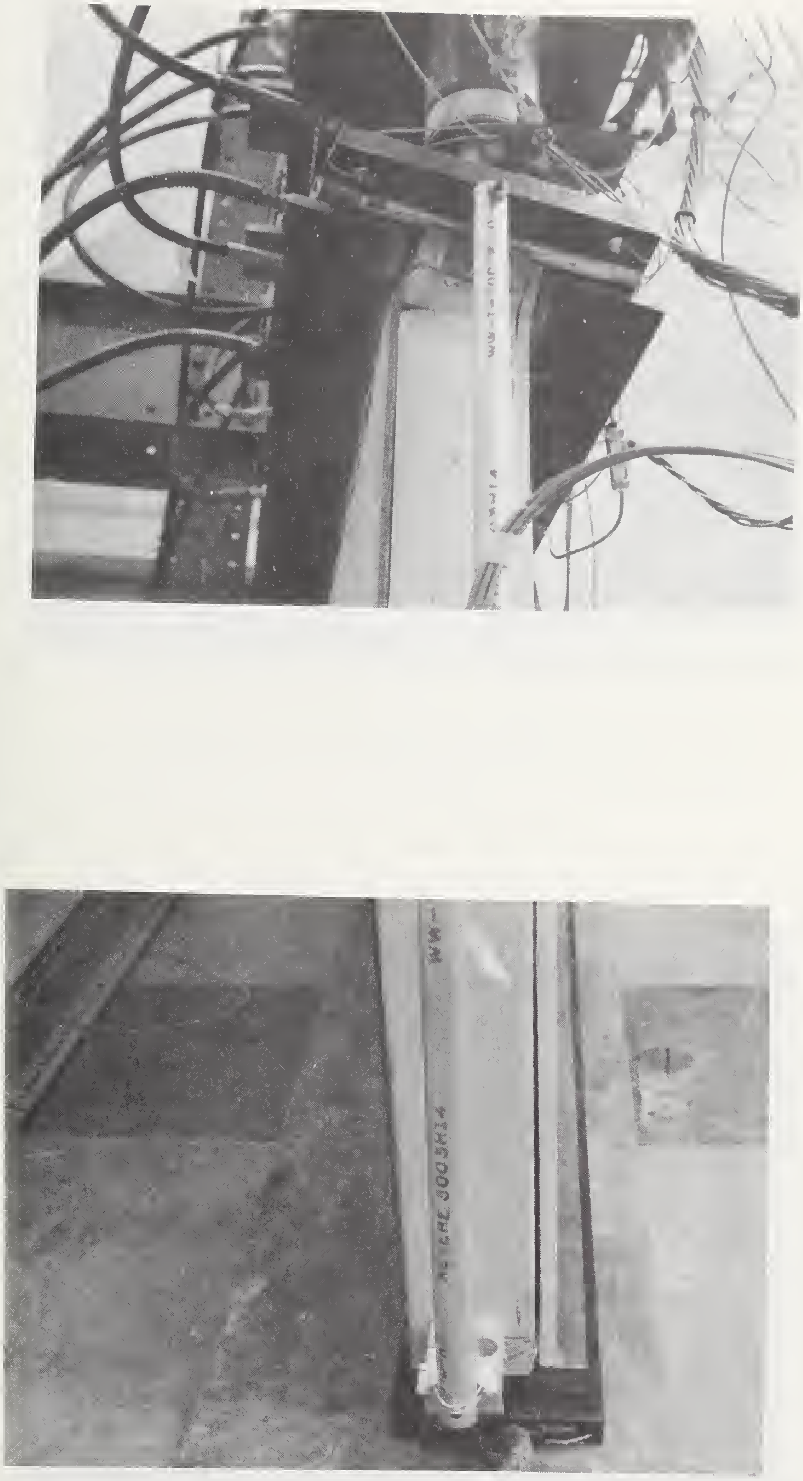
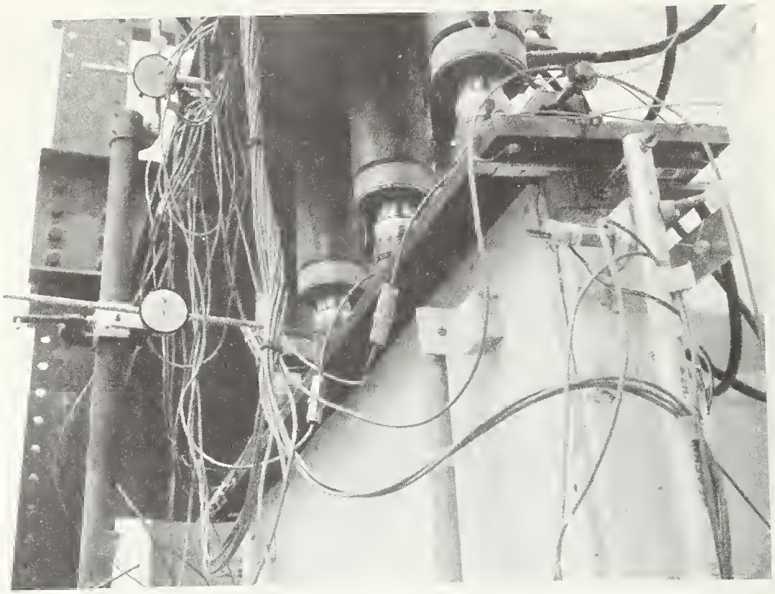


Figure 4.6 - Test run No. 7 (WFFR).



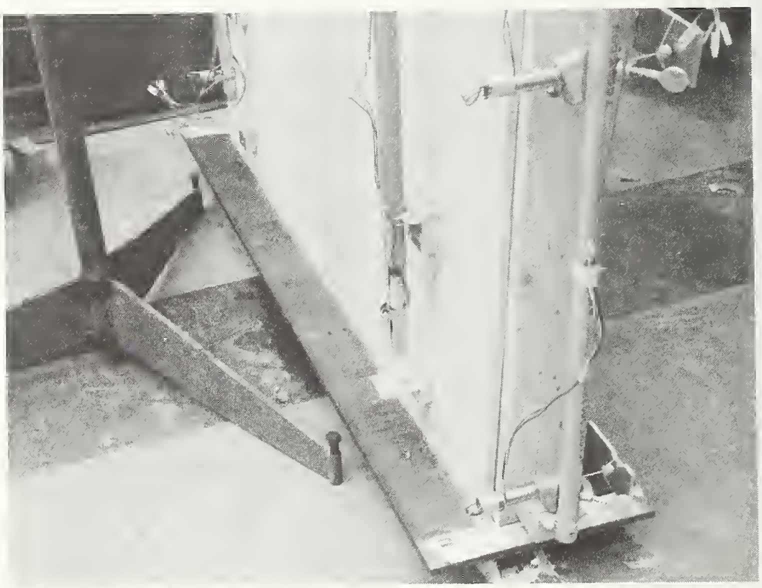
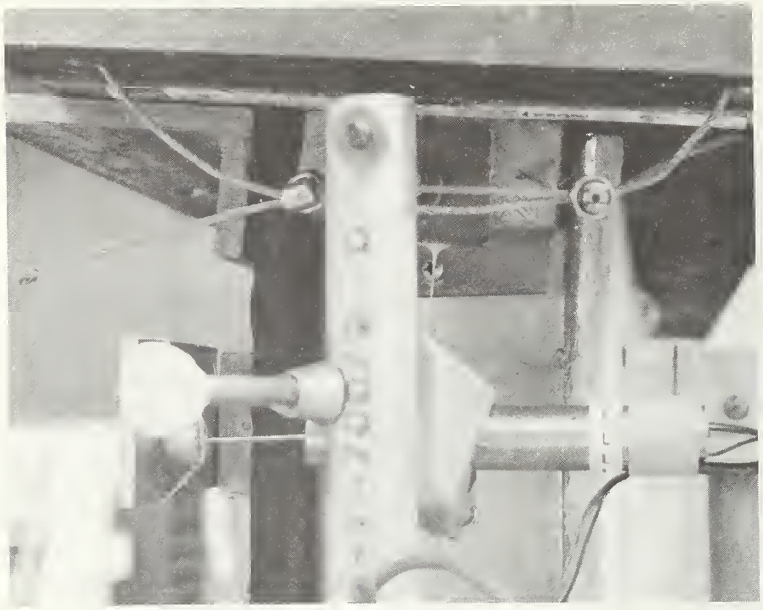


Figure 4.7 - Test run No. 9 (WFFR).

Figure 4.8 - Test run No. 10 (STLS)



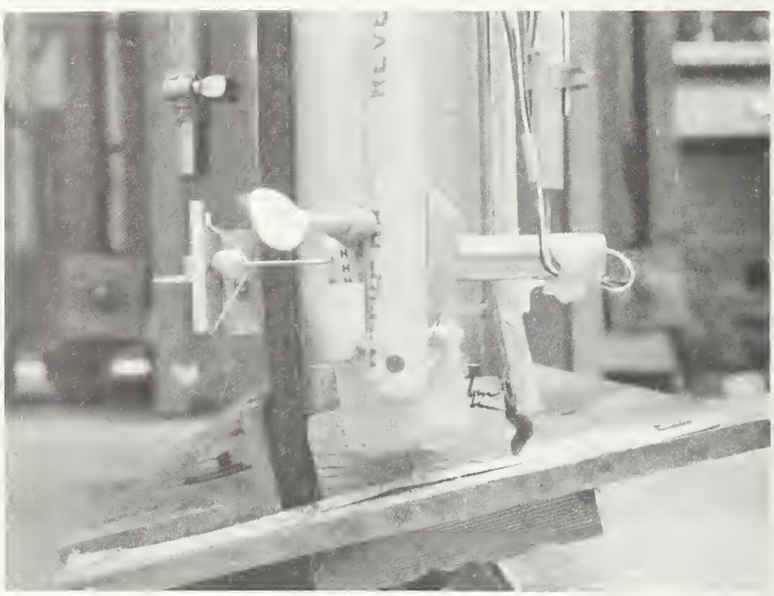


Figure 4.9 - Test run No. 11 (STLS).

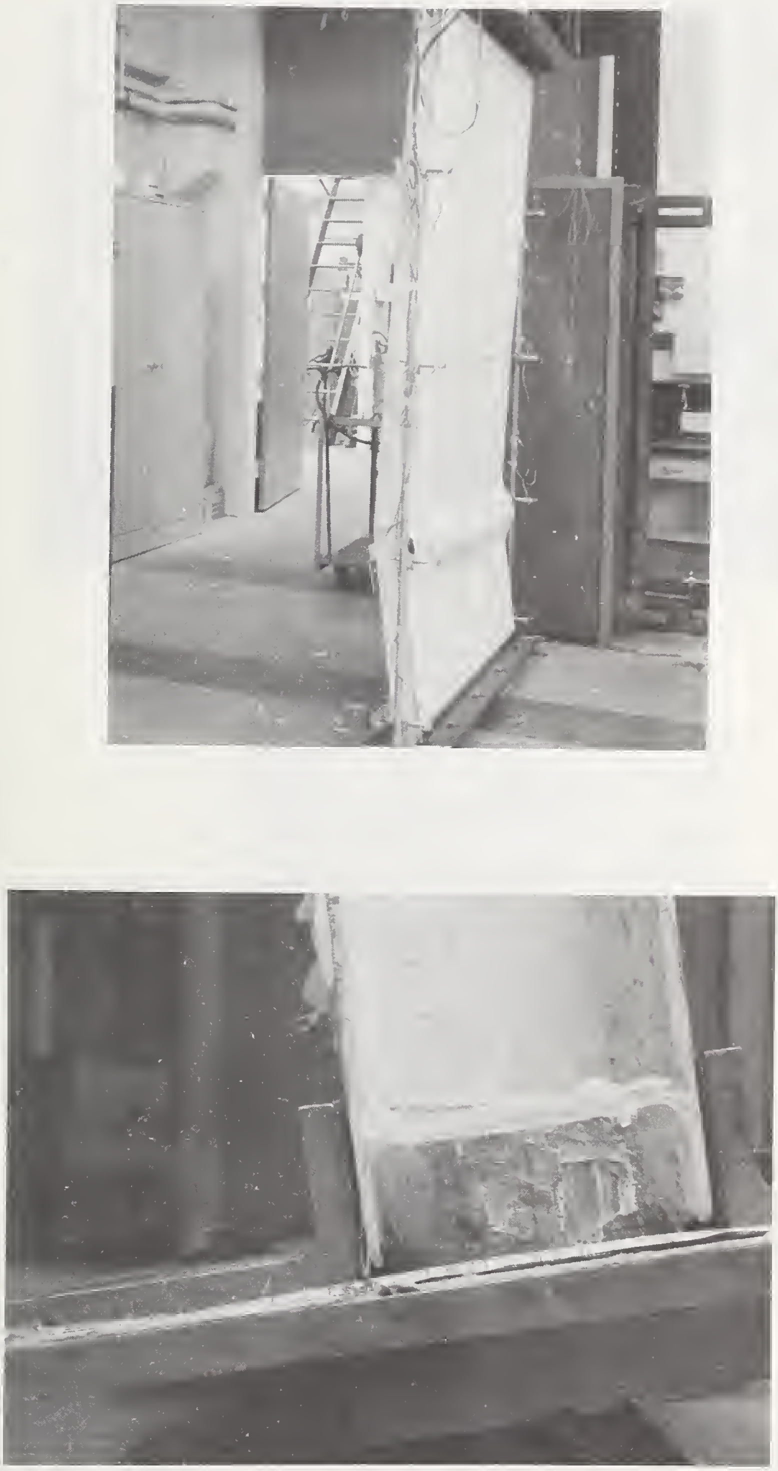


Figure 4.10 - Test run No. 12 (GFRP).

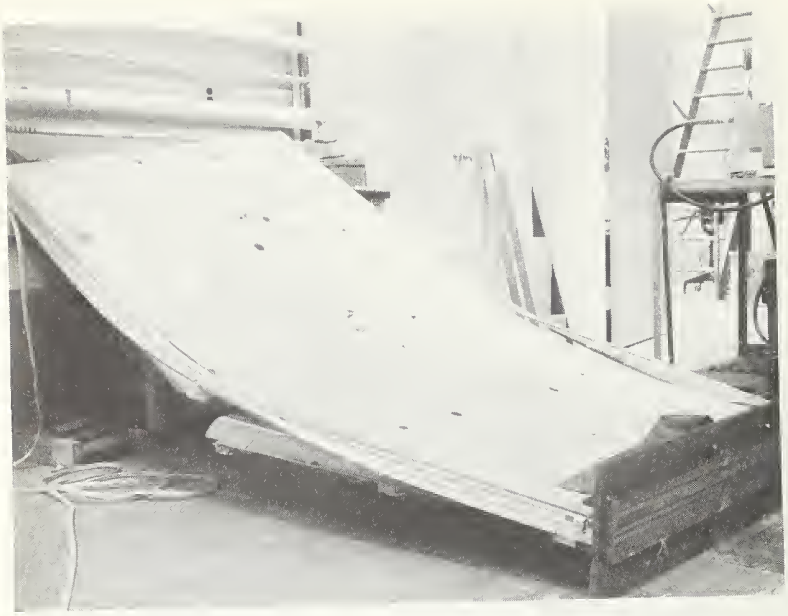


Figure 4.11 - Test run No. 13 (ACUP).

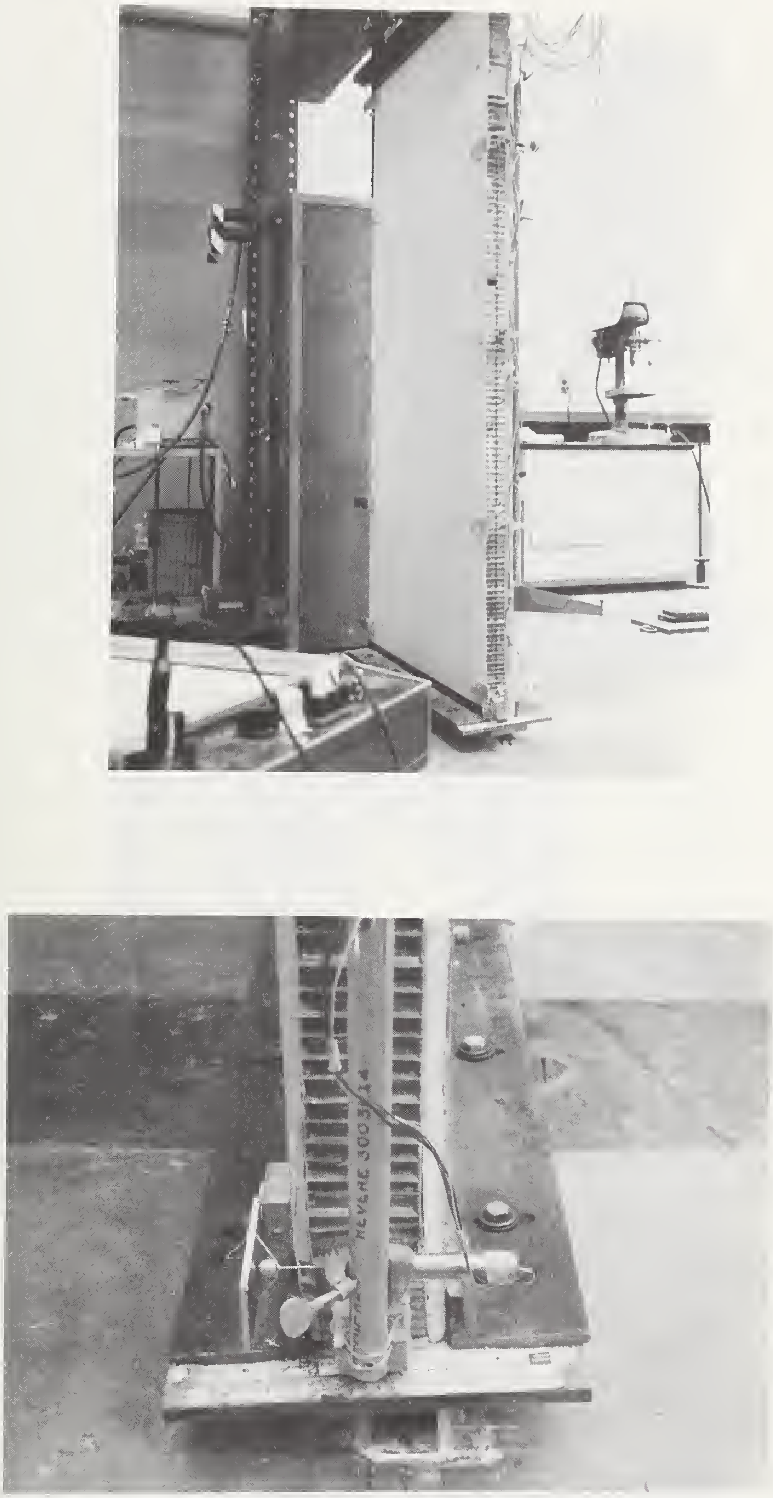
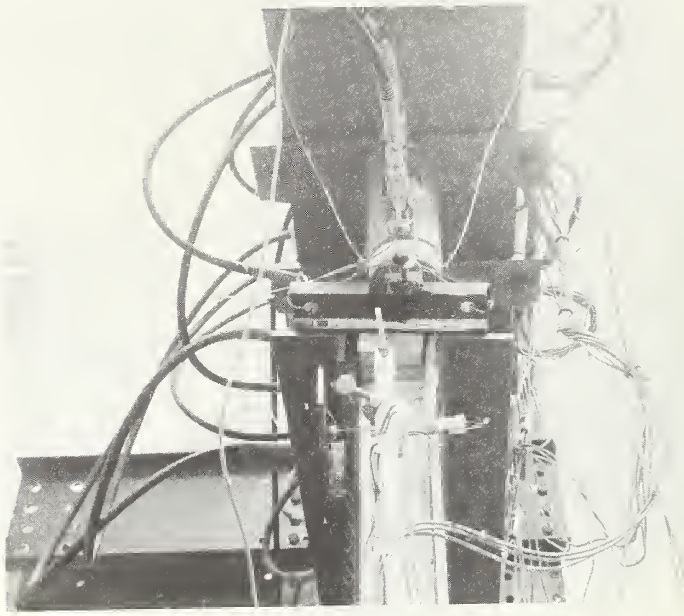


Figure 4.12 - Test run No. 14 (GPHC).



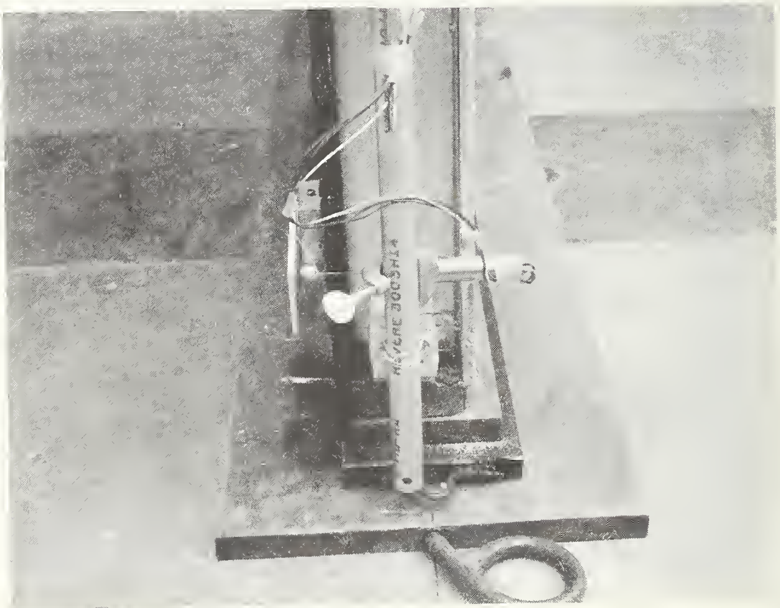


Figure 4.13 - Test run No. 15 (STLS).

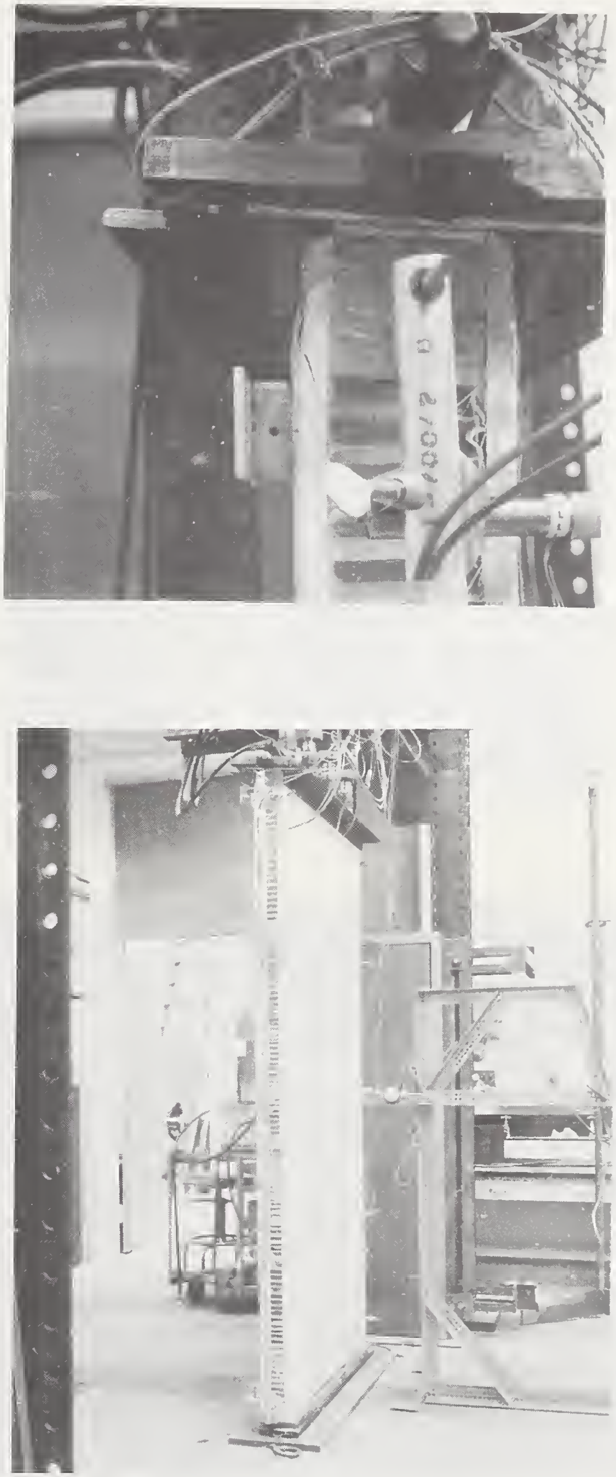


Figure 4.14 - Test run No. 16 (GPHC).

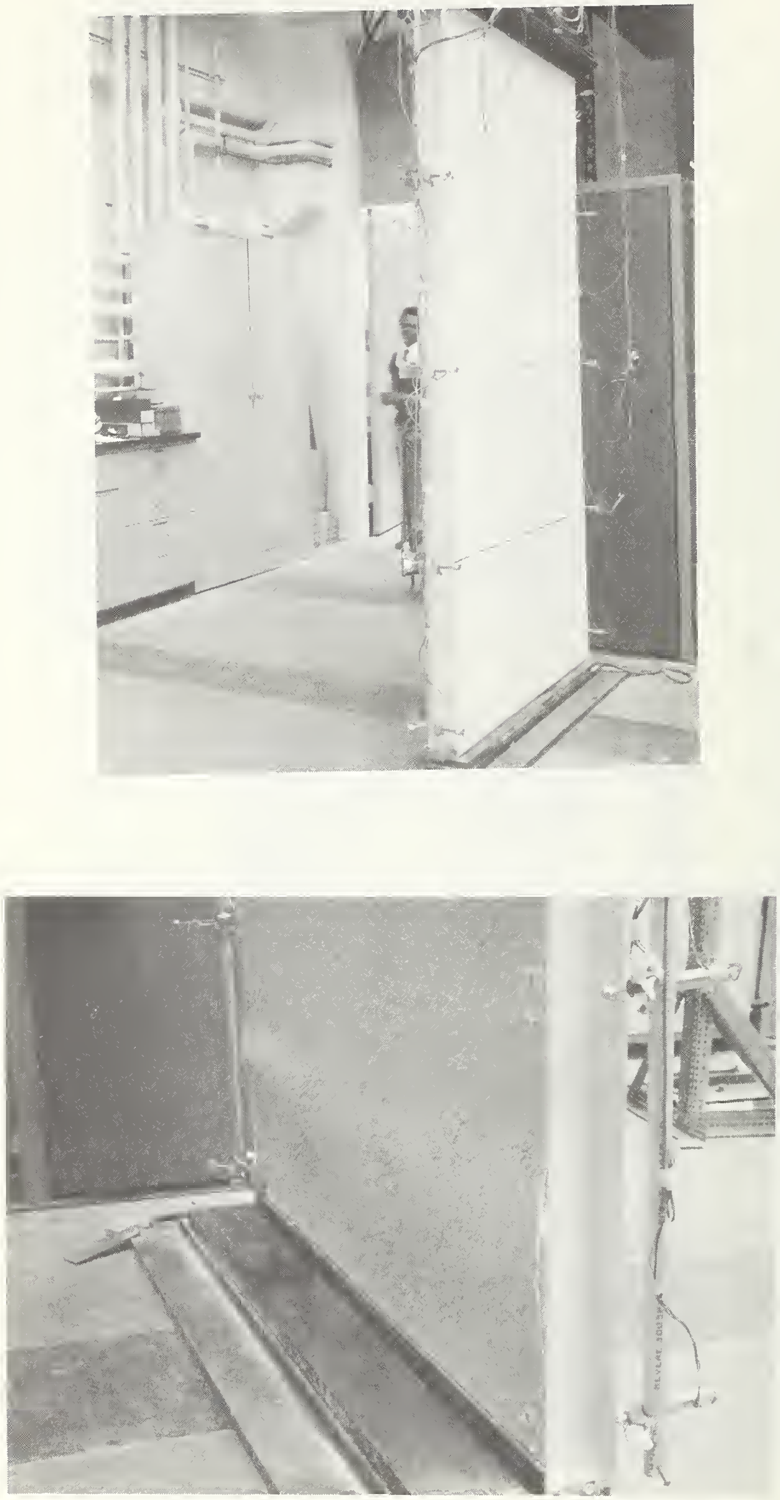


Figure 4.15 - Test run No. 17 (GFRP).

Figure 4.16 - Test run No. 18 (ACUP).

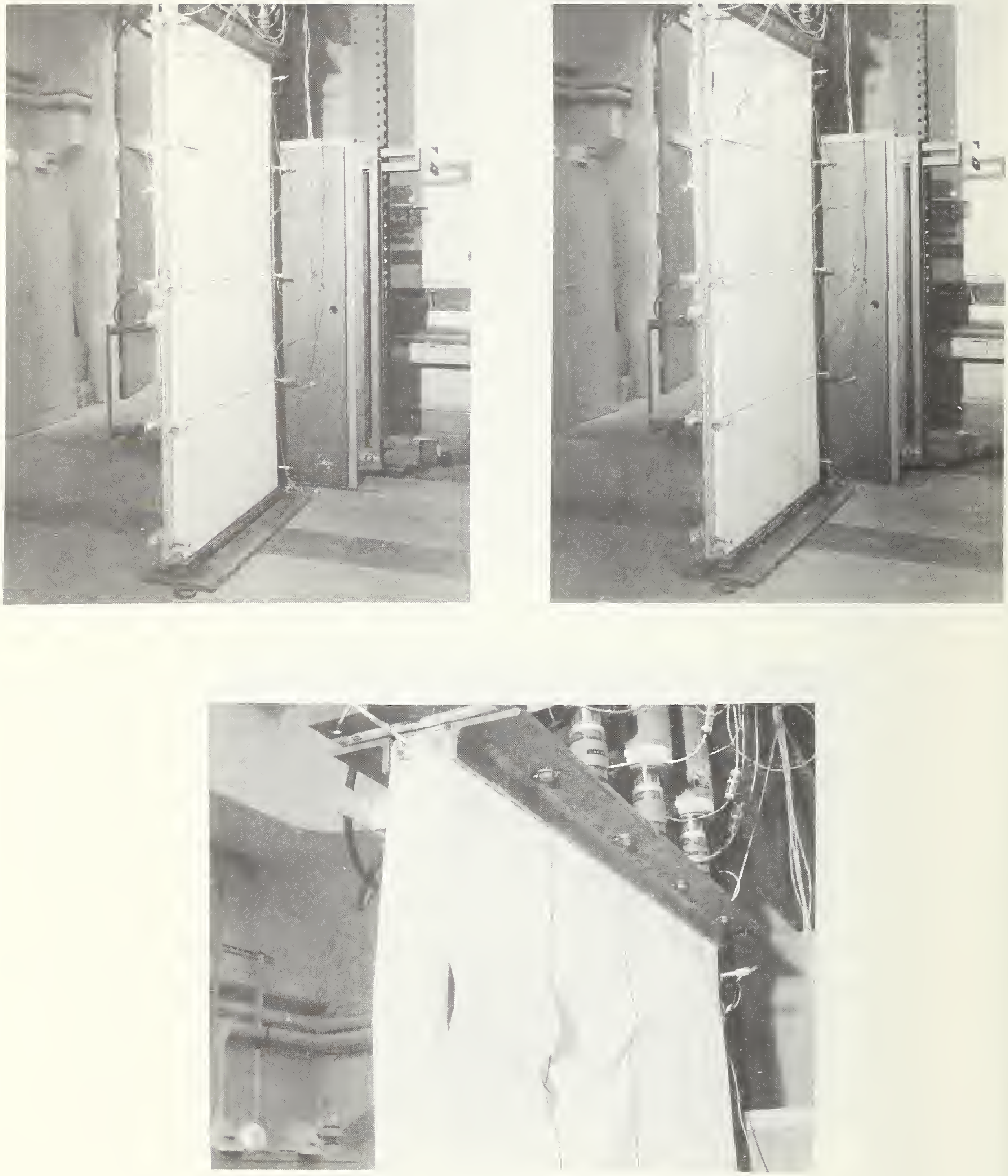


Figure 4.17 - Test run No. 19 (GFRP).

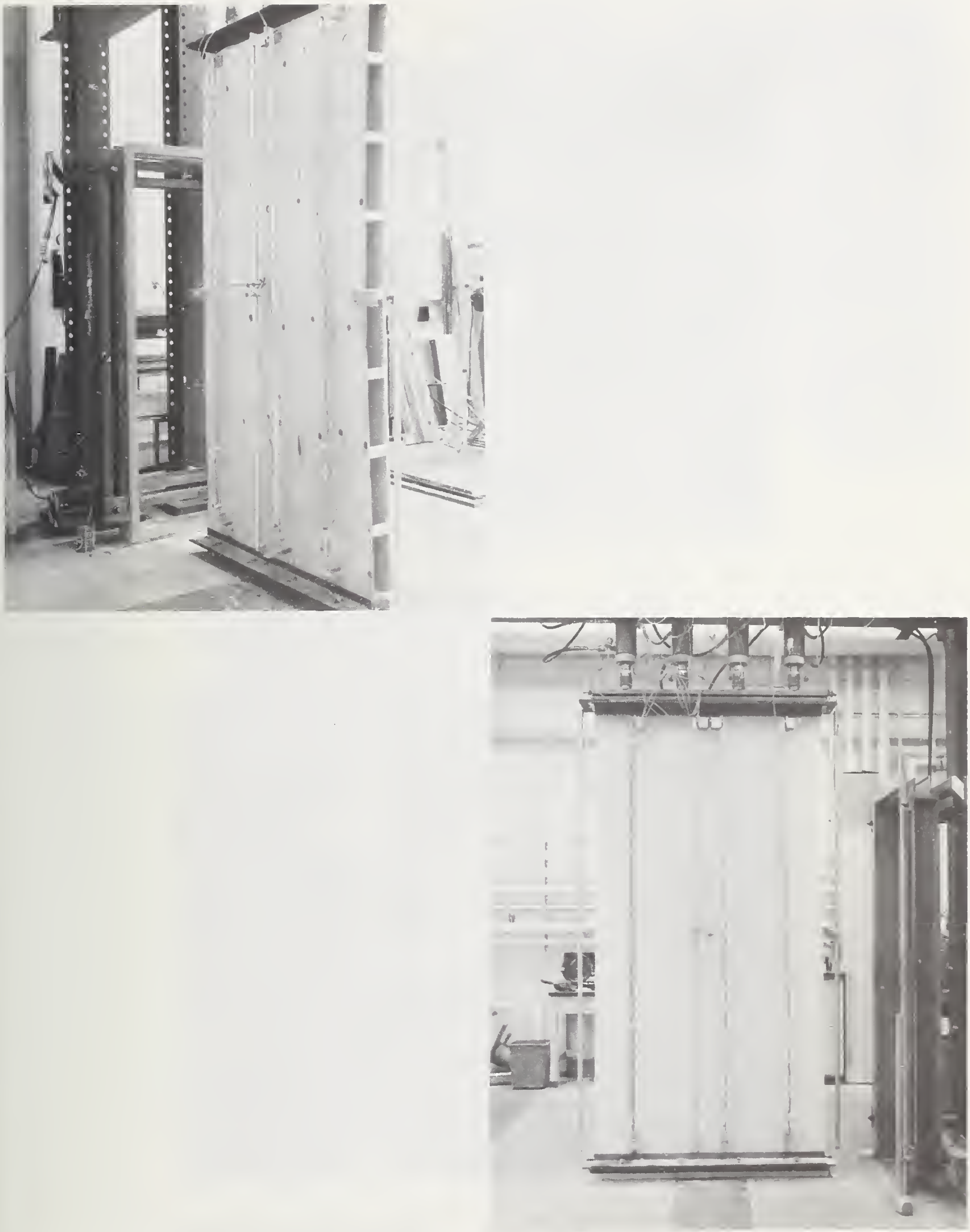


Figure 4.18 - Typical setup of test run Nos. 20 to 25 (WFTH).

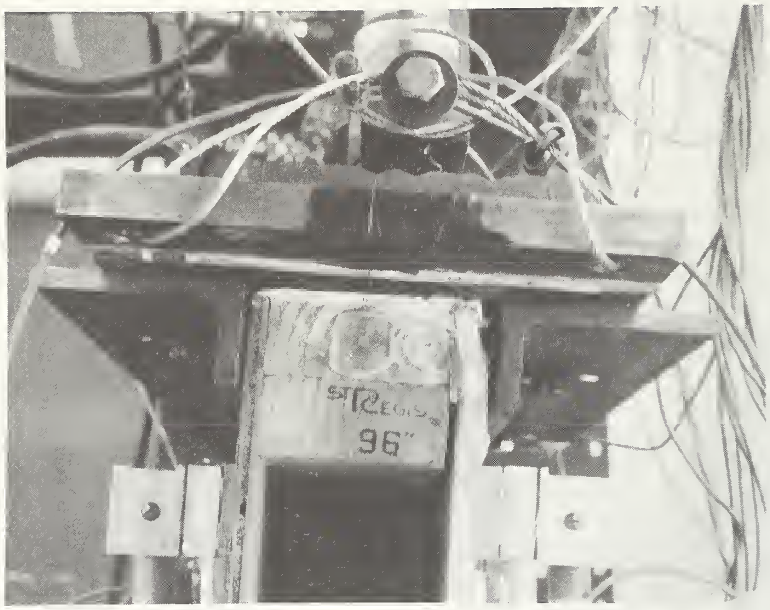


Figure 4.19 - Test run No. 20 (WFTH, face bearing).

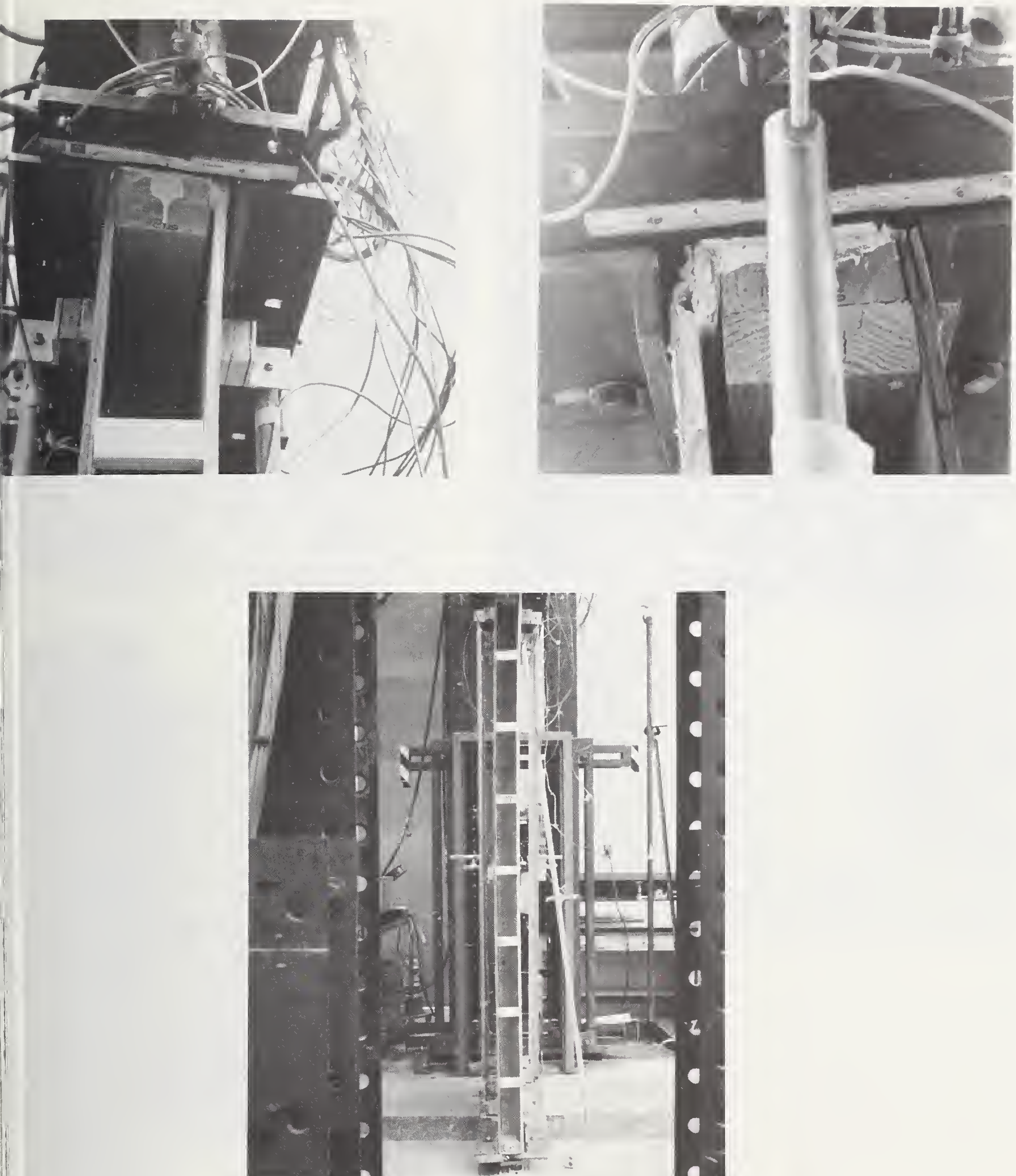
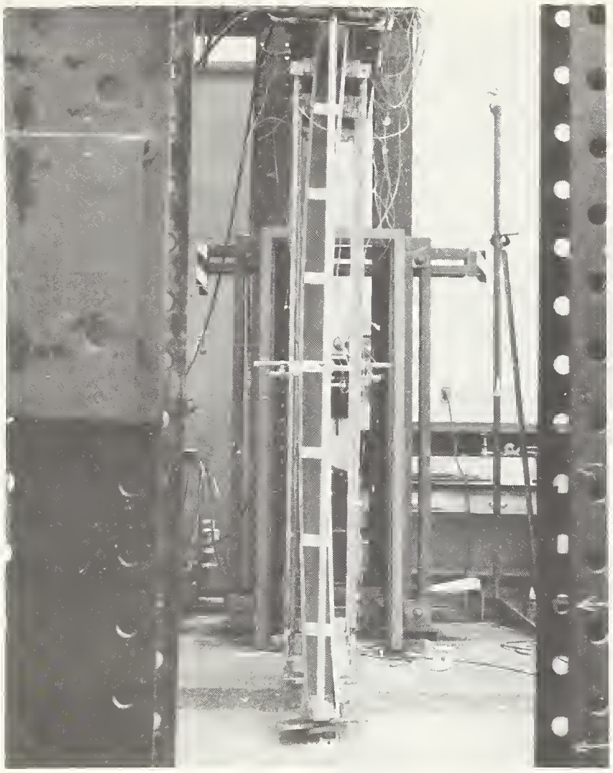


Figure 4.20 - Test run No. 21 (WFTH, face bearing).



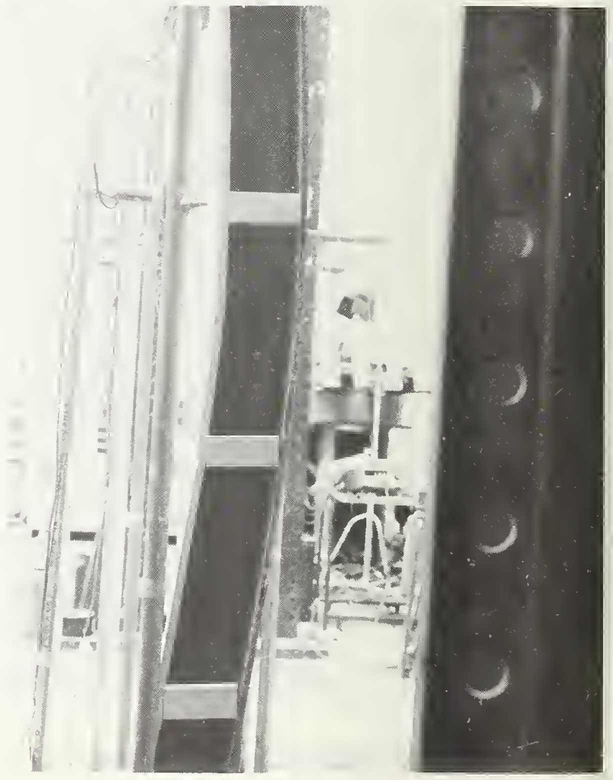


Figure 4.21 - Test run No. 22 (WFTH, face bearing).

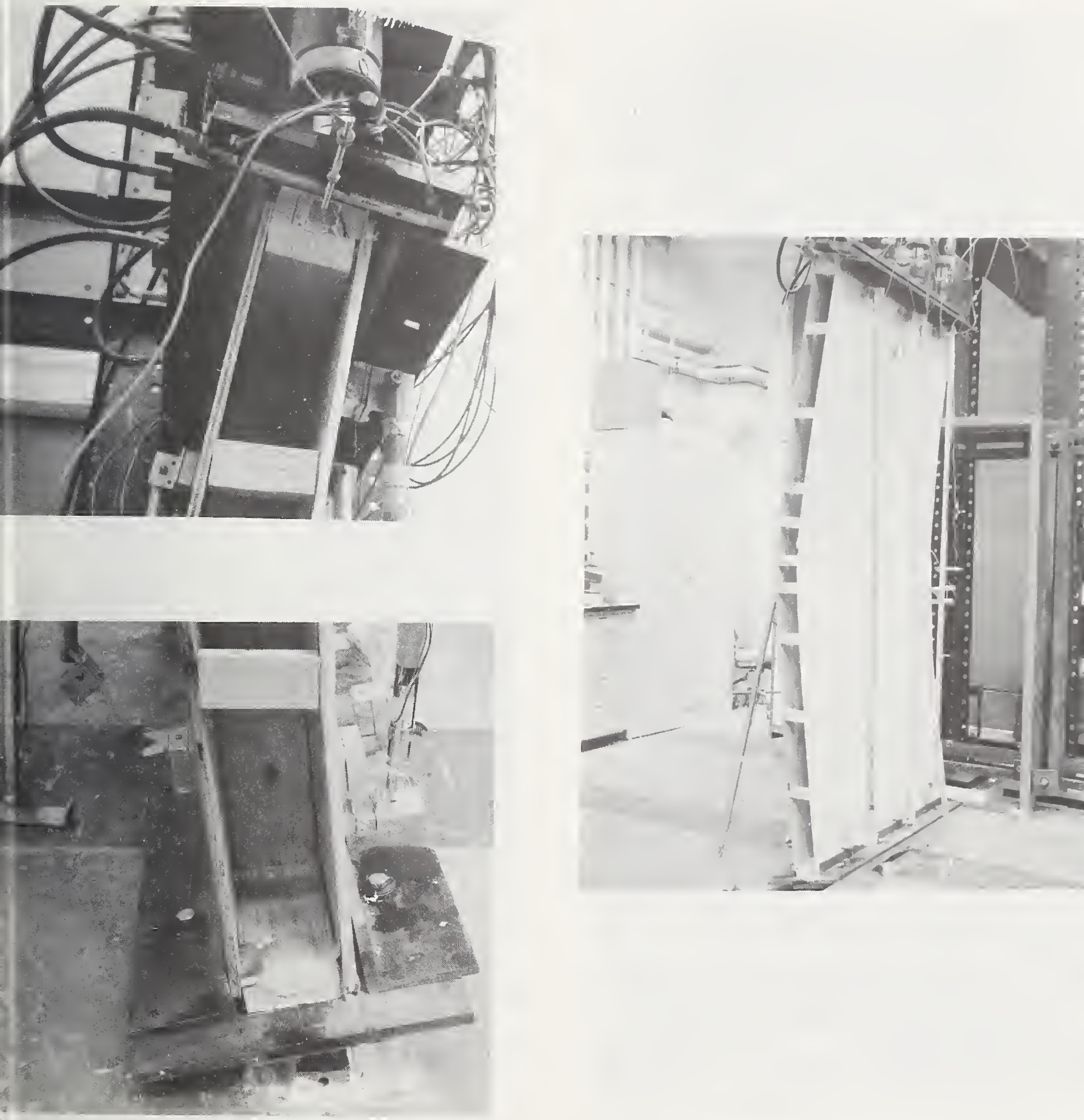


Figure 4.22 - Test run No. 23 (WFTH).

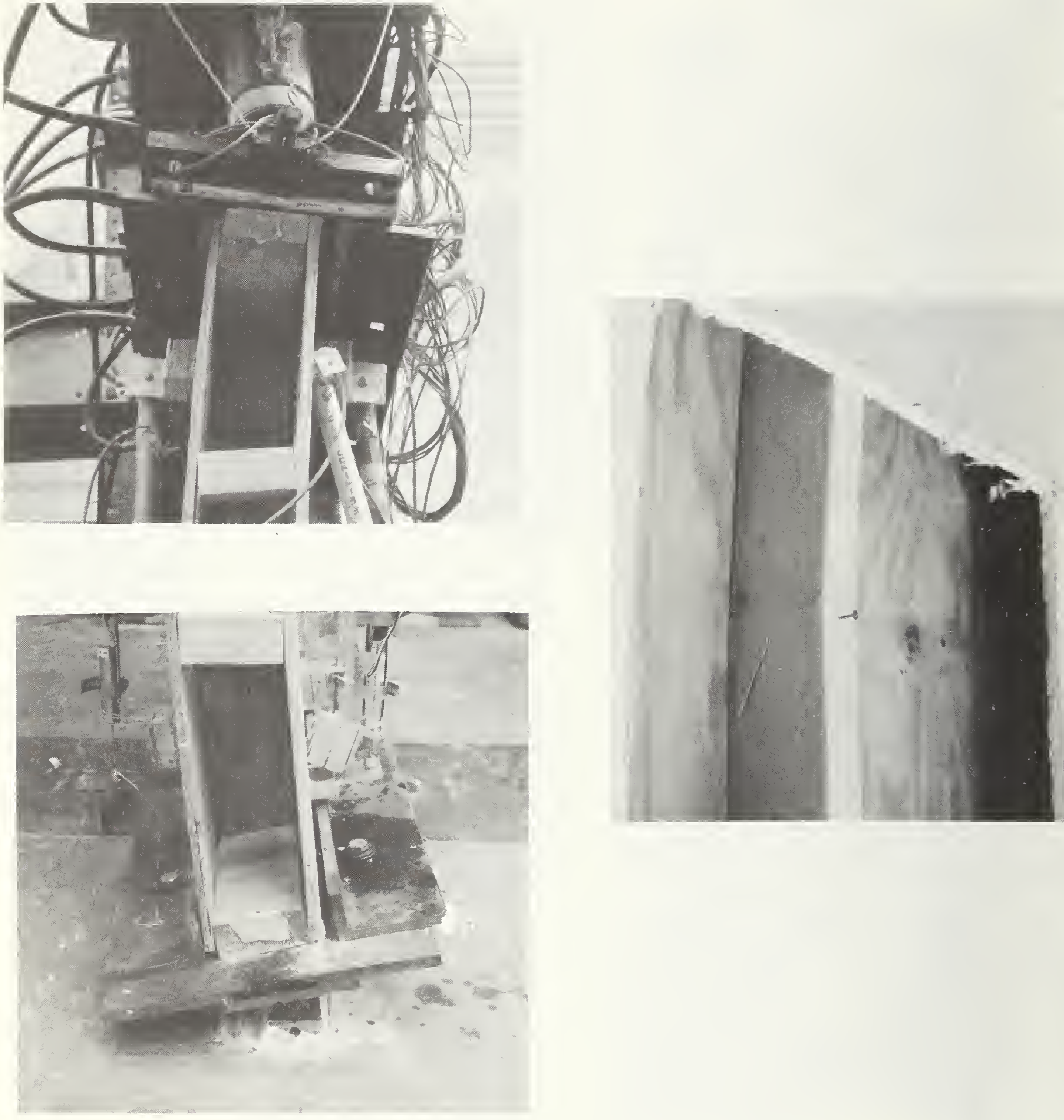


Figure 4.23 - Test run No. 24 (WFTH).

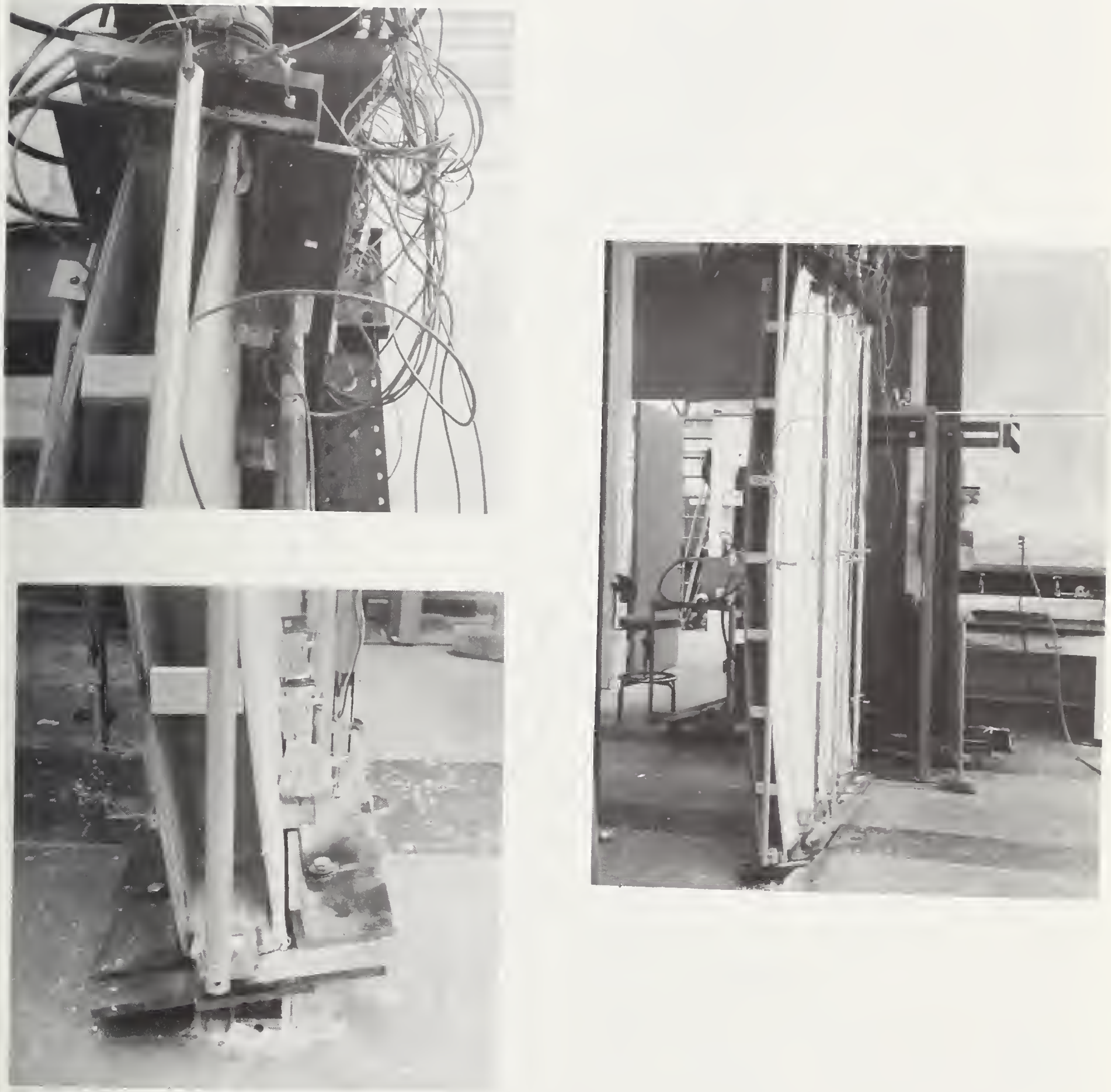


Figure 4.24 - Test run No. 25 (WFTH).

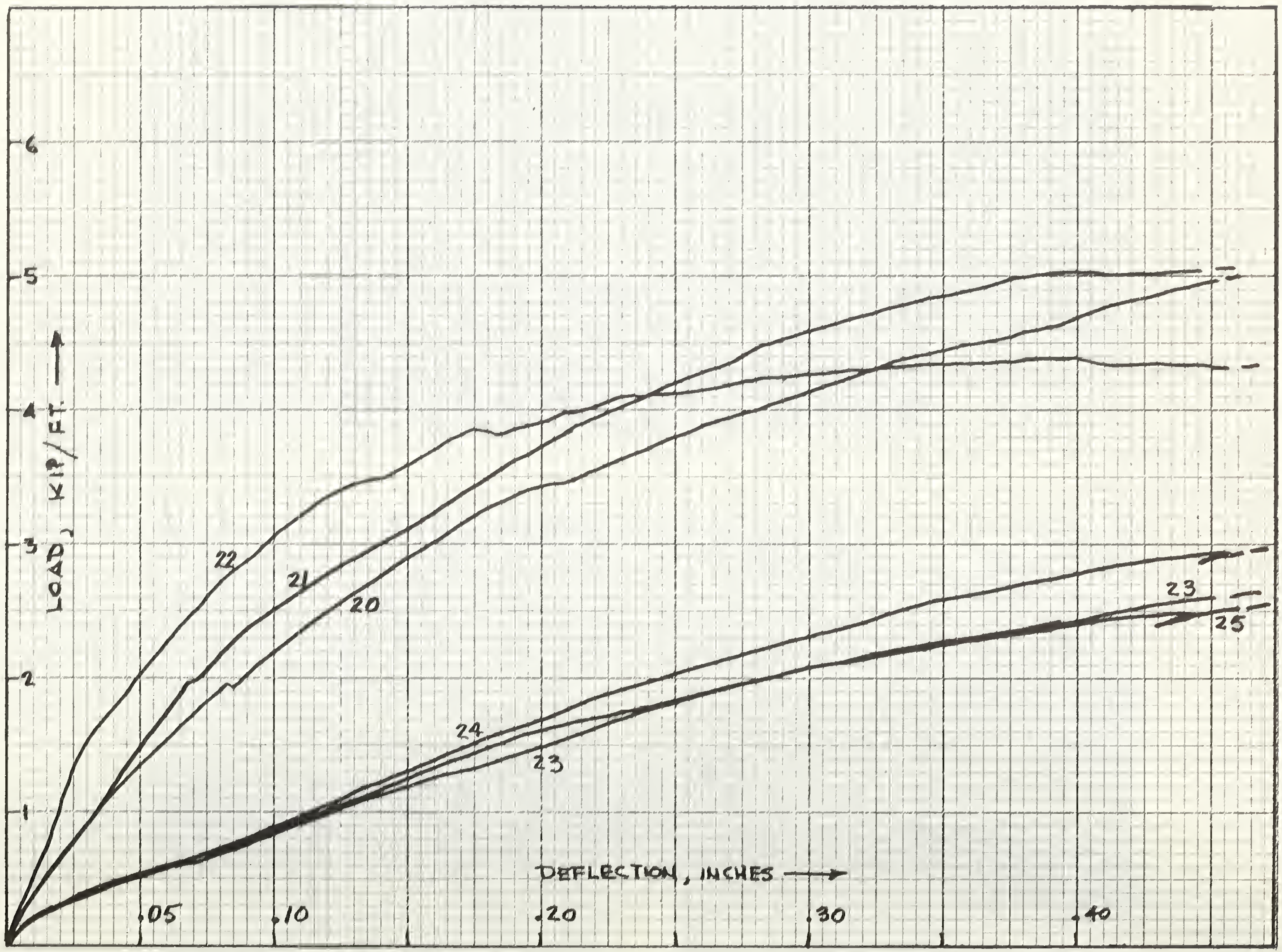


Figure 4.25 - Partial load-deflection curves for test run Nos. 20 to 25 (WFTH).

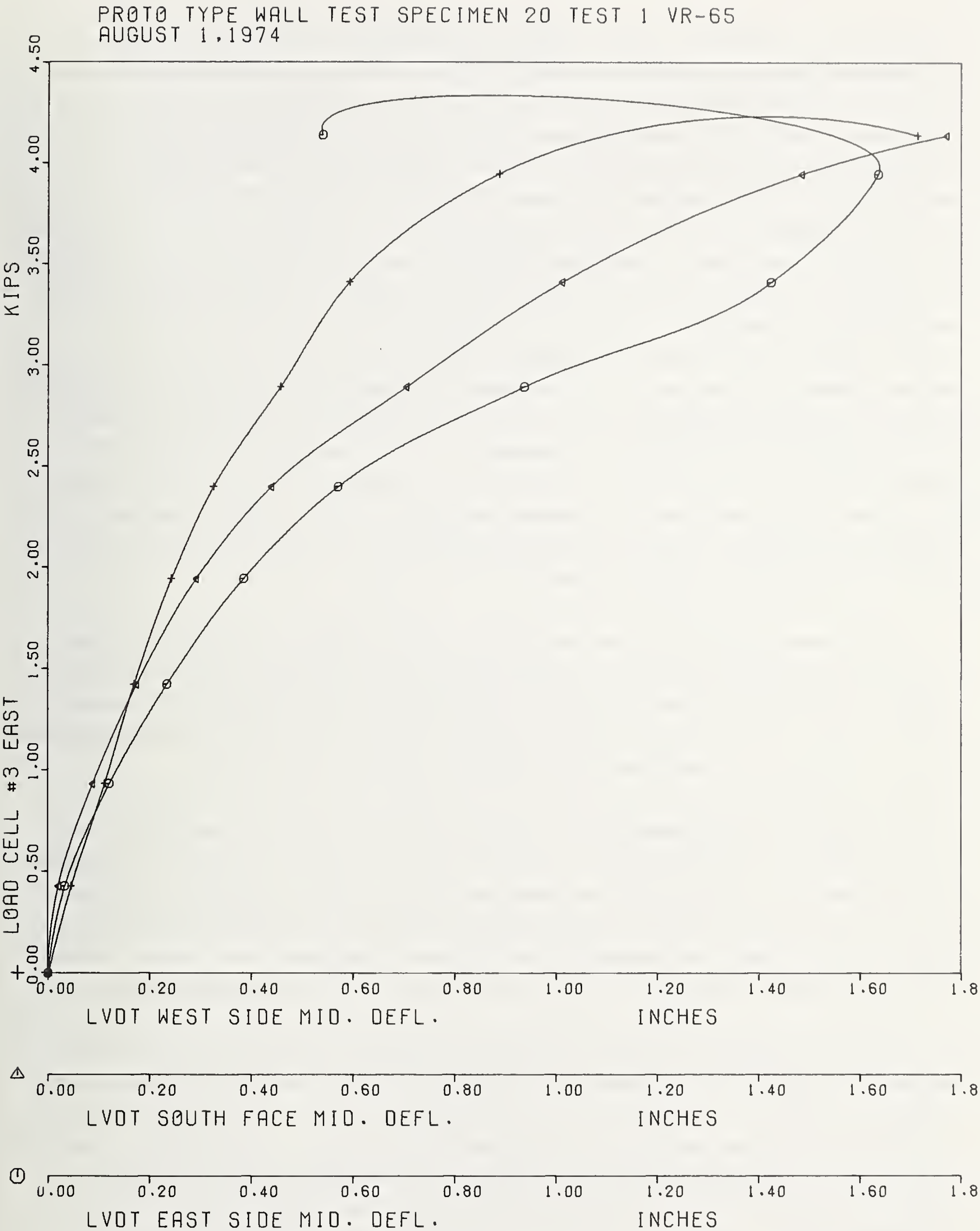


Figure 4.26 - Computer-plotted load-deflection curves, test run No. 24.

5. SUMMARY AND CONCLUSIONS

5.1 Summary - An experimental and analytical investigation of the primary factors involved in the test of wall panel prototypes under axial compression loading was reported. In order to establish the test parameters that warranted investigation, a survey of the literature on performance testing and a canvass of the members of an ASTM Task Group that is concerned with testing of vertical structures was conducted. Several test factors which were of common concern were identified and a research plan was established as a result of the survey. Twenty-five laboratory tests were conducted on five types of wall panels to fulfill the objective of developing an improved method of testing for compressive load resistance. The wall panel specimens were loaded incrementally to failure, while axial and transverse deformation responses were measured with electro-mechanical gages. As the tests were developmental in nature and not intended to evaluate the relative resistance capacity of the types of construction, only selected results were presented herein. The proposed standard test method was written according to the recommended format for ASTM Standard Methods and is presented as a part of this report. The proposed method describes the apparatus and procedure to be used for applying either a specific test load or an unknown maximum test load to a wall panel specimen which is a prototype of a designated wall construction.

The principal revisions of the existing standard method for compression testing are: (1) a provision for variable eccentricity of loading through the use of adjustable loading fixtures, (2) the incorporation of a procedure for selecting a load distribution assembly that applies an approximately uniform load to the top of the wall panel specimens and (3) a change in the end restraint condition from flat-end to pin-end.

A computer-aided analytical study of the variables affecting the degree of non-uniformity of loading was conducted. The conditions of the technical problem were found to have a theoretical analogy in the theory of beams supported on elastic foundations. The theoretical case of a beam of finite length, with free ends, being subjected to a number of concentrated loads relates to the laboratory testing of a wall panel specimen to which concentrated loads are applied through the use of a distribution assembly.

Since it is often desirable to optimize the number of concentrated loads and the stiffness of the distribution assembly, the objective was to derive a mathematical tool to aid in making these selections. Using Hetenyi's solutions of the basic differential equation for different loading cases, a mathematical procedure was derived and a computer program was written to perform the calculations. The procedure requires the input of several variables: (1) the stiffness of the proposed load distribution assembly, (2) the number and location of the concentrated loads and (3) the maximum variation, from ideal uniformity, that will be allowed. The output from the iterative calculations is the value of the maximum axial stiffness (i.e., foundation constant) that is permissible for testing with the proposed distribution assembly, without exceeding the stipulated maximum allowable variation from uniformity. The proposed test method in section 6 contains two useful tables that resulted from this study. The first table (6.A.1) contains experimental values of foundation constants for panels representative of some typical wall constructions. The second table (6.A.2) illustrates typical output from the computer program as generated from the input of five different load cases, two different wall panel widths and seven different stiffnesses of distribution assemblies. The Fortran V computer program that was used to generate this output is printed in its entirety in Appendix A.

The ASTM Standard E72 specifies an apparently arbitrarily chosen procedure for obtaining the eccentricity of loading. During this study the evolution of ASTM E72 was traced, but no insight was gained as to the rationale for designating the single eccentricity of one-third the wall thickness value. However, it was inferred that the one-third fraction was conceived from the postulate of a kern radius that is explained in discussions on eccentrically loaded compression members. The question of what is the true eccentricity at the end of a wall panel is a complex one due to such factors as the relatively thin cross sections, the inherent inaccuracies of loading fixture positioning and the heterogeneous composition of most types of wall construction. On the basis of assumed composite behavior of these heterogeneous panels, it is possible to compute the theoretical core or kern radius for a given type of construction. As an analytical exercise, the core radius was computed for three commonly used types of wall construction and the values were compared to the eccentricity derived by the ASTM E72 procedure.

A problem associated with standard test methods for building components in general is how to utilize the test results in a design or evaluation decision process. In this context, it is important to determine, prior to testing, the type of test that is to be conducted. That is, the evaluator must establish what kind of prediction he desires to make, about the behavior of the actual component, on the basis of the test results. In view of the fact that tests for compressive load resistance are usually conducted on representative samples (i.e., prototypes) of the actual wall panel construction, it was concluded in this study that the evaluator would desire to make a highly confident prediction (i.e. with a probability near 1.0) about the performance of a large proportion of the actual walls constituting the population. A study of the statistical parameters commonly used to describe test results, revealed that the use of one-sided tolerance limits was appropriate for this purpose. The use of one-sided tolerance limits was recommended in the proposed standard test method and the step-by-step procedure is presented in section 6.9.3.

5.2 Conclusions - Many of the conclusions based on the observations and results of this investigation were incorporated into the proposed standard test method which follows in section 6. The more notable findings acquired from both the experimental program and the analytical exercises are presented as follows:

1. Wall panels of multi-component (i.e., with facings, stiffener and perimeter components) construction exhibit localized, and generally, heterogeneous response when they are subjected to eccentric compressive loads. Consequently, the number of displacement gages cannot be reduced by replacing two or more gages with one gage located at the centroid of the several gages. In choosing instrument locations, a judgement should be made as to the influence of expected localized behavior on the measurements. Observations of such localized behavior should be reported with the test data.

If a wall panel specimen is expected to flex as a slender column, deflection gages should be mounted on the edges of a wall specimen to approach alignment with the neutral-surface (N.S.) location. Nevertheless, even if two gages are so mounted along opposite wall edges, and their data is subsequently "averaged" to approximate overall behavior, the individual records should also be reported. Furthermore, it is to be noted that if it is physically necessary to mount the deflectometers over a gage length which is less that the full height of the specimen, the deflection measurements must be given additional consideration. If the specimen gives evidence of beam-column action throughout its length, an appropriate classical beam-column deflection formula should be used to extrapolate the deflection measured over the shorter gage length to a value representative of the full specimen length or to establish that the error incurred by the length difference involved can be tolerated. If, on the other hand, the deflection is influenced by local deformations, so that the shape of the specimen cannot be approximated mathematically, the deflectometer gage length can only be made to approach the full length as nearly as possible, and the deflection measurements reported with accompanying qualifying comments.

When displacement gages must, of necessity, be mounted in pairs on opposite faces of a wall specimen for the purpose of averaging the data (e.g., measurements of wall height shortening or of wall deflection), the distances of both instruments' points of suspension from the neutral surface must be equal. This requirement is necessary to obviate tedious arithmetic corrections which must be made when opposite instruments are mounted at unequal distances from the N.S.; when the opposing gages are mounted at equal distances from the N.S. and their measurements are averaged, there is an automatic compensation of the errors which are introduced into each instrument by rotation of the facing-based mountings as the wall undergoes flexural response.

2. As demonstrated in figure 4.25, participation of wall panel facings in resisting compressive loads can appreciably affect the panel's stiffness characteristics as well as its strength (see table 1). Therefore it is important to establish the extent to which the various components of such types of wall construction are intended to contribute to the structural performance of the wall in practice, so that laboratory testing can be conducted accordingly. For example, a multi-component wall construction may have facings

which are not intended to be subjected to direct vertical load bearing at their ends. Then, the test panel should be prepared in such a way to ensure that the ends do not bear against the loading fixtures. Also, the wall cross section to be used in calculating the centroid must be determined on the basis of the expected contribution of the individual components to the flexural resistance of the wall panel.

3. While the standard test method for evaluating wall panel resistance to compressive loading should specify eccentric load application, designating a single, arbitrary eccentricity to be applicable to all types of wall construction is not recommended. Rather, the loading fixtures should be adjustable to facilitate a reasonable range of eccentricities. The testing agency can make the final decision on test eccentricity depending on the framing details of the actual building.

4. As a practical alternative to a complex and expensive experimental procedure to locate the centroid of a given wall cross section, it seems reasonable to assume composite resistance contributed by the full cross section and then locate the centroid by the technique of transformed areas. Once the centroid of the cross section is located, it can serve as a reference for establishing the test eccentricity of loading. Either, the kern radius can be calculated and used as the designated eccentricty or some other value based on the framing details of the actual building can be specified.

5. The range of deviation percentage obtained from the three examples presented in Appendix B extended from 2% (Example 1) to 38% (Example II). Such a broad range suggests that the use of the fixed value, $t_T/3$, as measured from the inside face, in place of the analytical procedure of establishing the eccentricity by computing the kern radius needs further study. The agreement between the two approaches is dependent upon the stiffness of the facing materials relative to that of the core material. Moreoever, because the interior facing material for this type of construction is usually gypsum wallboard, the agreement is strongly dependent upon the stiffness of the exterior facing material relative to that of the core material.

6. Numerous observations regarding instrument usage, which were mentioned in section 4 can be summarized as follows: Measuring instruments must be mounted and used with cognizance of surrounding conditions which might be adverse to the functioning of the instruments. The detector of a displacement gage must be accurately aligned with the direction of the component of the specimen displacement being sought. Also, provision must be made in the mounting procedure to neutralize the effect on the gage of unwanted components of specimen displacement. The unwanted components often prevent proper functioning of the gage. In many instances, these difficulties can be reduced by suitably mounting the gages on the specimen rather than on surrounding reference bases such as the test reaction frame or the test floor.

7. Since the end conditions of load bearing walls vary widely in practice and cannot usually be simulated realistically, it is recommended that wall compresssion tests be conducted with eccentric loading applied through pin-end conditions at both ends. This is recommended over a pin-flat combination from the standpoint of being conservative, and of ease of modelling for analytical evaluation. As an illustration, for the conditions of a "partially restrained" flat bottom and pin-end top, the location of the point of maximum transverse deflection is, in general, not well established, while for the pin-pin end conditions the maximum transverse deflection occurs at mid-height.

8. Because the ends of a wall specimen supported by two pinned ends may undergo substantial rotation during testing, it is necessary to specify fixtures that can provide adequate containment of the wall ends. However, the fixtures must be employed with caution in order to avoid damaging the specimen through excessive restraint.

9. Shadow lighting and quadrilling of the faces of a specimen can be used to facilitate early detection of certain modes of failure (e.g., the localized buckling of the thin skins used in some sandwich panel construction).

10. Safety restraints should be used around specimens whose expected brittle behavior at failure would be hazardous to personnel and test equipment.

11. It is not possible to recommend a specific rate of loading applicable to a variety of wall construction types on the basis of the laboratory or analytical investigations reported here. The differences in response characteristics exhibited by wall panels possessing different material properties would render such an undertaking impractical. A basic requirement when selecting a nominal rate of loading is that the rate should not be so fast as to cause the corresponding deformation to significantly lag the application of the load.

6. PROPOSED STANDARD METHOD OF TEST FOR COMPRESSIVE STRENGTH OF WALL CONSTRUCTIONS, ASTM DESIGNATION E-___ [2]

With the continuing development of building technology, stimulated in part by new methods and materials of prefabrication and by emerging performance criteria, there is an attendant need for accurate technical data on the strength and rigidity of present-day wall constructions. It is the purpose of this test method to provide an improved systematic procedure for obtaining engineering data on the strength and rigidity of wall constructions which are of value to designers, producers, consumers, and building officials interested in the performance of bearing-wall constructions.

6.1 Scope

6.1.1 This method of test describes a procedure for determining the performance of load-bearing wall construction segments under the action of vertical, eccentric, compressive, static loading. The method describes the apparatus and procedure to be used for applying either a specific test load (proof or acceptance test) or an unknown maximum value test load (failure test) to a prototype specimen. Failure is defined as the inability to support a specified test load or to fulfill a specified performance requirement.

6.2 Summary of Method

6.2.1 A wall segment oriented in a vertical position is loaded by a row of hydraulic rams acting vertically in the plane of the wall segment. The line of rams is eccentric to the centroidal axis of the wall's cross section so that loading tends to cause out-of-plane bending as well as other accompanying displacements. Load and displacements are monitored autographically to detect critical occurrences in performance.

6.3 Significance

6.3.1 The structural testing of complete buildings is expensive, time consuming and sometimes indeed impossible. Such tests of integrated elements in a whole structure have the further disadvantage of being terminated by the failure of the weakest element. Usually, the only practical approach involves testing a small sample of wall segments, using a test method which approximates service conditions.

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[2] This proposed standard test method has been submitted to the American Society for Testing and Materials (ASTM) for consideration as an ASTM standard. As such it has been prepared as a completely self-contained document. It is to be considered as a review draft in that it will no doubt undergo several revisions before being adopted as a standard.

6.3.2 Current building codes do not specify allowable deflections for walls under vertical load. Nevertheless, the need often arises for comparing the strength and stiffness of a wall system of unknown performance with that of a wall of established performance. This test method serves to determine the load carrying characteristics of a wall segment subjected to vertical loading which is eccentric within the dimension of the wall thickness. Since the line of loads is applied at each end through a fulcrum, actual end bearing conditions are not simulated. No attempt is made to achieve such simulation because actual end conditions are a function of type of construction and must be considered as a variable. The test report must, therefore, address the differences between end conditions used in the test and those of the wall in the actual structure. The test results will generally be on the conservative side in the evaluation of the relative strength or stiffness of wall constructions. In this sense the method is a rating test. Also, these procedures can be employed toward the objective of establishing either a load factor-of-safety, given a certain design load or an allowable load, given a safety factor.

6.4 Apparatus

6.4.1 The apparatus shall be assembled as shown in fig. 6.1. and shall conform to the detailed requirements for component parts prescribed in 6.4.1.1 through 6.4.1.7.

6.4.1.1 Testing shall be performed within a test frame which has a stiffness in the direction of loading that is at least 100 times greater than that expected for the specimen.

6.4.1.2 A loading system employing hydraulic rams which are driven by a motorized pump shall be used to apply a distributed load to the top of the specimen along its width. The pump shall be equipped with regulators capable of controlling load application and of maintaining the load. The least number of rams (points of load application) shall be determined with the aid of tables 6.A.1, 6.A.2 and the accompanying text of Appendix 6.A. A hydraulic testing machine of adequate size and load capacity may be used for load application provided due consideration is given to the uniform distribution of loading.

6.4.1.3 An electrical resistance type load cell (dynamometer) shall be mounted in tandem with each ram to measure the applied load. See Paragraph 6.5.1 (under Safety Precautions).

6.4.1.4 A 135°-V-grooved steel plate shall be used as a receptacle for a knife-edge fulcrum and as a bearing plate for the top of the wall specimen. Machining of this plate and all other metal parts shall be performed in accordance with good machine shop practice. The position of the bearing plate shall be adjustable across the thickness of

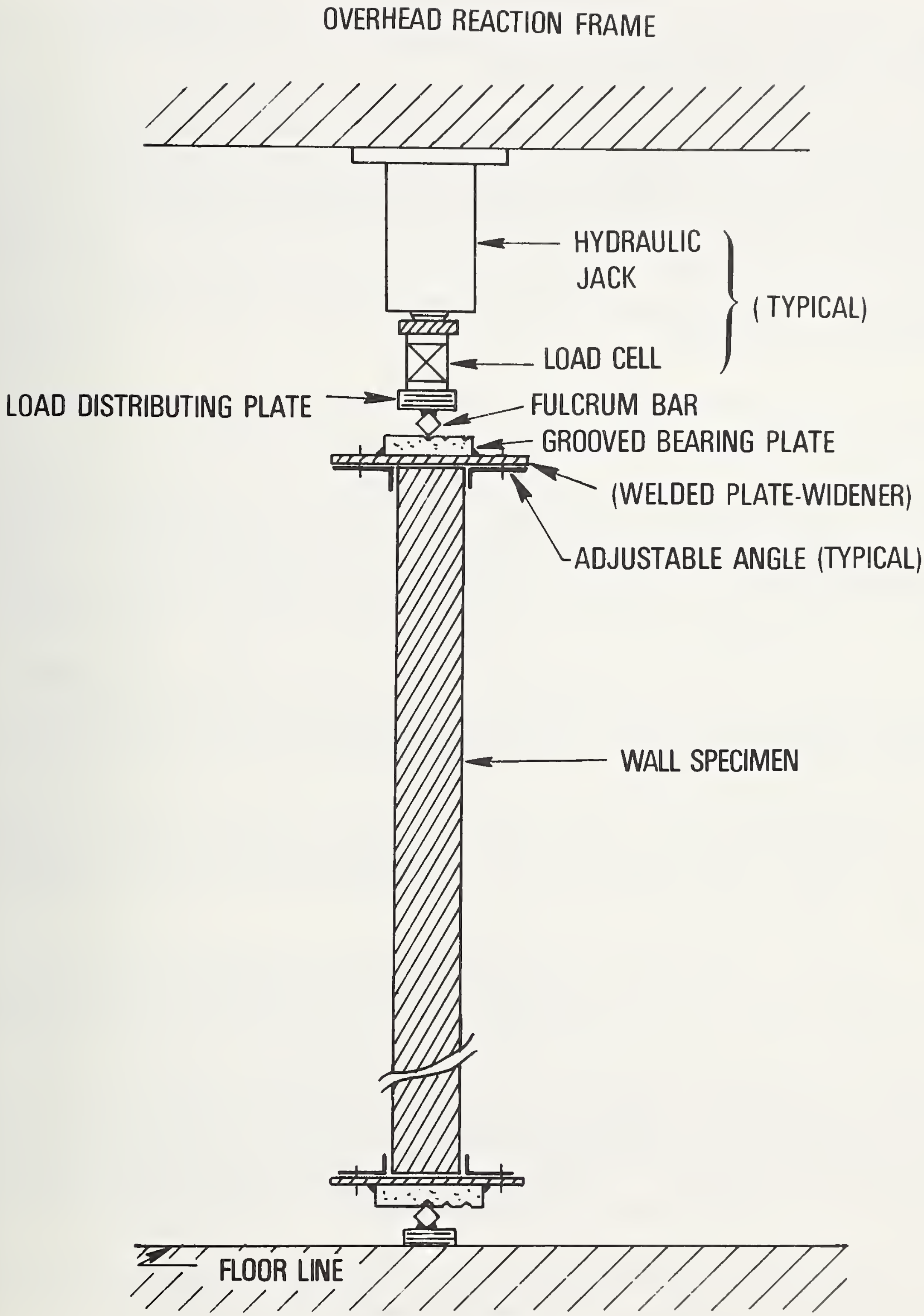

Figure 6.1 - Recommended pin-pin loading setup.

the wall specimen to achieve the designated eccentricity of the loading fulcrum. Structural angles which can be adjusted into position and bolted to the bearing plate shall be used to contain the top of the specimen laterally.

6.4.1.5 A steel plate welded to a 90°- knife-edge fulcrum shall be used to distribute the load from the hydraulic ram system uniformly along the width of the specimen. Welding of the knife-edge fulcrum to the plate shall be performed according to the requirements of American Welding Society standards and American Institute of Steel Construction specifications. Continuity of seating of the fulcrum in the V-groove shall be obtained by establishing that the surfaces are in contact at the 1/10 (of total width) points as a minimum.

6.4.1.6 A corresponding assembly of steel plates, knife-edge fulcrum and angles shall be used in an inverted arrangement at the bottom of the wall specimen.

6.4.1.7 A deflectometer shall be mounted on both vertical edges of the wall specimen to measure mid-height deflection. These deflectometers should be electromechanical so that load and deflection can be monitored continuously and autographically by an X-Y plotter. The deflectometers shall be mounted over the longest span possible, approximating full height, on the specimen itself (i.e. not mounted on the floor or test frame) in order to measure net deflection and eliminate effects of test set up movement. Alternatively, the deflectometers may be dial micrometers. It should be noted, however, that this alternate instrumentation does not allow continuous monitoring of the deflection response.

6.5 Safety Precautions

6.5.1 All components of the loading system, such as rams, load cells, plates, etc. must be interfastened and/or suspended from the reaction frame so that they can be supported without a specimen present in the test position.

6.5.2 At the start of the test, the vertical legs of the structural angles used for containment of the ends of the specimen must be in full contact with the surface which is designated as the exterior face (face which is furthest from the eccentric load) of the wall. The angles on the interior face of the wall should be spaced at a suitable distance from the wall in order to avoid gouging of the interior surface of the wall by the vertical legs of the angles during the test. If rotation of the wall-end fixtures is enough to cause closure of the allowed space, the interior angles must be readjusted during the test to prevent contact and possible crushing of the interior wall surface.

6.5.3 Caution must be exercised against injury to personnel, and damage to equipment by unexpected release of potential strain energy accumulated during the test.

6.6 Test Specimens

6.6.1 Wall specimens shall be representative of the construction to be used in service. The height of the specimen shall conform to the height of the wall in actual use. The width of the specimen shall be a whole number multiplied by the spacing of the principal vertical load-bearing members and shall extend from mid-space to mid-space, except for prefabricated panels, for which the width shall be as manufactured. To the extent practicable, in accordance with these requirements, specimens shall be nominally 4 ft (1.2 m) wide.

6.6.2 Compression tests shall be made on a sample consisting of at least five similar specimens.

6.7 Conditioning

6.7.1 Specimens shall be in a general state of readiness to be placed in service to undergo design loads and before testing shall be exposed for 7 days to temperature and humidity conditions equal to those of the testing laboratory (e.g., 73 ± 3°F, 50 ± 5% R.H.). Conditioning requirements for specific materials which might be damaged by exposure to other environmental service conditions shall be in accordance with appropriate specifications.

6.8 Procedure

6.8.1 The specimen shall be tested in compression as a beam column having fulcrum ends at top and bottom (fig. 6.1). Compressive load shall be applied to the top, uniformly along a line parallel to the faces of the wall. The eccentricity of loading shall be equal to the theoretical kern radius of the cross section, unless the eccentricity dictated by the actual framing details is known and used. The reaction fulcrum at the bottom shall have the same eccentricity.

6.8.2 Load shall be gradually applied to the specimen in increments. The size of the increments shall be chosen so that no more than 1/10 of the specified test load is applied during each increment.

6.8.3 Load shall be applied continuously (i.e., continuously during each increment)in a manner that effects a nominally uniform rate of deformation. In the absence of loading rate guidelines, it is recommended that the rate of loading during each increment not exceed 1/10 the specified test load per minute.

6.8.4 The first set of load and deflection readings shall be recorded prior to the application of any test loading. These initial load readings shall serve as reference points for all subsequent measurements. Individual load cells and deflectometers may be calibrated for visual reading with a digital voltmeter or may be read and recorded automatically. One load cell and one deflectometer shall be used for autographic monitoring of the test by an X-Y plotter.

6.8.5 Load shall be increased to the end of the first increment and the load and deflection measurements shall be recorded in accordance with paragraph 6.8.6. The recording procedures specified in paragraph 6.8.6 shall apply for all subsequent observations.

6.8.6 Load and measurements shall be recorded upon reaching the end of each specified increment of loading. Having maintained the load at the specified magnitude for a 5-minute period, load and deformation measurements shall be recorded again. When the loading is decreased, the load and deformation measurements shall be recorded instantaneously upon reaching the initial load level. After a recovery period of 5 minutes, the set of readings shall again be recorded. The schematic load-time plot in figure 6.2 illustrates the loading and unloading cycle with the recording points denoted by dots.

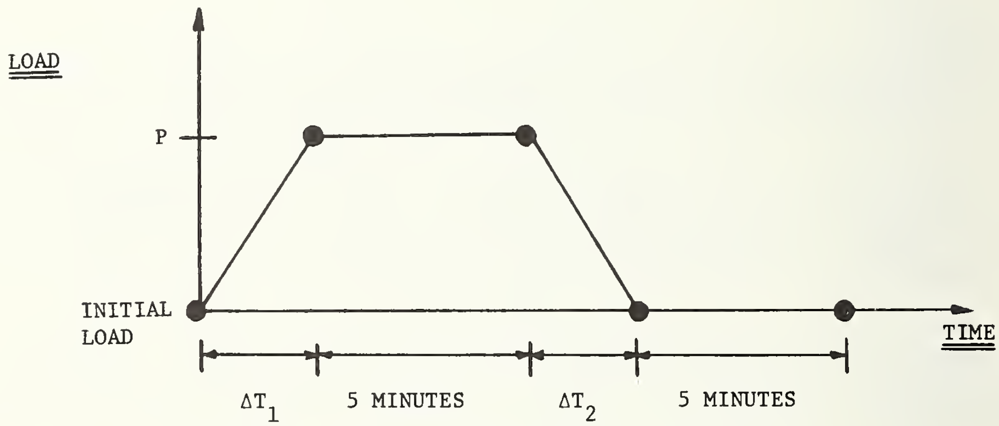

Figure 6.2 - Load-time schematic diagram.

6.8.7 Load shall be increased in specified increments until the test load is reached.

6.9 Calculation of Results

6.9.1 A load value at any test stage shall be calculated by summing the load cell net values and dividing by the width of the specimen. (lb/ft, N/m)

6.9.2 The lateral deflection at any stage shall be calculated as the average of the two deflectometer net readings. (in, m)

6.9.3 Test results shall be calculated as follows:

a. Obtain results, for a given parameter such as maximum load, maximum deflection, deflection at allowable load, etc., from five, or more, specimens: $x_1, x_2, x_3, x_4, x_5 \ldots x_n$

b. Calculate the sample average from the data obtained:

$$\bar{x} = \frac{1}{n} \sum_{i=1}^{n} x_i$$

c. Calculate the sample estimate of the standard deviation:

$$s = \sqrt{\frac{\sum_{i=1}^{n} (\bar{x} - x_i)^2}{n - 1}}$$

d. Establish tolerance limits to satisfy the degree of confidence that is desired. General formulation: The probability is γ (e.g., 0.95) that at least a proportion P (e.g., 0.90) of a population will be greater than $\bar{x}$ - ks (or less than $\bar{x}$ + ks) where $\bar{x}$ and s are estimates of the mean and standard deviation calculated from a sample of size n (e.g., 5). The value of k (e.g., 3.407) is obtained from a table of factors for one-sided tolerance limits for normal distribution.

6.9.4 A sample set of calculations using the procedure outlined in paragraph 6.9.3 is presented as follows:

a. The transverse deflection (x) at a given allowable load was measured for five (n) wall specimens.

b. The sample average calculated from the data is $\bar{x}$ = 0.1260 in.

c. The calculated value of the sample standard deviation is s = 0.00359 in.

d. Use a table containing factors for one-sided tolerance limits for normal distribution to calculate the value of deflection (x_u) below which we may predict with 95% confidence that 90% of the population will lie. Therefore let P = 0.90 and γ = 0.95.

For n = 5, γ = 0.95 and P = 0.90, k = 3.407

$x_u = \bar{x} + ks$

$x_u = 0.1260 + 3.407\ (0.00359)$

$x_u = 0.1260 + 0.0122 = 0.1382 \text{ in} \simeq 0.138 \text{ in}$

6.10 Report

6.10.1 The report shall include the following.

6.10.1.1 Dates of the tests and of the report.

6.10.1.2 Statement that the tests were conducted in accordance with the prescribed methods. Deviations from these methods shall be described.

6.10.1.3 Identification of the sample specimens, including manufacturer, source, physical description, detailed engineering drawings, photographs and other pertinent information.

6.10.1.4 Detailed information of test set-up including engineering drawings and photographs to make possible the duplication of test set-up by others.

6.10.1.5 Description (in addition to photographs) and interpretation of mode of failure.

6.10.1.6 Force - displacement graphs or other (if applicable) graphic, test progress records (such as stress-strain curves).

6.10.1.7 Tables of test results (for various parameters such as maximum load, maximum deflection at allowable load, etc.) including sample average, sample estimate of standard deviation and tolerance limits calculated for stated confidence level (cf. 6.9.3, d).

6.A Appendix - Designing the Test Load Distributing System

6.A.1 Uniform test load distribution - An approximation of a uniformly distributed load, parallel to the plane of the wall test specimen can be achieved by using several hydraulic jacks to apply concentrated loads to a load distributing beam which, in turn, transfers the total load to the top of the wall. The greater the number of jacks and the stiffer the distributing beam, the more uniform will be the wall load. Restrictions on test site space and economy are practical considerations which will limit the selection of the stiffness (EI) of the loading beam and the number of jacks. After a decision has been made regarding the maximum variation from a uniform load condition that can be tolerated, a computer program can be used to investigate combinations of the number of loads, their placement and the distributing beam stiffness (EI) that will provide the tolerable load distribution on a test wall of known or estimated stiffness (foundation constant, K).

6.A.2 Computerized solution to the problem - The computer program is designed to provide solutions to the problem, considered as an application of the theory of beams supported on elastic foundations (in this case, a load distributing beam bearing along the top of the test wall). The basic theory and equations are found in "Beams on Elastic Foundation" by M. Hetenyi, University of Michigan Press, 1946. The beam of length, L, has a flexural stiffness, EI and is supported on an elastic foundation (wall) which exerts a distributed reaction proportional to the vertical displacement, y, of the foundation. The beam is loaded by n equal, vertical loads spaced at a distance, D, from each other and beginning at a distance, αD, from each end. Figure 6.A.1 shows such an arrangement. The object of the computer program is to obtain a maximum foundation constant, K, (expressed in lb per in of width, per inch of vertical displacement) so that the maximum absolute value of the variation of the distributed foundation reaction (wall load) from the average value, divided by the average value, is equal to a prescribed upper limit, A1. However, because of computational difficulties it is not feasible to solve the problem exactly as stated, but rather to compute a foundation constant which makes the maximum load variation ratio greater than a lower limit, A2, but less than an upper limit, A1. Thus, as an example, the upper limit of the maximum load variation ratio, A1, may be made equal to 0.10 and the lower limit of the maximum load variation ratio, A2, may be made equal to 0.05 or some other value smaller than, but as close to A1 as may be desired.

6.A.3 Computer output - Each trial solution gives the stiffness of a wall which can be tested by the combination of beam and load arrangement being considered (i.e., entered as program input), without exceeding the chosen maximum load variation. The distance of the first and last loads from the ends of the beam, αD, may be varied to maximize the foundation constant which can be accommodated. By estimating the stiffness of the wall which is to be tested, the user can select the most desirable test arrangement for which the computer gives a foundation constant larger than that which was estimated for the test wall. To assist the user in checking initial estimates of specimen wall stiffness (i.e., foundation constants), table 6.A.1 provides such values derived from the testing experience of the sources indicated.

Table 6.A.1- Experimental Values of Foundation Constants

for Panels Representative of Some Typical Wall Constructions[1/]

PANEL DESCRIPTION				REFERENCE	FOUNDATION CONSTANT[2/]
INTERIOR FACING	CORE	EXTERIOR SHEATHING	CLADDING OR VENEER	(See List of Refs. Below)	(lb/in/in)
Textured 0.02" thick aluminum	3" paper honeycomb	Textured 0.02" thick aluminum	-	No. (1)	3.16×10^3
1/2" gypsum wallboard	2x4 - wood studs @ 16" oc	1/2" insulating fiberboard	1" x 4" air dried select wood siding	No. (1)	4.72×10^3
1/2" gypsum wallboard	2x4 - fir studs @ 16" oc	1/2" insulating fiberboard	Common face brick	No. (1)	5.21×10^3
Glass fiber reinforced polyester laminate	3 5/8" thick corrugated core-glass fiber reinforced polyester laminate	Glass fiber reinforced polyester laminate	-	No. (2)	$5.42\text{-}6.67 \times 10^3$
Fiber reinforced polyester; woven roving	3" paper honeycomb	Fiber reinforced polyester; woven roving	1/2" gypsum wallboard interior & exterior	No. (3)	7.34×10^3
1/2" gypsum wallboard	18 ga. galv. steel "Z" studs at 24" oc	1/2" nail base insulating fiberboard	-	No. (5) pp. 5 and 8	$7.81\text{-}9.06 \times 10^3$
1/2" gypsum wallboard	4" x 8" x 16" concrete block	-	4" common face brick	No. (1)	8.68×10^3
1/2" foilbacked gypsum wallboard	18 ga. galv. steel "Z" studs at 24" oc	3/8" cedar plywood--exterior grade	-	No. (1)	11.58×10^3
1/4" philippine mahogany plywood	1 1/2" expanded polystyrene w/2x4 stringers @ 48"oc	5/16" douglas fir plywood	aluminum cladding	No. (6) pp. 52-65	$14.29\text{-}17.94 \times 10^3$
1/4" fir plywood	2 1/2" polystyrene w/ 2 1/2" x 2 1/2" fir stringers at 48" oc	1/4"fir plywood	-	No. (1)	14.88×10^3
-	8" med. strength brick-common Amer. bond-joints not completely filled	-	-	No. (4) Wall AB, Fig. 11	54.10×10^3
-	8" med. strength brick--common Amer. bond--joints completely filled	-	-	No. (4) Wall AC, Fig. 17	61.60×10^3
-	8" stone concrete block	-	-	No. (4) Wall AF, Fig. 42	66.6×10^3
-	8" clay tile-tiles laid on side	-	-	No. (4) Wall AE, Fig. 34	71.00×10^3

[1/] *The tabulated values of "foundation constants" are presented with the intention of supplementing Table 6.A.2. It should be recognized that this table has been derived from single sets of experimental data obtained by testing 8-ft high prototypes of a variety of wall constructions with generally high variability of performance. The values presented are intended to indicate orders of magnitude of axial stiffness and should not be construed to indicate absolute measure.*

[2/] *The term "foundation constant", taken from the theory of beams on elastic foundations, is used herein as a measure of axial stiffness of wall systems. It is expressed in units of lbf/in (of wall)/in (of deflection). The foundation constants presented in this table were calculated by measuring the slope in the elastic range, of test curves that plotted axial load versus shortening.*

Table 6.A.1 - Experimental values of foundation constants.

Table 6.A.1 Continued - List of References Cited Above

1. "Performance Criteria for Exterior Wall Systems," National Bureau of Standards (U.S.) Report 9817, by Staff of Building Research Division, April 1968, Table 5-3.

2. "Structural Tests on Housing Components of Glass Fiber Reinforced Polyester Laminate," Reichard, T. W. et al, NTIS PB 221-183, National Bureau of Standards (U.S.), April 1973, 90 pp.

3. "Structural Tests for a Housing System Using Sandwich Panels with Gypsum Board Facings," Greene, W. E. et al, NTIS PB 214-336, National Bureau of Standards (U.S.), Nov. 1972, 75 pp.

4. "Structural Properties of Six Masonry Wall Constructions," Whittemore, H. L., Stang, A. H. and Parsons, D. E., National Bureau of Standards (U.S.), Building Materials and Structures Report (BMS) 5, November 1938, 31 pp.

5. "Report on Compression Tests of Prefabricated Wall Panels," University of Louisville (IIR), Report No. 40208, McIntosh, W. R. and Snowden, J. R., February 12, 1974.

6. "Structural and Materials Performance," Volume II HUD Austin Oaks Project, Fowler, D. W. and Houston, J. T., December 1971.

Table 6.A.2 shows values of input and output for various trial solutions to the problem of providing satisfactory test loading for a variety of circumstances. Input values are the same for both the left and right halves of the table except for the edge distance-load spacing ratio. The number of load cases considered is 5, namely 2, 3, 4, 5 and 6 equidistant concentrated loads. The number of wall specimen widths considered loaded by each number of loads is 2, namely 48 in and 96 in. The number of attempted beam stiffnesses is 7; these values of EI represent a variety of loading beams such as steel rolled sections and timbers. The tabulated values of EI ($lb\text{-}in^2$) are expressed as a decimal to be multiplied by 10 raised to the indicated power which follows the plus sign. The edge distance-load spacing ratio, α, (which indicates proximity of the outer loads to the wall edges) is 0.5 for cases in the left half of the table and 1.0 for the right half. The stated values of upper maximum load variation (0.10) and lower maximum load variation (0.05) indicate that, for the solutions obtained, the acceptable maximum variation from uniform load distribution lies between 5% and 10%. For each combination of (1) edge distance-load spacing ratio, (2) number of load points, (3) wall width and (4) distributing beam flexural stiffness (EI), the information and results are presented in a single line of output under the column headings. The solutions (under column heading FOUNDK expressed as a decimal to be multiplied by 10 raised to the indicated exponent) give the range of specimen axial stiffnesses which can be tested by the corresponding loading arrangements within the acceptable limitations imposed on variation from uniform load distribution. It is intended that the range of values presented in table 6.A.2 together with the illustrative foundation constants in table 6.A.1, assist the user in determining a suitable loading arrangement for applying a satisfactorily uniform load to the specimen to be tested.

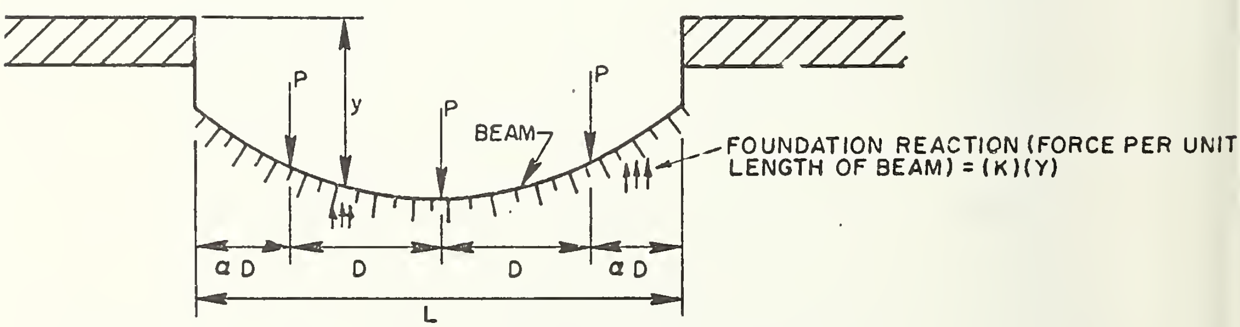

Figure 6.A.1 - Beam on elastic foundation.

NUMBER OF LOAD CASES IS 5
NUMBER OF WIDTHS IS 2
NUMBER OF BEAM STIFNESSES IS 7
EDGE DISTANCE-LOAD SPACING RATIO IS .50
UPPER MAX. LOAD VARIATION IS .10
LOWER MAX. LOAD VARIATION IS .05

LOADS	WIDTH	EI	FOUNDK
2	48.0	.1900+08	.3904+03
2	48.0	.7200+08	.1559+04
2	48.0	.1800+09	.3113+04
2	48.0	.1600+10	.3594+05
2	48.0	.2934+10	.6875+05
2	48.0	.3990+10	.6875+05
2	48.0	.6120+10	.1250+06
2	96.0	.1900+08	.2441+02
2	96.0	.7200+08	.9764+02
2	96.0	.1800+09	.1953+03
2	96.0	.1600+10	.2338+04
2	96.0	.2934+10	.3113+04
2	96.0	.3990+10	.4663+04
2	96.0	.6120+10	.6201+04
3	48.0	.1900+08	.1170+04
3	48.0	.7200+08	.4663+04
3	48.0	.1800+09	.9277+04
3	48.0	.1600+10	.1125+06
3	48.0	.2934+10	.2250+06
3	48.0	.3990+10	.2500+06
3	48.0	.6120+10	.4500+06
3	96.0	.1900+08	.7324+02
3	96.0	.7200+08	.2929+03
3	96.0	.1800+09	.5856+03
3	96.0	.1600+10	.4663+04
3	96.0	.2934+10	.9277+04
3	96.0	.3990+10	.1836+05
3	96.0	.6120+10	.1836+05
4	48.0	.1900+08	.3894+04
4	48.0	.7200+08	.1543+05
4	48.0	.1800+09	.3047+05
4	48.0	.1600+10	.2500+06
4	48.0	.2934+10	.4500+06
4	48.0	.3990+10	.5500+06
4	48.0	.6120+10	.1000+07
4	96.0	.1900+08	.2441+03
4	96.0	.7200+08	.9758+03
4	96.0	.1800+09	.1950+04
4	96.0	.1600+10	.1836+05
4	96.0	.2934+10	.3047+05
4	96.0	.3990+10	.3594+05
4	96.0	.6120+10	.6875+05
5	48.0	.1900+08	.7764+04
5	48.0	.7200+08	.3047+05
5	48.0	.1800+09	.6875+05
5	48.0	.1600+10	.9000+06
5	48.0	.2934+10	.1100+07
5	48.0	.3990+10	.1800+07
5	48.0	.6120+10	.2200+07
5	96.0	.1900+08	.4881+03
5	96.0	.7200+08	.1950+04
5	96.0	.1800+09	.4663+04
5	96.0	.1600+10	.3594+05
5	96.0	.2934+10	.6875+05
5	96.0	.3990+10	.1125+06
5	96.0	.6120+10	.1375+06
6	48.0	.1900+08	.1543+05
6	48.0	.7200+08	.5938+05
6	48.0	.1800+09	.1375+06
6	48.0	.1600+10	.1800+07
6	48.0	.2934+10	.3600+07
6	48.0	.3990+10	.3600+07
6	48.0	.6120+10	.7200+07
6	96.0	.1900+08	.9758+03
6	96.0	.7200+08	.3894+04
6	96.0	.1800+09	.9277+04
6	96.0	.1600+10	.1125+06
6	96.0	.2934+10	.2250+06
6	96.0	.3990+10	.2250+06
6	96.0	.6120+10	.4500+06

NUMBER OF LOAD CASES IS 5
NUMBER OF WIDTHS IS 2
NUMBER OF BEAM STIFNESSES IS 7
EDGE DISTANCE-LOAD SPACING RATIO IS 1.00
UPPER MAX. LOAD VARIATION IS .10
LOWER MAX. LOAD VARIATION IS .05

LOADS	WIDTH	EI	FOUNDK
2	48.0	.1900+08	.9764+02
2	48.0	.7200+08	.3904+03
2	48.0	.1800+09	.1170+04
2	48.0	.1600+10	.9277+04
2	48.0	.2934+10	.1836+05
2	48.0	.3990+10	.2422+05
2	48.0	.6120+10	.3594+05
2	96.0	.1900+08	.6103+01
2	96.0	.7200+08	.2441+02
2	96.0	.1800+09	.7324+02
2	96.0	.1600+10	.5856+03
2	96.0	.2934+10	.1170+04
2	96.0	.3990+10	.1559+04
2	96.0	.6120+10	.2338+04
3	48.0	.1900+08	.1465+03
3	48.0	.7200+08	.5856+03
3	48.0	.1800+09	.1559+04
3	48.0	.1600+10	.1230+05
3	48.0	.2934+10	.2422+05
3	48.0	.3990+10	.3594+05
3	48.0	.6120+10	.4688+05
3	96.0	.1900+08	.9155+01
3	96.0	.7200+08	.3662+02
3	96.0	.1800+09	.9764+02
3	96.0	.1600+10	.7805+03
3	96.0	.2934+10	.1559+04
3	96.0	.3990+10	.2338+04
3	96.0	.6120+10	.3113+04
4	48.0	.1900+08	.1953+03
4	48.0	.7200+08	.7805+03
4	48.0	.1800+09	.1559+04
4	48.0	.1600+10	.1836+05
4	48.0	.2934+10	.2422+05
4	48.0	.3990+10	.3594+05
4	48.0	.6120+10	.6875+05
4	96.0	.1900+08	.1221+02
4	96.0	.7200+08	.4883+02
4	96.0	.1800+09	.9764+02
4	96.0	.1600+10	.1170+04
4	96.0	.2934+10	.1559+04
4	96.0	.3990+10	.2338+04
4	96.0	.6120+10	.4663+04
5	48.0	.1900+08	.1953+03
5	48.0	.7200+08	.7805+03
5	48.0	.1800+09	.2338+04
5	48.0	.1600+10	.1836+05
5	48.0	.2934+10	.3594+05
5	48.0	.3990+10	.4688+05
5	48.0	.6120+10	.6875+05
5	96.0	.1900+08	.1221+02
5	96.0	.7200+08	.4883+02
5	96.0	.1800+09	.1465+03
5	96.0	.1600+10	.1170+04
5	96.0	.2934+10	.2338+04
5	96.0	.3990+10	.3113+04
5	96.0	.6120+10	.4663+04
6	48.0	.1900+08	.2929+03
6	48.0	.7200+08	.1170+04
6	48.0	.1800+09	.2338+04
6	48.0	.1600+10	.2422+05
6	48.0	.2934+10	.3594+05
6	48.0	.3990+10	.6875+05
6	48.0	.6120+10	.8750+05
6	96.0	.1900+08	.1831+02
6	96.0	.7200+08	.7324+02
6	96.0	.1800+09	.1465+03
6	96.0	.1600+10	.1559+04
6	96.0	.2934+10	.2338+04
6	96.0	.3990+10	.3113+04
6	96.0	.6120+10	.6201+04

Table 6.A.2 - Correlation of compression-test setup factors for uniform load distribution.

7. ACKNOWLEDGMENTS

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Mrs. R. Hocker, Miss J. Reinhold, Clerk-Typists, performed the typing of the several drafts of this report.

Messrs. L. Payton and M. Glover, Engineering Technicians, erected the test frame, installed test hardware and prepared the test specimen.

Mr. J. Owen, Electronics Technician, executed the instrumenting of the specimen and operated the data acquisition equipment during testing.

Mr. M. Lemay, Engineering Technician, performed most of the fabrication and machine work.

Messrs. R. Williams, Physicist and F. Rankin, Supervisory Engineering Technician, supervised the instrumentation-data acquisition activity and the laboratory-fabrication, erection and specimen preparation activity, respectively.

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Appendix A - Computer Program for Load Distribution Analysis

A computer program designed to provide solutions to a problem based upon the theory of beams on elastic foundations is described herein. The problem definition follows.

A beam of length L having a flexural stiffness EI is supported on an elastic foundation which exerts a distributed restoring force proportional to the vertical displacement, y, of the foundation. The beam is loaded by n equal, vertical loads spaced at a distance, D, from each other and beginning a distance, αD, from each end. Figure A.1 shows such a beam. The object of the computer program is to obtain a maximum foundation constant K, which may be expressed in lbs/in/in so that the maximum absolute value of the variation of the distributed foundation force from the average value, when divided by the average value, is equal to a prescribed limit, A1. Because of computational difficulties it is not feasible to solve the problem exactly as stated, but instead a foundation constant is computed which makes the ratio greater than A2 but less than A1. Thus, for example, A1 may be made equal to 0.10 and A2 may equal .08 or some other value smaller than, but as close to A1 as may be desired.

The program has at least two known practical applications. Firstly, it may be used in the design of multiple column foundation footings where the footing is prismatic and equal loads are applied by columns spaced at regular intervals. If the designer wishes to minimize variations in the foundation pressure he can use the program to determine the flexural stiffness required of the foundation. Another use for the program would be in the design of a compression test for a wall or wall panel where it is desired to apply a uniformly distributed load either in the axial plane or in a plane parallel to it. An approximation of a uniform load is usually achieved by using several jacks to apply concentrated loads to a load distributing beam which in turn transfers the loads to the wall. The greater the number of jacks and the stiffer the distributing beam, the more uniform will be the wall load. Test site space restrictions and economy are practical

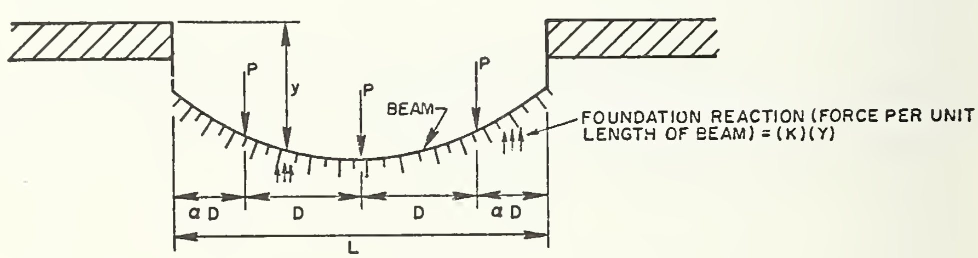


Figure A.1 - Beam on elastic foundation.

considerations which limit the EI of the beam and the number of jacks. Consequently, a decision is often made as to the maximum variation from the uniform load condition which is to be tolerated.

The program permits a user who plans a wall test to try several combinations of beam stiffness and number of jack loads once he has chosen a range for maximum variation of the distributed wall load from the average value. Each trial gives the stiffness of a wall which may be tested by the contemplated test arrangement without exceeding the maximum load variation specified by the user. The program also permits the ratio, α, (see figure A.1) to be varied to maximize the foundation constant (wall stiffness) calculated by the computer. The user must make an estimate of the stiffness of the actual test wall. However, once this is made he can select the cheapest or the most convenient of the contemplated test arrangements for which the computer output gives a larger foundation constant than his estimate of the axial stiffness of the test wall.

The basic theory and equations are found in the book, "Beams on Elastic Foundation," by M. Hetenyi [10]. The principal equations used are equation 42a on page 54 and the unnumbered equation, 4 lines from the bottom of page 53 in Hetenyi's book. The program contains a main routine and two subroutines, designated subroutine SUB and subroutine BUS. Table A.1 contains a listing of the more important variables as used in this appendix and their names as they appear in the computer program. The program which is written in FORTRAN language, is listed in table A.2. Each statement occupies one or more of the numbered lines of the listing. The line numbers will be used to reference statements and parts of the program.

The input is supplied to the main program in groups of four cards. The first card (see line 8) supplies the number of load-point cases, the number of wall-width cases, the number of beam-stiffness (EI) cases, the values α, A1 and A2. The second card (see line 16) supplies the number of load points corresponding to each load-point case. Thus, if there are 3 load-point cases, e.g., 4 placed in the first data column would correspond to 4 load points, 5 placed in the second column corresponds to 5 load points and 6 placed in the third column corresponds to 6 load points. The third card (see line 18) supplies the width value for each wall-width case. The fourth card (see line 20) supplies the EI value for each EI case. The four data cards may be repeated to obtain additional sets of results as, for example, when it is desired to change A1, A2 or α. A format statement follows each read statement so that the user can easily determine the proper method of supplying data on punched cards.

The output (see the top section of table 6.A.2) is preceded by six statements which give the number of load cases, width cases, EI cases and α, A1 and A2(see line 11 for the format). Following these statements there are twelve columns in a full printout. (Table 6.A.2 was edited for the proposed test method presented in section 6 to provide values of LOADS, WIDTH, EI and FOUNDK, which are of the most assistance to the user). These are headed as follows: LOADS, WIDTH, EI, FOUNDK, DEVA, DEVL, DEVC, ABMAX, YSUBA, YSUBL, YSUBC and YAVE. All the information and results for a complete case are printed

TABLE A.1 - A PARTIAL LIST OF VARIABLES

Variable Name Description	THIS REPORT	MAIN PROGRAM	SUBROUTINE SUB	SUBROUTINE BUS
NUMBER OF POINT LOADS	n	IPOINT(JJ), N	L1	L2
LENGTH OF WALL	L	WIDTH(KK)	WIDTHS	WIDTHT
FLEXURAL STIFFNESS OF LOAD DISTRIBUTING BEAM	EI	EI(I)	EIS	EIT
FOUNDATION CONSTANT	K	FOUNDK	FOUNKD	KFOUND
DISTANCE BETWEEN LOADS	D	D	DS	DT
RATIO OF EDGE DISTANCE TO FIRST LOAD TO DISTANCE BETWEEN LOADS	α	ALFA	ALAF	AFLA
MAGNITUDE OF A JACK LOAD	P	-	-	-
BEAM OR FOUNDATION DEFLECTION	y	-	-	-
BEAM DEFLECTION AT END DIVIDED BY A JACK LOAD	-	YFORA	YSUBA	ADEFL
BEAM DEFLECTION AT CENTER DIVIDED BY A JACK LOAD	-	YFORC	YSUBC	CDEFL
BEAM DEFLECTION AT MOST CENTRAL LOAD DIVIDED BY A JACK LOAD	-	YFORL	YSUBL	LDEFL
PRESSURE DEVIATIONS FROM AVERAGE DIVIDED BY THE AVERAGE PRESSURE - AT END, UNDER A CENTRAL LOAD AND AT CENTER	- - -	DEVA DEVL DEVC	DIFA DIFL DIFC	ADEV LDEV CDEV
AVERAGE FOUNDATION PRESSURE DIVIDED BY A JACK LOAD	-	AVEY	YAVE	AVER
MAXIMUM PERCENT DEVIATION OF THE PRESSURE FROM THE AVERAGE DIVIDED BY 100	-	ABMAX	ABMXA	MAXAB

Table A.1 - A partial list of computer program variables.

Table A.2 - Computer program for load distribution analysis.

```
@DATA,L  TEMPDATA.
DATA T7 RL70-5 04/09-16:22:21
C     *
C     *
C     *
      REAL LK
      COMMUN X,Y,N,FDUNDK,ALFA,ABMAX,DEVC,DEVL,DEVA,YFORA,YFDRL,YFORC,
     1AVEY,D
      DIMENSIDN IPOINT(10),WIDTH(10),EI(10)
    3 READ(5,10,END=2)J,K,L,ALFA,A1,A2
   10 FORMAT(3I2,3F4.2)
      WRITE(6,16)J,K,L,ALFA,A1,A2
   16 FORMAT(1H133HNUMBER DF LOAD CASES IS           I5,/1H 33HNUMBER DF
     1WIDTHS IS                  I5,/1H 33HNUMBER DF BEAM STIFNESSES IS
     2 I5,/1H 35HEDGE DISTANCE-LOAD SPACING RATIO ISF5.2,/1H 28HUPPER MA
     3X. LOAD VARIATION ISF10.2,/
     4                          1H 28HLOWER MAX. LOAD VARIATION ISF10.2)
      READ(5,20)(IPDINT(I),I=1,J)
   20 FDRMAT(10I2)
      READ(5,50)(WIDTH(I),I=1,K)
   50 FDRMAT(10F3.0)
      READ(5,70)(EI(I),I=1,L)
   70 FDRMAT(10E8.0)
      WRITE(6,15)
   15 FDRMAT(1H01OHLDADS      10HWIDTH      9H   EI      11HFOUNDK      10H
     1DEVA      10H DEVL      10H DEVC      10HABMAX      10HYSUBA      10HYS
     2UBL      10HYSUBC      5H YAVE/)
      DD11JJ=1,J
      N=IPDINT(JJ)
      DO12KK=1,K
      D=WIDTH(KK)/(FLOAT(N-1)+2.*ALFA)
      X=(ALFA+FLOAT(N/2-1))*D
      Y=X+D
      DO13LL=1,L
      IF(MOD(N,2))51,51,52
   51 FOUNDK=200000.
   71 CDNTINUE
      CALL SUB(EI(LL),WIDTH(KK))
      IF(ABMAX- A1)31,32,33
   31 FDUNDK=2.*FOUNDK
      GO TD 71
   32 GO TO 13
   33 HK=FOUNDK
      FOUNDK=FOUNDK/2.
  171 CDNTINUE
      CALL SUB(EI(LL),WIDTH(KK))
      IF(ABMAX-A2)34,35,36
   36 FOUNDK=FOUNDK/2.
      GD TO 171
   35 GO TD 13
   34 LK=FOUNDK
  271 FDUNDK=(LK+HK)/2.
      CALL SUB(EI(LL),WIDTH(KK))
      IF((ABMAX- A2).GE.0..AND.(ABMAX-A1).LE.0.)GO TO13
      IF(ABMAX.GT. A1)HK=FOUNDK
      IF(ABMAX.LT.A2  )LK=FDUNDK
      GO TO 271
   52 FOUNDK=200000.
```

Table A.2 cont'd. - Computer program for load distribution analysis.

```
   72 CONTINUE
      CALL BUS(EI(LL),WIDTH(KK))
      IF(ABMAX-A1 )41,42,43
   41 FOUNDK=2.*FOUNDK
      GO TO 72
   42 GO TO 13
   43 HK=FOUNDK
      FOUNDK=FOUNDK/2.
  172 CONTINUE
      CALL BUS(EI(LL),WIDTH(KK))
      IF(ABMAX- A2)44,45,46
   46 FOUNDK=FOUNDK/2.
      GO TO 172
   45 GO TO 13
   44 LK=FOUNDK
  272 FOUNDK=(LK+HK)/2.
      CALL BUS(EI(LL),WIDTH(KK))
      IF((ABMAX- A2).GE.0..AND.(ABMAX-A1).LE.0.)GO TO13
      IF(ABMAX.GT. A1)HK=FOUNDK
      IF(ABMAX.LT.  A2)LK=FOUNDK
      GO TO 272
   13 WRITE(6,91)IPOINT(JJ),WIDTH(KK),EI(LL),FOUNDK      ,DEVA,DEVL,DEVC,A
     1BMAX,YFORA,YFORL,YFORC, AVEY
   91 FORMAT(1H I5,F10.1,2E10.4,4F10.3,4E10.4)
   12 CONTINUE
   11 CONTINUE
      GO TO 3
    2 STOP
      END
C     *
C     *
C      *
      SUBROUTINE SUB(EIS,WIDTHS)
      COMMONZ,T,L1,FOUNKD,ALAF,ABMXA,DIFC,DIFL,DIFA,YSUBA,YSUBL,YSUBC,
     1YAVE,DS
      REAL LAMOK,LAMDL    ,LAMD
      LAMD=(FOUNKD/(4.*EIS))**.25
      LAMOK=LAMD/FOUNKD
      LAMDL=LAMD*WIDTHS
      XLAMD=LAMD*Z
      YLAMD=LAMD*T
      CLAMD=LAMD*WIDTHS/2.
      YSUBA=0.
      YAVE=FLOAT(L1)/(FOUNKD*WIDTHS)
      DO21M=1,L1
      A=(ALAF+FLOAT(M-1))*DS
      B=WIDTHS-A
      ALAMD=LAMD*A
      BLAMD=LAMD*B
   21 YSUBA=YSUBA+LAMOK/(SINH(LAMDL)**2.-SIN(LAMDL)**2.)
     1                       *2.*(SINH(LAMDL)*
     2COS(ALAMD)*COSH(BLAMD)-SIN(LAMDL)*COSH(ALAMD)*COS(BLAMD))
      I0=L1/2
      I1=L1/2+1
      I2=L1/2+2
      YL1=0.
      DO22M=I0,L1
      A=(ALAF+FLOAT(M-1))*DS
```

Table A.2 cont'd. - Computer program for load distribution analysis.

```
      B=WIDTHS-A
      ALAMD=LAMD*A
      BLAMD=LAMD*B
   22 YL1=YL1+LAMOK/(SINH(LAMDL)**2.-SIN(LAMDL)**2.)*(2.*COSH(XLAMD)*COS
     1(XLAMD)*(SINH(LAMDL)*COS(ALAMD)*COSH(BLAMD)-SIN(LAMDL)*COSH(ALAMD)
     2*COS(BLAMD))+(COSH(XLAMD)*SIN(XLAMD)+SINH(XLAMD)*COS(XLAMD))*(SINH
     3(LAMDL)*SIN(ALAMD)*COSH(BLAMD)-SINH(LAMDL)*COS(ALAMD)*SINH(BLAMD)+
     4SIN(LAMDL)*SINH(ALAMD)*COS(BLAMD)-SIN(LAMDL)*COSH(ALAMD)*SIN(BLAMD
     5)))
      YL2=0.
      IF(I2.GT.L1)GO TO 32
      DO23M=I2,L1
      A=(ALAF+FLOAT(M-1))*DS
      B=WIDTHS-A
      ALAMD=LAMD*A
      BLAMD=LAMD*B
   23 YL2=YL2+LAMOK/(SINH(LAMDL)**2.-SIN(LAMDL)**2.)*(2.*COSH(YLAMD)*COS
     1(YLAMD)*(SINH(LAMDL)*COS(ALAMD)*COSH(BLAMD)-SIN(LAMDL)*COSH(ALAMD)
     2*COS(BLAMD))+(COSH(YLAMD)*SIN(YLAMD)+SINH(YLAMD)*COS(YLAMD))*(SINH
     3(LAMDL)*SIN(ALAMD)*COSH(BLAMD)-SINH(LAMDL)*COS(ALAMD)*SINH(BLAMD)+
     4SIN(LAMDL)*SINH(ALAMD)*COS(BLAMD)-SIN(LAMDL)*COSH(ALAMD)*SIN(BLAMD
     5)))
   32 YSUBL=YL1+YL2
      YSC=0.
      DO24M=I1,L1
      A=(ALAF+FLOAT(M-1))*DS
      B=WIDTHS-A
      ALAMD=LAMD*A
      BLAMD=LAMD*B
   24 YSC=YSC+LAMOK/(SINH(LAMDL)**2.-SIN(LAMDL)**2.)*(2.*COSH(CLAMD)*COS
     1(CLAMD)*(SINH(LAMDL)*COS(ALAMD)*COSH(BLAMD)-SIN(LAMDL)*COSH(ALAMD)
     2*COS(BLAMD))+(COSH(CLAMD)*SIN(CLAMD)+SINH(CLAMD)*COS(CLAMD))*(SINH
     3(LAMDL)*SIN(ALAMD)*COSH(BLAMD)-SINH(LAMDL)*COS(ALAMD)*SINH(BLAMD)+
     4SIN(LAMDL)*SINH(ALAMD)*COS(BLAMD)-SIN(LAMDL)*COSH(ALAMD)*SIN(BLAMD
     5)))
      YSUBC=2.*YSC
      DIFA=(YSUBA-YAVE)/YAVE
      DIFL=(YSUBL-YAVE)/YAVE
      DIFC=(YSUBC-YAVE)/YAVE
      ABMXA=AMAX0(ABS(DIFA),ABS(DIFL),ABS(DIFC))
      RETURN
      END
C     *
C     *
C     *
      SUBROUTINE BUS(EIT,WIDTHT)
      COMMON Q,R,L2,KFOUND,AFLA,MAXAB,CDEV,LDEV,ADEV,ADEFL,LDEFL,CDEFL,
     1AVER,DT
      REAL LAMOK,LAMDL,LAMD,LDEFL,MAXAB,KFOUND,LDEV
      LAMD=(KFOUND/(4.*EIT))**.25
      LAMOK=LAMD/KFOUND
      LAMDL=LAMD*WIDTHT
      CLAMD=LAMD*WIDTHT/2.
      ADEFL=0.
      AVER=FLOAT(L2)/(KFOUND*WIDTHT)
      DO111M=1,L2
      A=(AFLA+FLOAT(M-1))*DT
      B=WIDTHT-A
```

Table A.2 cont'd. - Computer program for load distribution analysis.

```
      ALAMD=LAMD*A
      BLAMD=LAMD*B
  111 ADEFL=ADEFL+LAMOK/(SINH(LAMDL)**2.-SIN(LAMDL)**2.)
     1                   *2.*(SINH(LAMDL)*
     2COS(ALAMD)*COSH(BLAMD)-SIN(LAMDL)*COSH(ALAMD)*COS(BLAMD))
      CDEFL=0.
      I3=(L2+3)/2
      IF(I3.EQ.2)GO TO 1
      DO112M=I3,L2
      A=(AFLA+FLOAT(M-1))*DT
      B=WIDTHT-A
      ALAMD=LAMD*A
      BLAMD=LAMD*B
  112 CDEFL=CDEFL+
     A          LAMOK/(SINH(LAMDL)**2.-SIN(LAMDL)**2.)*(2.*COSH(CLAMD)*COS
     1(CLAMD)*(SINH(LAMDL)*COS(ALAMD)*COSH(BLAMD)-SIN(LAMDL)*COSH(ALAMD)
     2*COS(BLAMD))+(COSH(CLAMD)*SIN(CLAMD)+SINH(CLAMD)*COS(CLAMD))*(SINH
     3(LAMDL)*SIN(ALAMD)*COSH(BLAMD)-SINH(LAMDL)*COS(ALAMD)*SINH(BLAMD)+
     4SIN(LAMDL)*SINH(ALAMD)*COS(BLAMD)-SIN(LAMDL)*COSH(ALAMD)*SIN(BLAMD
     5)))
    1 CDEFL=CDEFL*2.
      CDEFL=CDEFL+LAMOK*(COSH(LAMDL)+COS(LAMDL)+2.)/(2.*(SINH(LAMDL)+
     1SIN(LAMDL)))
      LDEFL=CDEFL
      ADEV=(ADEFL-AVER)/AVER
      LDEV=(LDEFL-AVER)/AVER
      CDEV=(CDEFL-AVER)/AVER
      MAXAB=AMAX0(ABS(ADEV),ABS(LDEV),ABS(CDEV))
      RETURN
      END
END DATA.

@BRKPT   PRINT$
```

on a single line of output aligned under the twelve column headings. The first four columns contain the essential information. Column 4 contains the maximum foundation constant sought. It is dependent on the number of load points, the wall width and the distributing beam's flexural stiffness (displayed in columns 1 through 3) plus the value of α , A1 and A2. The 5th, 6th and 7th columns of the output are quotients of the deviations of the foundation deflections, computed at 3 locations, from the average deflection (the uniform deflection which would occur if the load distributing beam were completely rigid), divided by the average deflection. These 3 locations are (a) at either end of the beam (DEVA); (b) at the load points closest to the center for even numbered load cases or at the centrally located load for odd numbered load cases (DEVL); and (c) at the center of the beam (DEVC). Columns 9, 10 and 11 give the quotients of deflections divided by the magnitude of the concentrated loads, for the same 3 (2 for odd numbered loads) points. Column 8 is the largest of the absolute values appearing in columns 5 through 7 and column 12 is the average deflection divided by the magnitude of the concentrated load. It should be noted that the values appearing in column 8 are always between the limits of A1 and A2.

It is assumed that the largest deviation of the displacement from the average displacement - and therefore the largest deviation of the foundation pressure from the average pressure - will occur either at an end of the beam, under a central or near central load, or under the center of the beam. While a proof is not offered, it is believed that the foregoing assumption is a valid one.

Lines 1 through 32 of the main program (see table A.2) are concerned with the furnishing of data and the establishment of iterative loops for the processing of individual sets of data. Line 33 differentiates between odd numbered loads and even numbered loads. These cases are differentiated because odd numbered load cases require a deflection computation at the center and at one end of the beam while even load cases require computed deflections under one of the loads which is closest to the center, at the center and at one end.

Lines 34 to 55 of the main program are used for even numbered load cases. An initial foundation constant of FOUNDK = 200000 is assumed. Subroutine SUB is called upon to give ABMAX. This is the maximum absolute value of deviation of the deflection from the average divided by the average; it is obtained from a consideration of the deflections at the end, under the most central loads and at the center. FOUNDK is doubled until ABMAX is equal to or greater than the foundation constant, which is the object of the search. Then the current value of FOUNDK is halved successively and SUB is called each time until ABMAX is less than A2, the lower limit of the maximum variation which has been chosen. Once an upper (HK) and a lower (LK) limit have been found for FOUNDK, the program takes the average of these until ABMAX falls between A1 and A2. After each unsuccessful trial a new value of HK or LK is established and the process of finding an acceptable FOUNDK is continued. When an acceptable FOUNDK is found the computations are transferred to the write statement on line 78.

Lines 56 to 77 employ the same technique except that subroutine BUS is called during the search for an acceptable value of FOUNDK. Subroutine BUS is suitable for odd numbers of concentrated loads. Statements on lines 81 to 83 complete the iterative loops and allow new sets of four data cards to be accepted when it is desired to vary α, A1 or A2.

Subroutines SUB and BUS use the principal of superposition, together with the appropriate equations for the deflections. While SUB uses only equation 42a from Hetenyi's treatise, BUS also uses a special form (lines 194 and 195) suitable for determining the central deflection due to a central load.

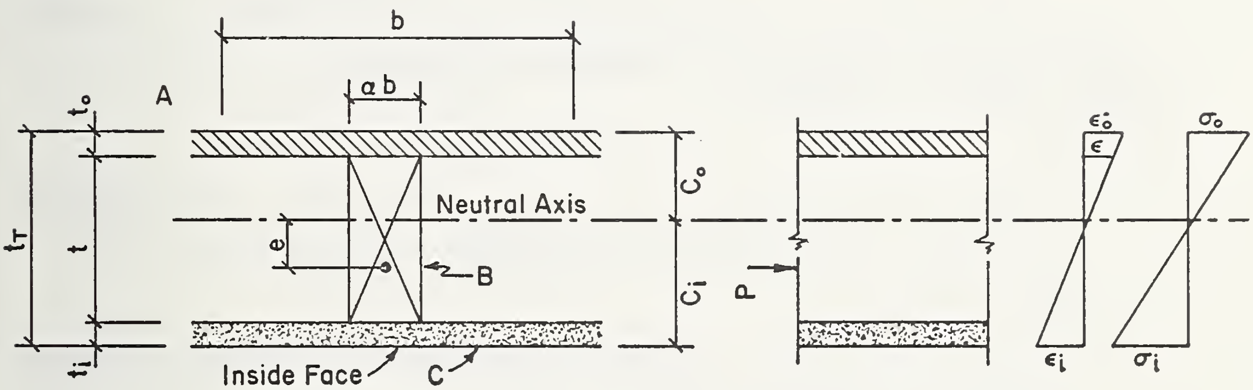
The units used in the computations must be consistent. For example, it has been found the EI in pount-inches squared, and wall width L, in inches, work well with the initialized values of 200000 for FOUNDK used in the main program on lines 34 and 56. The implied units of FOUNDK would then be pounds per inch of wall width per inch of wall shortening (or foundation deflection). Of course α (ALFA in the main program) is unitless since it is the ratio between edge distance and the distance between loads.

The output in columns 5, 6 and 7 are deviations of the deflections from the average divided by the average. A positive value indicates a smaller than average deflection. One can infer from the output of these columns whether the load distributing beam is generally bent so that the concave side is the upper or the lower. Some results have indicated that for larger values of ALFA (above 0.5) the concave side tends to be the top side while for lower values of ALFA the lower side tends to be the concave side. This appears a reasonable result in view of the following considerations. The distributed upward load on the beam can be thought of as having a large uniform value with small superimposed perturbations. When α is large, the concentrated loads acting down are bunched closer to the center than when α is small. This causes the beam to curve so that the concave side tends to be the top side. When α is small, the concave side tends to be the bottom side. An extreme example of the latter case would occur if α were zero and the number of loads were 2. The distribution and amplitudes of the perturbations are dependent on the final value of FOUNDK and the beam's flexural stiffness EI. The influence of the perturbations on the ultimate curvature of the beam is usually negligible; however, in some instances their effects are sufficient so that the sense of the beam's curvature cannot be predicted a priori from a consideration of the number of loads and the value of α alone. In short, the user is warned that in some applications the shape of the beam may differ from intuitive expectations.

Appendix B - Core Radius Calculations for Some Typical Wall Sections

This appendix presents example calculations that were performed using the cross sectional properties of three typical bearing wall constructions. The purpose of the exercise was to demonstrate a method of comparing a "theoretical" core (kern) radius with the eccentricity of loading specified in ASTM Standard Method E72-74. The theoretical core radius was determined on the basis of an assumption of composite behavior exhibited by the walls as they are subjected to eccentric axial loading. The General Formulation describes the step-by-step procedure for computing the properties of the transformed section, which is formed by using the core element as the base material.

GENERAL FORMULATION - TRANSFORMED SECTION CALCULATIONS



I. ASSUMING LINEAR STRAIN & STRESS

$$\varepsilon_o = \sigma_o/E_o, \quad \varepsilon = \sigma/E, \quad \varepsilon_i = \sigma_i/E_i$$

$$\frac{\varepsilon_o}{c_o} = \frac{\varepsilon}{(c_o - t_o)} = \frac{\varepsilon_i}{(t_T - c_o)}$$

$$\frac{\sigma_o}{E_o c_o} = \frac{\sigma}{E(c_o - t_o)} = \frac{\sigma_i}{E_i(t_T - c_o)}$$

II. AREA OF TRANSFORMED SECTION, A_T

$$A_A = (E_o/E)bt_o; \text{ Let } \lambda_o = E_o/E$$

$$A_B = abt$$

$$A_C = (E_i/E)bt_i; \text{ Let } \lambda_i = E_i/E$$

$$A_T = A_A + A_B + A_C$$

$$A_T = \lambda_o bt_o + abt + \lambda_i bt_i$$

$$A_T = b(\lambda_o t_o + at + \lambda_i t_i) \qquad [1]$$

III. TENSILE & COMPRESSIVE STRESSES

$$\sigma_o = \frac{P}{A_T} - \frac{Pec_o}{I_T}$$

$$\sigma_i = \frac{P}{A_T} + \frac{Pec_o}{I_T}$$

If $c_o = 0$, then

$$\frac{P}{A_T} - \frac{Pec_o}{I_T} = 0$$

Let $e = r_c$, since $P \neq 0$

$$r_c = \frac{I_T}{A_T c_o} \qquad [2]$$

IV. LOCATION OF NEUTRAL AXIS

$$c_o = \frac{\Sigma Ay}{\Sigma A}$$

$$c_o = \frac{A_A(t_o/2)+A_B(t_o+t/2)+A_C(t_o+t_i/2)}{A_T}$$

$$c_o = \frac{\lambda_o \cancel{b} t_o(t_o/2)+a\cancel{b}t(t_o+t/2)+\lambda_i \cancel{b} t_i(t_o+t+t_i/2)}{\cancel{b}(\lambda_o t_o+at+\lambda_i t_i)}$$

$$c_o = \frac{\lambda_o t_o^2+2att_o+at^2+\lambda_i(2t_i t_o+2tt_i+t_i^2)}{2(\lambda_o t_o+at+\lambda_i t_i)} \qquad [3]$$

Assuming t_o^2, t_i^2, $t_o t_i << 1$

$$c_o \simeq \frac{2att_o+at^2+2\lambda_i tt_i}{2(\lambda_o t_o+at+\lambda_i t_i)} \qquad [4]$$

V. MOMENT OF INERTIA OF TRANSFORMED SECTION, I_T

$$I_A = \frac{1}{12}(\lambda_o b)t_o^3 + A_A(c_o - t_o/2)^2$$

$$I_A = \frac{\lambda_o bt_o^3}{12} + \frac{\lambda_o bt_o}{4}(2c_o - t_o)^2$$

$$I_A = \lambda_o bt_o(c_o^2 - c_o t_o + 1/3\ t_o^2) \qquad [5]$$

$$I_B = \frac{i}{12}abt^2 + abt(t/2 + t_o - c_o)^2$$

$$I_B = \frac{1}{12}abt^3 + \frac{abt}{4}(t + 25_o - 2c_o)^2$$

$$I_B = \frac{1}{12}abt\ [t^2 + 3(t + 2t_o - 2c_o)^2] \qquad [6]$$

$$I_C = \frac{1}{12}\lambda_i bt_i^3 + A_c(c_i - t_i/2)^2$$

$$I_C = \frac{1}{12}\lambda_i bt_i^3 + \lambda_i bt_i\ (t_T - c_o - t_i/2)^2$$

$$I_C = \frac{1}{12}\lambda_i bt_i[t_i^2+3(2t_T-c_o-t_i)^2] \qquad [7]$$

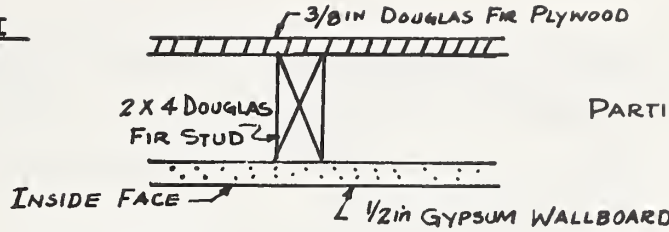
$$I_T = I_A + I_B + I_C \qquad [8]$$

VI. CALCULATING $A_T c_o$

$$A_T c_o = \frac{b}{2}[\lambda_o t_o^2+2att_o+at^2+\lambda_i(2t_i t_o+ 2tt_i+t_i^2)]$$

$$A_T c_o = \frac{b}{2}[\lambda_o t_o^2+2att_o+at^2+\lambda_i(2t_i t_o+ 2tt_i+t_i^2)] \qquad [9]$$

EXAMPLE I



PARTIAL PLAN VIEW

$\alpha b = 1.5$ in $\quad t_o = 0.375$ in
$b = 16$ in $\quad E_o = 1.5 \times 10^6$ psi
$t_i = 0.5$ in $\quad t = 3.5$ in
$E_i = 0.20 \times 10^6$ psi $\quad E = 1.6 \times 10^6$ psi

$t_T = 4.375$ in $\quad t_i^2 = 0.25 \quad t_i t = 1.75$
$\alpha = \alpha b / b = 0.0938 \quad t_o^2 = 0.14 \quad t_i t_o = 0.1875$
$\lambda_o = E_o/E = 0.9375 \quad t^2 = 12.25$
$\lambda_i = E_i/E = 0.125 \quad t_o t = 1.3125$

DISTANCE FROM OUTSIDE FACE TO NEUTRAL AXIS, c_o --- EQN. [3]

$$c_o = \frac{0.938(0.14) + 2(1.3125)(0.0938) + 0.0938(12.25) + 0.125(2 \times 0.1875 + 2 \times 1.75 + 0.25)}{2(0.938 \times 0.375 + 0.0938 \times 3.5 + 0.125 \times 0.5)}$$

$$c_o = 2.042/1.485 = 1.375 \text{ in.}$$

APPROXIMATE c_o --- EQN. [4]

$$c_o \simeq \frac{2(1.3125)(0.0938) + 0.0938(12.25) + 2(0.125)(1.75)}{1.485} = 1.234 \text{ in.}$$

MOMENT OF INERTIA OF TRANSFORMED AREA, I_T --- EQNS. [5], [6], [7] & [8]

$c_o^2 = (1.375)^2 = 1.8906 \qquad \lambda_o b t_o = 0.9375(16)(0.375) = 5.625$
$c_o t_o = (1.375)(0.375) = 0.5156 \qquad \lambda_i b t_i/12 = 0.125(16)(0.5)/12 = 0.0833$
$2c_o = 2(1.375) = 2.75 \qquad \alpha b t/12 = 1.5(3.5)/12 = 0.438$
$2t_T = 2(4.375) = 8.75$

$$I_A = 5.625\,(1.8906 - 0.5156 + 1/3 \times 0.14) = 5.625\,(1.422) = 7.996$$
$$I_B = 0.438\,[12.25 + 3(3.5 + 2 \times 0.375 - 2.75)^2] = 8.322$$
$$I_C = 0.0833\,[0.25 + 3(8.75 - 2.75 - 0.25)^2] = 8.283$$

$$I_T = I_A + I_B + I_C$$
$$I_T = 7.996 + 8.322 + 8.283 = 24.60 \text{ in.}^4$$

ECCENTRICITY TO MAKE $\sigma_o = 0$, r_c --- EQNS. [2] & [9]

$$r_c = I_T / A_T c_o$$

$$A_T c_o = \frac{16}{2}[0.938(0.14) + 2(0.0938)(1.3125) + 0.0988(12.25) + 0.125(2 \times 0.1875 + 2 \times 1.75 + 0.25)]$$

$$A_T c_o = 8(2.042) = 16.336$$

$$r_c = 24.60/16.336 = 1.51 \text{ in.}$$

COMPARISON OF r_c WITH THE ECCENTRICITY FROM SECT. 7 OF ASTM E72

CALCULATED FROM ABOVE -

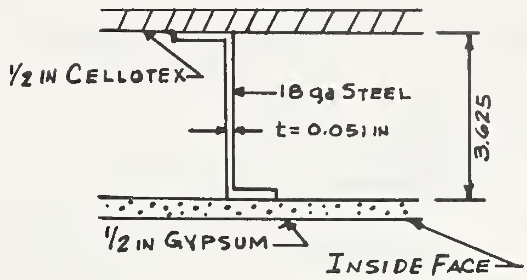
$t_T - (c_o + r_c) = 4.375 - (1.375 + 1.51) = 1.49$ (DIST. FROM INSIDE FACE)

FROM SECT. 7 OF ASTM E72 -

$t_T / 3 = 4.375/3 = 1.46$ IN (DIST. FROM INSIDE FACE)

PERCENT DIFFERENCE $= \dfrac{1.49 - 1.46}{1.49} = 2\%$

EXAMPLE II



PARTIAL PLAN

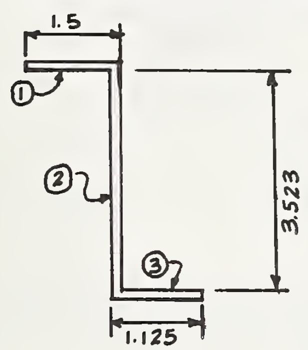
$b = 24$ IN
$t_i = 0.5$ IN
$E_i = 0.2 \times 10^6$ psi

$t_o = 0.5$ IN
$E_o = 0.1 \times 10^6$ psi
$E = 30 \times 10^6$ psi

$t_T = 4.625$ IN
$\lambda_o = 0.1/30 = 0.0033$
$\lambda_i = 0.2/30 = 0.0066$

$t_i^2 = 0.25$
$t_o^2 = 0.25$
$t_i t_o = 0.25$

CALCULATION OF MOMENT OF INERTIA OF ZEE SECTION



$t = 0.051$ IN.

AREA

$A① = 1.5(0.051) = 0.0765$
$A② = 3.523(0.051) = 0.1796$
$A③ = 1.125(0.051) = 0.0574$
$\Sigma A = 0.3135 \text{ IN}^2$

NEUTRAL AXIS LOCATION

$A① Y① = 0.0765(0.051/2) = 0.0020$

$A② Y② = 0.1796(3.523/2 + 0.051) = 0.3255$

$A③ Y③ = 0.0574(3.625 - 0.051/2) = 0.2066$

$\Sigma AY = 0.5341 \text{ IN}^3$

$$\bar{Y} = \frac{\Sigma AY}{\Sigma A} = \frac{0.5341}{0.3125} = 1.7037 \text{ IN (FROM TOP)}$$

MOMENT OF INERTIA

$I① = 1/12(1.5)(0.051)^3 + 0.0765(1.7037 - 0.051/2)^2$
$I① = 0.128$
$I② = 1/12(0.051)(3.523)^3 + 0.1796(1.813 - 1.7037)^2$
$I② = 0.1879$
$I③ = 1/12(1.125)(0.051)^3 + 0.0574(3.5995 - 1.7031)^2$
$I③ = 0.2660$

$\Sigma I = 0.5819 \text{ IN}^4$

DISTANCE FROM OUTSIDE FACE TO THE NEUTRAL AXIS, C_o

$$A_A = 0.033(24)(0.5) = 0.0399$$
$$A_B = 0.051(1.5 + 1.125 + 3.523) = 0.3135$$
$$A_C = 0.066(24)(0.5) = 0.0798$$
$$A_T = 0.4332 \text{ IN}^2$$

$$C_o = \frac{0.0399(0.25) + 0.3135(0.5 + 1.8125) + 0.0798(0.5 + 3.625 + 0.25)}{0.4332}$$

$$C_o = \frac{0.010 + 0.724 + 0.349}{0.4332} = 2.502 \text{ IN}$$

MOMENT OF INERTIA OF TRANSFORMED AREA, I_T

$$C_o^2 = (2.502)^2 = 6.26$$
$$C_o t_o = (2.502)(0.5) = 1.25$$
$$2C_o = 2(2.502) = 5.00$$
$$2t_T = 2(4.625) = 9.25$$

$$\lambda_o b t_o = 0.0033(24)(0.5) = 0.0399$$
$$\frac{\lambda_i b t_i}{12} = \frac{0.0066(24)(0.5)}{12} = 0.0066$$

$$I_A = 0.0399[6.26 - 1.25 + 1/3(0.25)] = 0.2032$$
$$I_B = 0.5819 + 0.3135[2.50 - (1.7037 + 0.5)]^2 = 0.6098$$
$$I_C = 0.0066[0.25 + 3(9.25 - 5.00 - 0.5)^2] = 0.2794$$
$$I_T = 1.0924 \text{ IN}^4$$

ECCENTRICITY TO MAKE $\sigma_o = 0$, r_c

$$r_c = \frac{1.0924}{0.4332(2.502)} = 1.008 \text{ IN}$$

COMPARISON OF r_c WITH ECCENTRICITY IN SECTION 7 OF ASTM E72

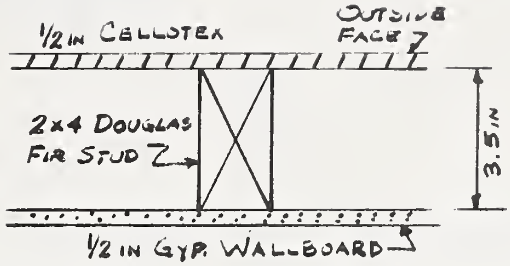
DISTANCE FROM INSIDE FACE

$$t_T - (r_c + c_o) = 4.625 - (1.008 + 2.502) = 1.115 \text{ IN}$$

$$t_T/3 = 4.625/3 = 1.541 \text{ IN}$$

$$\text{PERCENT DIFFERENCE} = \frac{1.541 - 1.115}{1.115} = 38\%$$

EXAMPLE III



PARTIAL PLAN

$\alpha b = 1.5$ IN
$b = 16$ IN
$t_i = 0.5$ IN

$E_i = 0.20 \times 10^6$ psi
$t_o = 0.375$ IN
$E_o = 0.10 \times 10^6$ psi

$t = 3.5$ IN
$E = 1.6 \times 10^6$ psi

$t_T = 4.5$ IN
$\alpha = 0.0938$
$\lambda_o = 0.1/1.6 = 0.0625$
$\lambda_i = 0.2/1.6 = 0.125$

$t_i^2 = 0.25$
$t_o^2 = 0.25$
$t^2 = 12.25$

$t_o t = 1.75$
$t_i t = 1.75$
$t_i t_o = 0.25$

DISTANCE FROM OUTSIDE FACE TO NEUTRAL AXIS, C_o

$$C_o = \frac{0.0625(0.25) + 2(1.75)(0.0938) + 0.0938(12.25) + 0.125[2(0.25) + 2(1.75) + 0.25]}{2[0.0625(0.5) + 0.0938(3.5) + 0.125(0.5)]}$$

$$C_o = \frac{2.024}{0.8441} = 2.3979 \text{ IN} \simeq 2.40 \text{ IN}$$

MOMENT OF INERTIA OF TRANSFORMED AREA, I_T

$C_o^2 = (2.40)^2 = 5.76$
$C_o t_o = (2.40)(0.5) = 1.20$
$2C_o = 2(2.40) = 4.80$
$2t_T = 2(4.50) = 9.00$

$\lambda_o b t_o = 0.0625(16)(0.5) = 0.50$
$\lambda_i b t_i/12 = 0.125(16)(0.5)/12 = 0.083$
$\alpha b t/12 = 1.5(3.5)/12 = 0.438$

$I_A = 0.5(5.76 - 1.20 + 0.25/3) = 2.322$
$I_B = 0.438[12.25 + 3(3.50 + 1.0 - 4.8)^2] = 5.484$
$I_C = 0.083[0.25 + 3(9.0 - 4.80 - 0.50)^2] = 3.442$

$I_T = 11.248 \text{ IN}^4$

ECCENTRICITY TO MAKE $\sigma_o = 0$, r_c

$A_T C_o = 8[0.0625(0.25) + 2(0.0938)(1.75) + 0.0938(12.25) + 0.125(0.5 + 3.5 + 0.25)]$
$A_T C_o = 8(2.024) = 16.336$

$r_c = 11.248 / 16.336 = 0.688$ IN

COMPARISON OF r_c WITH ECCENTRICITY IN SECTION 7 OF ASTM E72

DISTANCE FROM THE INSIDE FACE

$t_T - (r_c + C_o) = 4.5 - 3.088 = 1.412$ IN
$t_T/3 = 4.5/3 = 1.5$ IN

PERCENT DIFFERENCE $= \dfrac{1.5 - 1.412}{1.412} = 6\%$

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16. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)*

An experimental and analytical investigation of the primary factors involved in the testing of prototype wall panels under axial compression loading is reported. The objective of the investigation was to develop a method of testing wall specimens that incorporates the best features of ASTM Standard Method E 72 while at the same time incorporating improvements in the areas of dificiency in the Standard. Twenty-five laboratory tests were conducted on samples composed of five types of wall panel construction. The panels were tested to failure under either of two different eccentricities of load, while being supported with one of two types of idealized end conditions. Selected test results and detailed descriptions of the laboratory procedures used are presented. A computer-aided analytical study of the variables affecting the degree of uniformity of loading was conducted. Equations based on the analogy of beams supported on elastic foundations were used in the analysis. A study of the statistical parameters commonly used to interpret test results was conducted to establish useful guidelines for predicting structural performance on the basis of small sample test results. A compression test method applicable to traditional and innovative wall constructions is presented. The principal additions in the revised test method are as follows: (a) a provision for variable eccentricity, (b) a procedure for selecting a load distribution assembly which will be compatible with the test panel.

17. KEY WORDS *(six to twelve entries; alphabetical order; capitalize only the first letter of the first key word unless a proper name; separated by semicolons)* Compression; eccentric loading; flat-end; kern; loading rate; pin-end; test method; wall panels; walls.

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