3353

NATIONAL BUREAU OF STANDARDS REPORT

3353

TEST OF PRECAST PANEL FRAME NO. 1

by

Leopold F. Skoda and D. Watstein

Report to
Bureau of Yards and Docks
Department of the Navy

NBS

U. S. DEPARTMENT OF COMMERCE
NATIONAL BUREAU OF STANDARDS

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NBS PROJECT **NBS REPORT**

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Abstract

The stiffness and strength of a precast panel frame were determined under a load uniformly distributed over the roof area. The frame consisted of two wall panels and two roof panels assembled with welded and bolted connections. The span of the frame was 30 ft; the wall panels were 9 ft 8 in. high and 4 ft wide and the roof panels were 15 ft 5 in. by 4 ft forming a pitched roof with a rise of 1.87 in. per foot.

The frame was loaded with concrete sand. A uniformly distributed load of 40 psf was applied in four increments and was maintained on the frame for a period of 41 days. An additional load of 40 psf was applied then, and the total load of 80 psf was left on the structure for a period of 4 days. The load was then removed and the structure dismantled.

Measurements during this test included determination of rotation at the crown and the two knee joints, deflection at the center and measurements of the width of cracks at the three joints.

1. INTRODUCTION

The test of a precast panel frame was made at the request of the Bureau of Yards and Docks for the purpose of evaluating such a structure under severe uniformly distributed loads to determine any existing points of weakness in the proposed design.

The following report represents observations and data obtained during construction, erection, testing and dismantling of precast panel frame No. 1

2. CONSTRUCTION OF PANELS

2.1 General Description of Specimen

Precast panel frame No. 1 consisted of two wall panels and two roof panels constructed of precast, thin-shell reinforced concrete, in accordance with the Bureau of Yards and Docks plans and specifications titled, "Precast Panel Frame," Sketch A, dated September 8, 1953. (Fig. 1).

The wall panels were 9 ft 8-in. high and 4 ft wide with a 3 ft by 4 ft 1-in. window opening located 3 ft from the floor level. Tapered stiffening beams were located at the outer edges and varied in depth from 1 ft 2 in. at the top to 6 in. at the bottom of the panel, with the inner surface being held plumb. Transverse stiffening ribs were located at top and bottom of the panel and also above and below the window opening. The panel above the window had a 2-in. web thickness and the lower panel had a 1-in. web thickness.

The roof panels were 15 ft 5-in. long and 4 ft wide with tapered longitudinal stiffening beams at the outer edges which varied in depth from 1 ft 2 in. at the wall-roof joint to 8 in. at the crown. Transverse stiffening ribs were located at either end of the roof sections. The web thickness of the roof panels was 1 in. throughout. The roof panels were designed to produce a pitched roof spanning 15 ft each, with a rise of 1.866 in. per foot.

The sections were joined together with welded and bolted connections. The end and crown connections will be described more fully in succeeding paragraphs.

2.2 Forms

Due to the fact that the proposed structure was bilaterally symmetrical, it was only necessary to construct forms for one wall panel and one roof panel. The forms were constructed at the National Bureau of Standards carpenter shop. The materials used for all cast surfaces were 3/4 in. plywood. The forms were so constructed as to be easily disassembled and reused. They were rigidly braced with structural timber in order to eliminate warping and swelling. Further precautions were taken to assure duplication of the original specimens on subsequent castings by coating all surfaces with a special phenol formaldehyde resin varnish. Prior to casting, all interior surfaces were coated with a mixture of oil and asbestine powder to facilitate stripping. In spite of all these precautions some displacement of concrete was observed which

indicated that any mass production of specimens would require metal or concrete forms to produce the desired accuracy and duplication of specimens.

2.3 Concrete

The proportions of the concrete mix were 1:2.48:2.02 by weight. High-early-strength cement was used in order to expedite the curing of the specimens. The aggregates were White Marsh, Maryland sand and pea gravel, maximum size of the coarse aggregate being 3/8 in. Six batches of concrete were mixed for each casting operation and the slump varied from 4 1/2 in. to 6 in. Two standard 6 in. by 12 in. control cylinders were cast for each batch and an average compressive strength of 7200 psi for the first casting, and 6370 psi for the second casting was realized at time of test. Stress-strain curves were determined for each set of concrete cylinders and are shown in figure 5. All concrete specimens were cured for two days under damp burlap; the wall panels were then transferred to a curing chamber where they were cured at 72° F and 100 percent relative humidity until it was necessary to remove them for assembly. The roof panels were too large to be put into the curing chamber so the damp burlap curing was continued for a period of 10 days.

2.4 Reinforcement

The wall panels were reinforced with No. 4 and No. 7 reinforcing bars and 2- by 2-in. by 12 gage welded wire fabric. Two No. 4 reinforcing bars were located in a plane bisecting the web longitudinally at a distance of 1 1/4 in. from the outer edges. Two No. 4 bent bars were used to reinforce the concrete adjacent to the window openings and were placed so as to tie in the vertical legs with the transverse sill extending into the sill to a depth of 8 in. Two No. 4 bent bars were placed in the upper sill and extended 8 in. into both vertical legs. One No. 4 bent bar was placed in the lower sill and extended 8 in. into each vertical leg. A single No. 7 bar was placed in each edge beam along the outer edge and was bent so as to form anchorage for the weld connector plate. A 2 1/2- by 1/2- by 6-in. plate was welded to the end of the No. 7 bar and was so arranged as to be half exposed and match a similar plate extending from the roof panel.

The roof panels were reinforced with No. 4 and No. 6 reinforcing bars and 2- by 2-in. by 12 gage welded wire fabric. A single No. 4 bar was located at the bottom of each longitudinal edge beam and was bent at each end to extend upwards toward the web. A single No. 6 bar was placed near the top of each longitudinal edge beam with a 2 1/2- by 1/2- by 6-in. steel plate welded to it at the wall end of the panel. A 4 ft

piece of No. 4 bar was placed near the bottom of the transverse rib at the crown. It is emphasized that this bar was not anchored at the ends and the effect of the absence of anchorage is discussed under Test Results.

The 2- by 2-in. by 12 gage welded wire fabric was placed in a single layer throughout the web and all reinforcing members of both the wall and roof sections.

At each end of the roof panel 1-in. pipe sleeves were cast into the transverse ribs 3 ft on center. They were placed so as to allow 3/4-in. bolts to be used to hold the panels together. Matching sleeves were also cast into the upper web of the wall panel to permit bolted connections to be made.

3. ASSEMBLY OF PANELS

3.1 Footings

It was decided that the footings included on the proposed drawing of "Precast Panel Frame" were inadequate for this test due to the fact that it would have been necessary to cast a floor section to tie the footings together. Footings were then designed to resist the maximum possible thrust imparted to them by any extreme loading of the panel frame.

The footings consisted of heavily reinforced concrete blocks 10- by 26- by 60 in. with a 2- by 10-in. cavity centered along their length. Four No. 9 reinforcing bars anchored at each end served to tie the footings together. Control cylinders representing the footings were cast and an average compressive strength of 7200 psi was realized at time of test.

3.2 Wall and Roof Erection

In order to aid erection, a timber structure was designed to support the wall and roof panels while the bolted and welded connections were made. It consisted of a center pier of upright 4- by 4-in. timbers cross-braced with 2 by 4's. Planking extended from the center pier beyond the footings at a level with the top of the window openings to provide anchorage for the walls during the erection of the roof panels. Braces extended from the footings to the center pier to counteract any lateral movement.

The wall panels were then set in place with the aid of an electric lift truck and securely wedged and braced against the extended planking. The walls were purposely set out of plumb to allow more room for the roof panels to be placed.

Because of the limited space in the laboratory the roof panels had to be hoisted in place with the use of the electric lift truck and an overhead crane. The roof panels were placed with the crown end resting on the center pier and the wall end suspended from the overhead crane. The 3/4-in. bolts were inserted and tightened to make the wall roof connection. After the two roof panels had been placed and secured to the wall panels the bolts at the crown were inserted and tightened pulling the roof panels together and thereby plumbing the attached side walls.

3.3 Welding and Grouting

After all bolted connections were securely tightened, the welded connections were made at each corner of the roof-wall joint. All welding was done by a qualified Navy welder. A welding plate 1 1/2- by 1/2- by 4 in. was centered over the exposed connector plates so that welding could be accomplished on all four sides of the plates.

During the welding operation cracks appeared in the concrete adjacent to the embedded connector plates. The cracks appeared on both wall and roof panels parallel to the vertical end of the reinforcing bars which served to anchor the connector plates. The cracks averaged from 2 to 4 in. in length and appeared at all four welded connections. Due to the heat applied to the connector plates a slight amount of buckling occurred causing the connector plates to be displaced and break bond from the concrete. It was thought at first that the cracks would close on cooling of the plates but the distortion was permanent and no such closing was observed.

As soon as the welded connections had cooled sufficiently, the bolts were grouted in with a cement-water paste mixed to a cake batter consistency. This was accomplished with a caulking gun that had been adapted to fit the imbedded 1-in. pipe sleeves. Each bolt was grouted individually by removing the nut and washer, applying the grout, and replacing and tightening the assembly. Control cylinders of the grouting mixture had a compressive strength of 5810 psi at the time of test.

This procedure appeared to be satisfactory until the specimen was taken apart after the test. On close observation, it was evident that some of the grout had penetrated the pipe sleeves at the joint between the wall and roof thereby causing a slight separation of the wall from the roof. This separation was noticed during the test but it was thought to have been caused solely by the fact that a

cast surface was adjoining a troweled surface. These facts clearly indicate that some revision is necessary in the grouting procedure or in the bolt-sleeve assembly in order to allow more bearing area between the wall and roof sections.

After the bolt grouting was completed, the footing pockets, in which the wall sections were resting was filled with a 1:3 mortar mix. A compressive test of 6- by 12-in. control cylinders showed a strength of 6230 psi at time of test.

4. TESTING PROCEDURE AND RESULTS

4.1 Test Set-Up

It was decided to use measured amounts of damp sand as the medium for applying the specified uniformly distributed load. Plywood forms 12-in. high were made to extend about the perimeter of the roof sections in order to hold the sand in place. The forms had flexible couplings every two feet to compensate for bending in the roof panels. The amount of sand to be applied was determined by a calculation of the projected roof area. Sand was then weighed in 100 lb quantities and placed in heavy duty paper bags ready to be applied.

4.2 Instrumentation

Measurements of deflections, rotations and crack widths was accomplished with 0.001 in. Federal dial gages. Footing movements were measured with 0.001 in. Ames dial gages. A single gage was used to measure the deflection of the roof panels and was placed as near the center of the structure as possible. Rotations were measured at all four corner joints and at the center joint. Rotation of the side walls were measured with respect to the footings and also with respect to the floor. Crack widths were measured at the end joints and center joints. Side sway of the entire structure was measured at the center of the structure with respect to the floor. Details of the position of the gages can be seen in figures 2, 3, and 4.

4.3 Rate of Loading and Unloading

At the inception, this test was to have been a short term test to destruction, but due to the steady increase in deflection after application of the first two increments of load it was decided to apply a load equal to two "design loads" and allow enough time to elapse so that the specimen would reach a state of equilibrium before any further loads were applied.

Loads were applied in 10 lb/sq ft increments by distributing six 100 lb bags of sand over each roof panel. The sand was placed one bag at a time on each end and spread over one sixth of the area of each roof panel. The bags were distributed alternately on each panel until the required amount of sand had been applied.

One design load of 20 lb/sq ft was applied during the course of the first day and readings were taken before and after each increment of load. The following day another design load of 20 lb/sq ft was added making a total of 40 lb/sq ft. As before, readings were taken before and after application of additional load.

Readings were taken and recorded every day for a period of 24 days at which time the structure appeared to have reached equilibrium. On the twenty fifth day two additional design loads were applied making a total load of 80 lb/sq ft, or four times the design load. No planes of weakness were noticed at this time and the structure was allowed to remain so loaded for an additional five days until it again reached equilibrium.

The structure was then unloaded in a manner similar to the manner in which it had been loaded. Readings were taken to determine immediate recovery and a period of seven days elapsed until maximum recovery was realized. The structure was then carefully disassembled and stored for future testing.

4.4 Test Results

The stress-strain curves obtained for the concrete used in Precast Panel Frame No. 1 are shown in figure 5. The original casting schedule was such as to produce a wall panel and roof panel for each of two castings; however, the roof panel of the second pour was damaged during removal from the form and necessitated casting on additional roof panel. The concrete in the first wall and roof section was 62 days old at the beginning of the test and a compressive strength of 7210 psi was realized from the 6- by 12-in. control cylinders. Wall panel No. 2 was 48 days old and roof panel No. 3 was 31 days old at the beginning of the test and produced compressive strengths of 6530 psi and 6870, respectively.. The tangent modulus of the average of all sections was 4,650,000 psi.

Compressive tests were made of control cylinders of the 1:3 mortar mix that was used to anchor the wall sections in the footings and a strength of 6230 psi was realized at 25 days. The compressive strength of the neat cement paste used to grout in the bolts was 5810 psi at 27 days.

The first visible cracks occurred at the inside intersection of the wall with the footing at a load of 30 psf both at the north and south ends. No further cracking was observed until a load of 40 psf was applied at which time cracking was observed at the intersection of the transverse ribs and longitudinal legs of the roof panels at the crown. Cracks occurred at each of the four intersections and were the most prominent to appear throughout the test. These cracks are clearly visible in figure 14.

It is recalled that the reinforcing bars in the transverse ribs terminated at the midplanes of the longitudinal edge beams. Had the reinforcement in the transverse ribs been extended into the edge beams, the width of the cracks at the corners would have been considerably less, and the opening at the bottom of the crown joint would have been less pronounced.

Also observed at the 40 psf load was longitudinal cracking on the under-side of the roof panels located approximately at the third points and extending the length of the panels. Transverse cracks were also observed in the roof panels about 2 ft from the wall. The cracks in the roof panel are visible in figure 12.

Prominent cracking also occurred at the bolted connections of the wall and roof sections. A typical crack of this kind can be seen in figure 12. When these cracks appeared, it was believed that they were caused by high bearing stresses in the immediate vicinity of the bolted connections. This hypothesis was confirmed by examination of the bearing surfaces of the roof and wall panels upon dismantling the structure. It was found that some of the cement grout which had been forced between the bolts and pipe sleeves had come between the wall and roof panels causing a slight separation and therefore a stress concentration. This result dictates either a change in the application of the bolt grout or abandonment of the procedure altogether to insure more bearing area between the wall and roof panels. It appears that greater precautions will have to be taken during the casting of sections to insure proper fit and consequently greater bearing area between all sections.

The greatest movement was observed, at the crown. A single 1/1000 in. dial gage was placed at the center of the specimen at the crown as seen in figure 4. The maximum deflection occurred just prior to the removal of the total load of 80 psf and amounted to 2.88 inches. Upon removal of load the observed recovery was 1.15 inches.

The spread of the entire structure reached a maximum at the same time and the average spread of both ends was .46 in. accompanied by a recovery of .20 in.

Figure 7 shows the deflection of the center joint and the spread at ends plotted against the applied load. The displacements plotted are the values observed immediately after application of each load increment and do not include the increases resulting from the loads being sustained over long periods of time. It is noted that the structure was subjected to a load of 40 psf for a period of 41 days, after which the load was increased to 80 psf. Figure 7 shows that the structure was apparently stiffened by the sustained load test, as can be seen from the steeper slope of the load-deflection curve for the increments of load added subsequent to the sustained load test.

Measurements were made of rotation at the end joints, center joint and also of the side walls with respect to the footings. Figures 3 and 4 show the instrumentation used for the measurement of the rotation at the end joints and center joint, respectively. Figures 8 and 9 give the results of these measurements plotting rotation in radians against time. Rotation at the center and end joints reached a maximum four days after the application on the 80 psf increment of load.

The maximum observed rotation at the center joint was .0325 radians with a recovery of .0120 radians observed after removal of live load. The average of north and south end rotations was .0080 radians with a recovery of .0053 radians.

The observed rotation at the footings reached a maximum just prior to removal of the live load and was .000117 radians with a resulting recovery of .000047 radians.

The calculated instantaneous rotations as shown in figure 10 verify the conclusions stated with regard to the instantaneous deflections shown in figure 7.

Changes in the width of crack between the wall and roof sections and also at the crown were measured with 1/1000 in. dial gages as seen in figures 3 and 4. The gages were placed in such a way as to measure the maximum opening. Figure 11 is a graphic picture of the results of these measurements. The maximum crack opening at the crown occurred at the maximum load of 80 psf and was .25 in. just prior to removal of load. The maximum crack opening between the wall and roof panels was .075 in. Recovery was observed for six days after the removal of the live load and amounted to .098 in. for the crack at the crown and .030 in. for the cracks between the wall and roof panels.

As can be seen in figures 6, 8, 9, and 11, the displacements and rotations observed in the structure were affected markedly by the duration of the test as well as by the magnitude of the applied load. A time increment of 41 days elapsed

between the application of the 40 psf load and the application of the 80 psf load. The continued increase in deflections and rotations under the second increment of 40 psf indicates that considerably larger movements in the structure would have been observed had the maximum load of 80 psf been allowed to remain on the structure for a longer period of time.

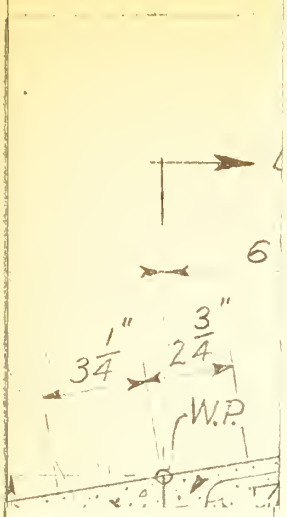
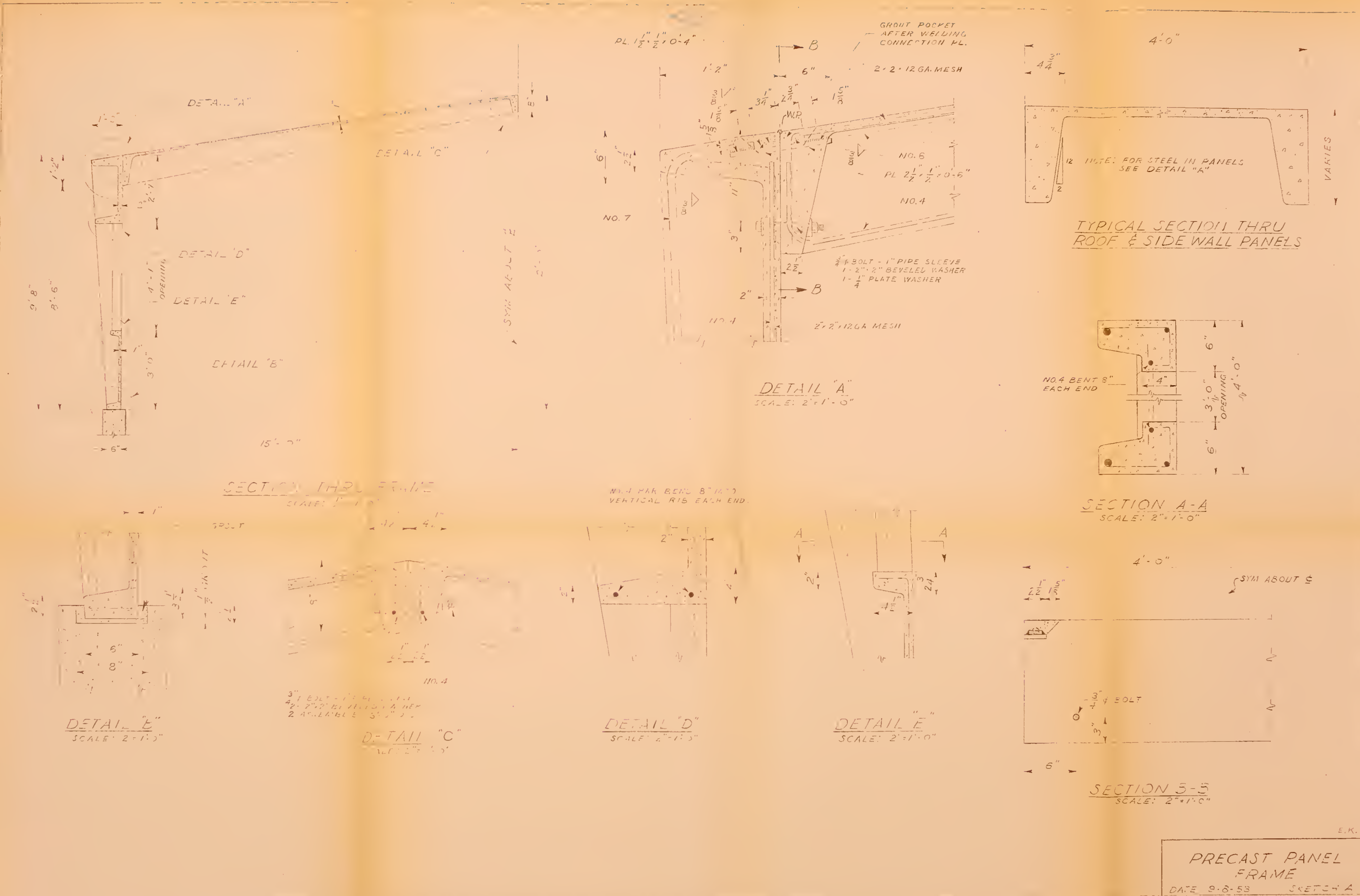

6
3 1/4"
2 3/4"
W.P.

FIG. 1



SECTION THRU FRAME
DETAIL "A"
SCALE: 2"=1'-0"
GROUT POCKET AFTER WELDING CONNECTION PL.
2×2×12 GA. MESH
NO. 6
NO. 4
NO. 7
3/4" φ BOLT - 1" PIPE SLEEVE
1 - 2"×2" BEVELED WASHER
1 - 1/4" PLATE WASHER
2"×2"×12 GA. MESH
TYPICAL SECTION THRU ROOF & SIDE WALL PANELS
NOTE: FOR STEEL IN PANELS SEE DETAIL "A"
VARIES
SECTION A-A
SCALE: 2"=1'-0"
NO. 4 BENT 8" EACH END
OPENING
SECTION B-B
SCALE: 2"=1'-0"
SYM ABOUT ℄
3/4" φ BOLT
DETAIL "B"
DETAIL "C"
DETAIL "D"
DETAIL "E"
SCALE: 2"=1'-0"
PRECAST PANEL FRAME
DATE 9-8-53
SKETCH A

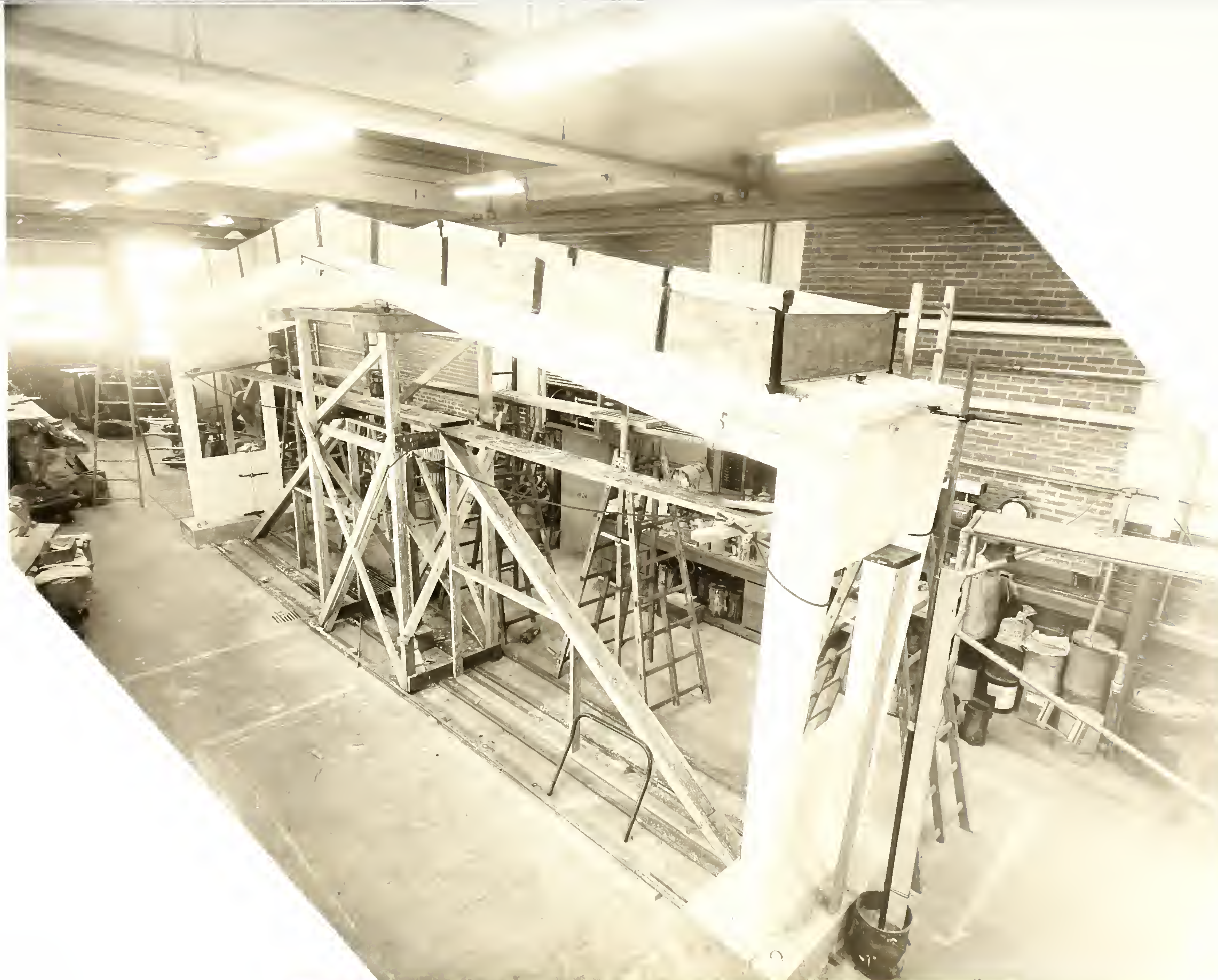


Fig. 2 - Precast panel frame (before test)

Fig. 3 - South end joint (before test)

Fig. 4 - Center joint (before test)

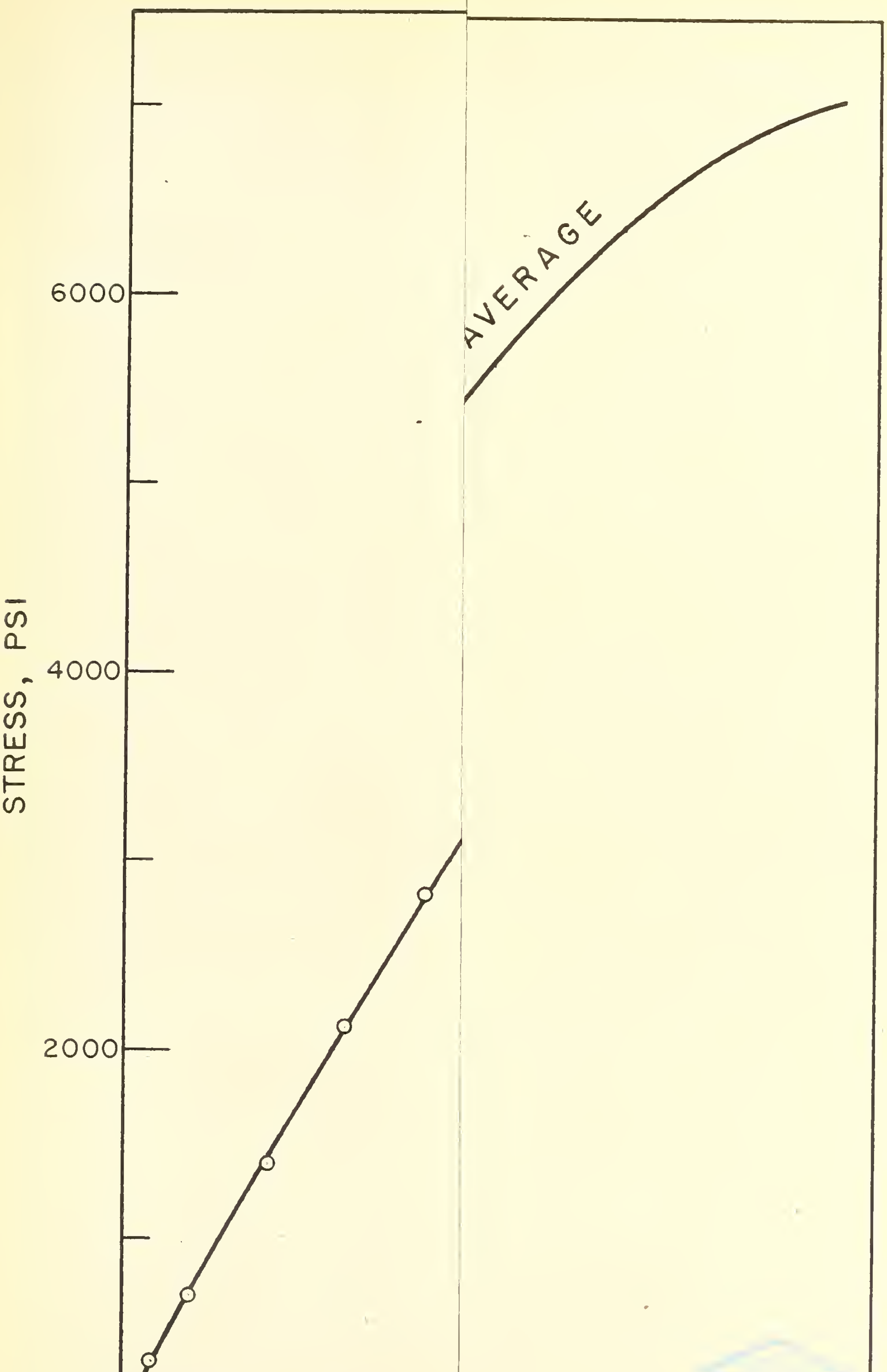
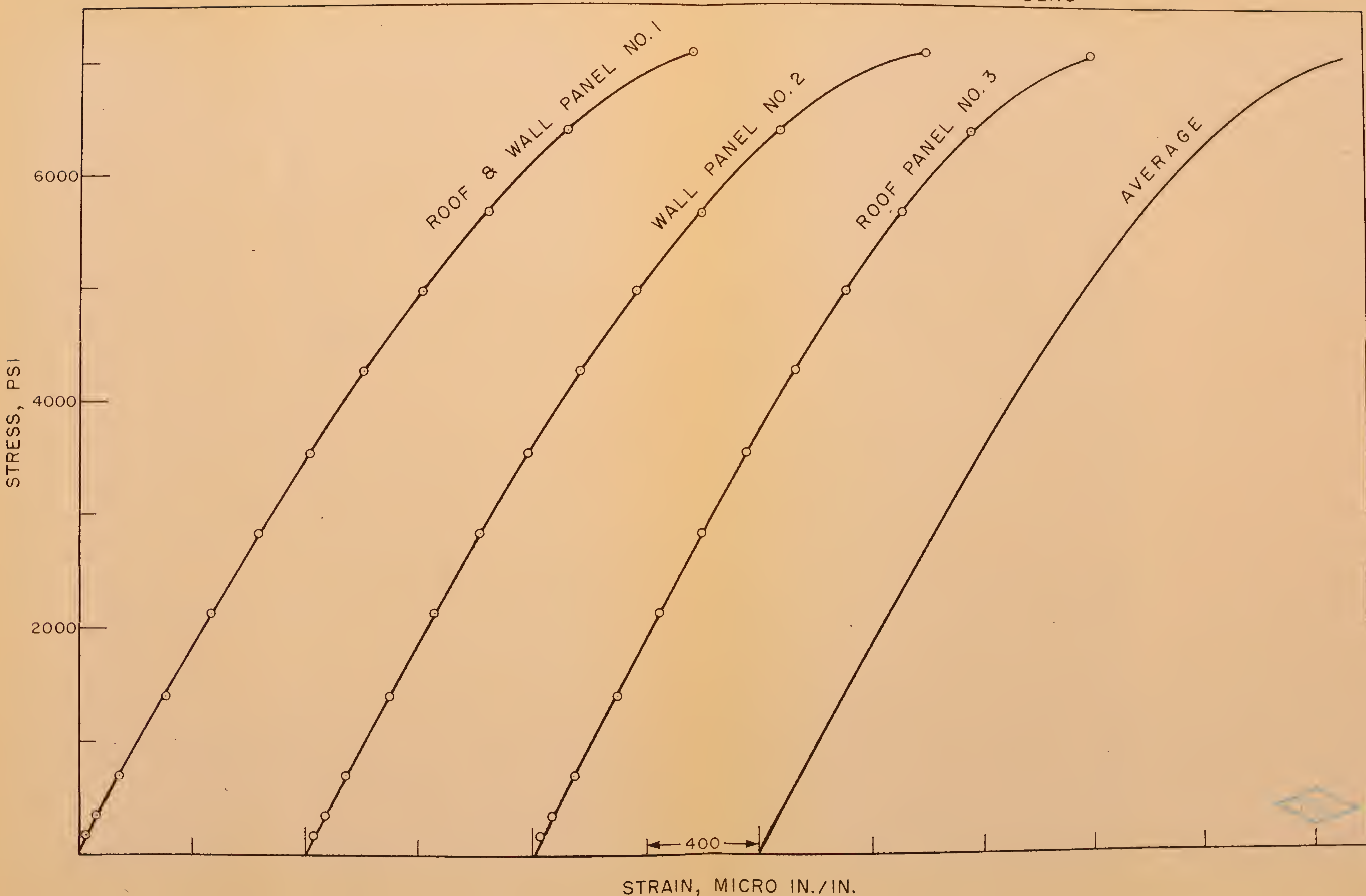

STRESS, PSI
6000
4000
2000
AVERAGE

FIG. 5

STRESS-STRAIN CURVES OF PRECAST PANEL FRAME NO. 1

COMPRESSIVE TESTS OF 6" x 12" CONCRETE CONTROL CYLINDERS



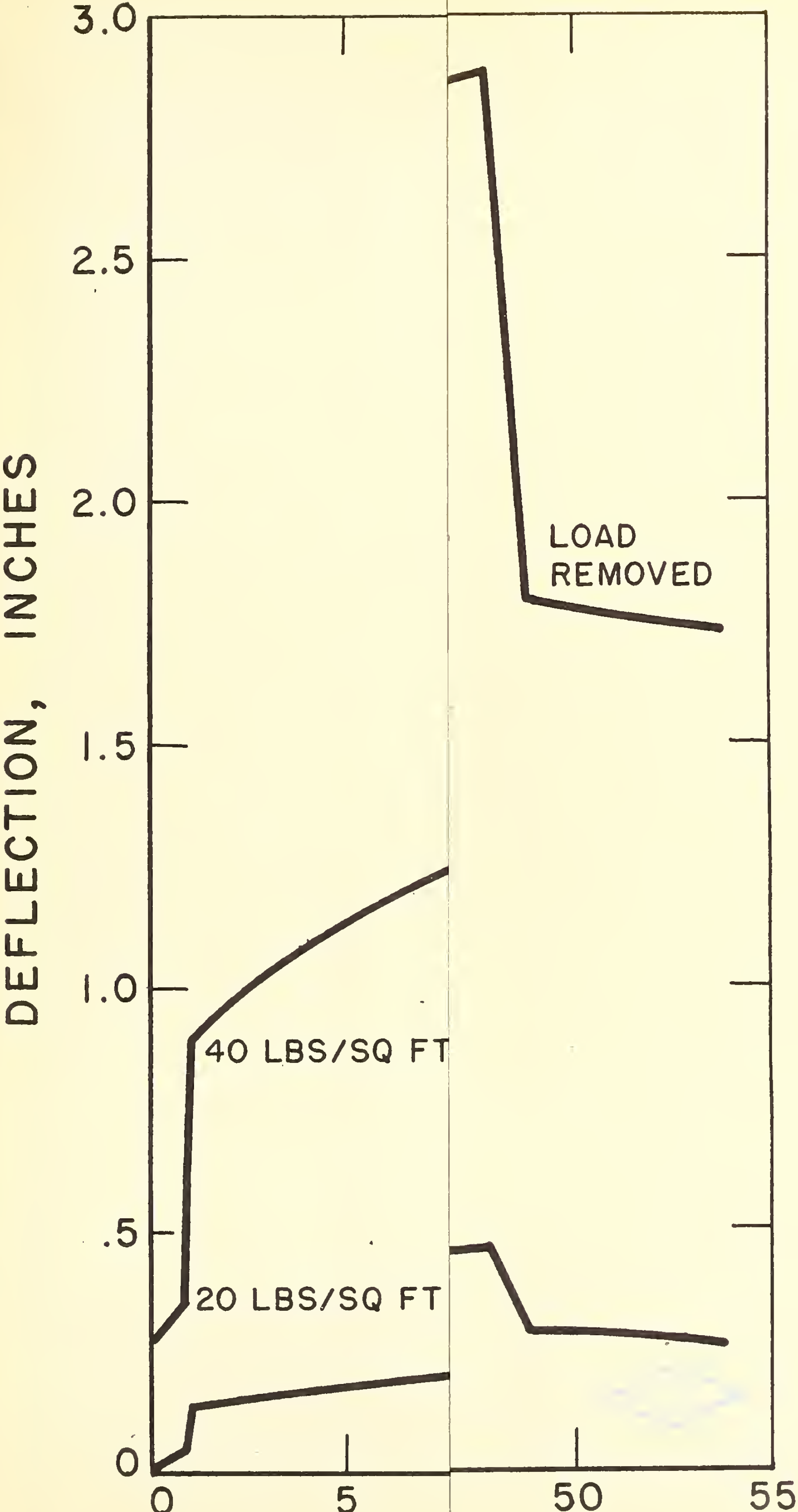

DEFLECTION, INCHES
3.0
2.5
2.0
1.5
1.0
.5
0
0
5
50
55
LOAD
REMOVED
40 LBS/SQ FT
20 LBS/SQ FT

FIG. 6

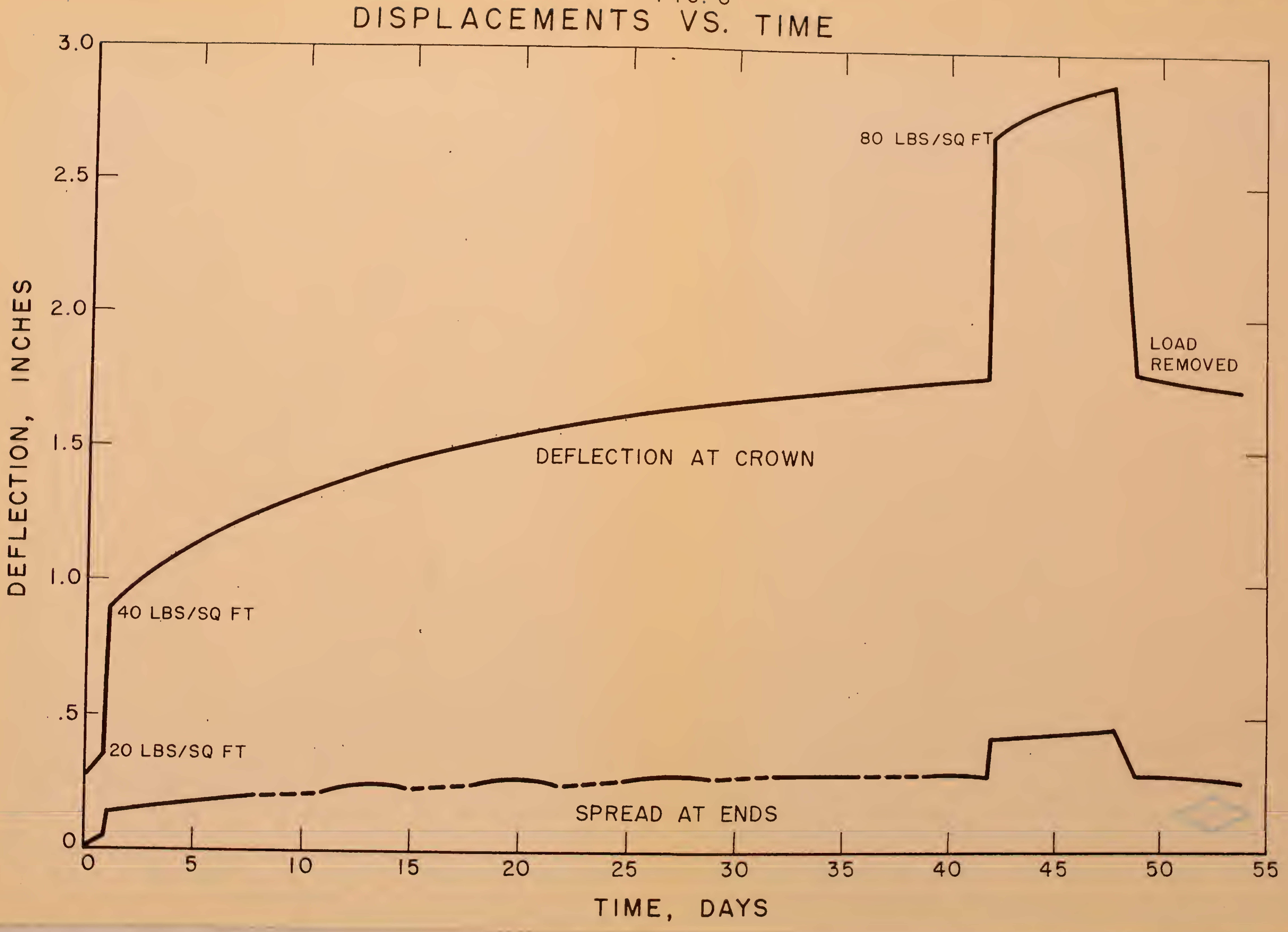
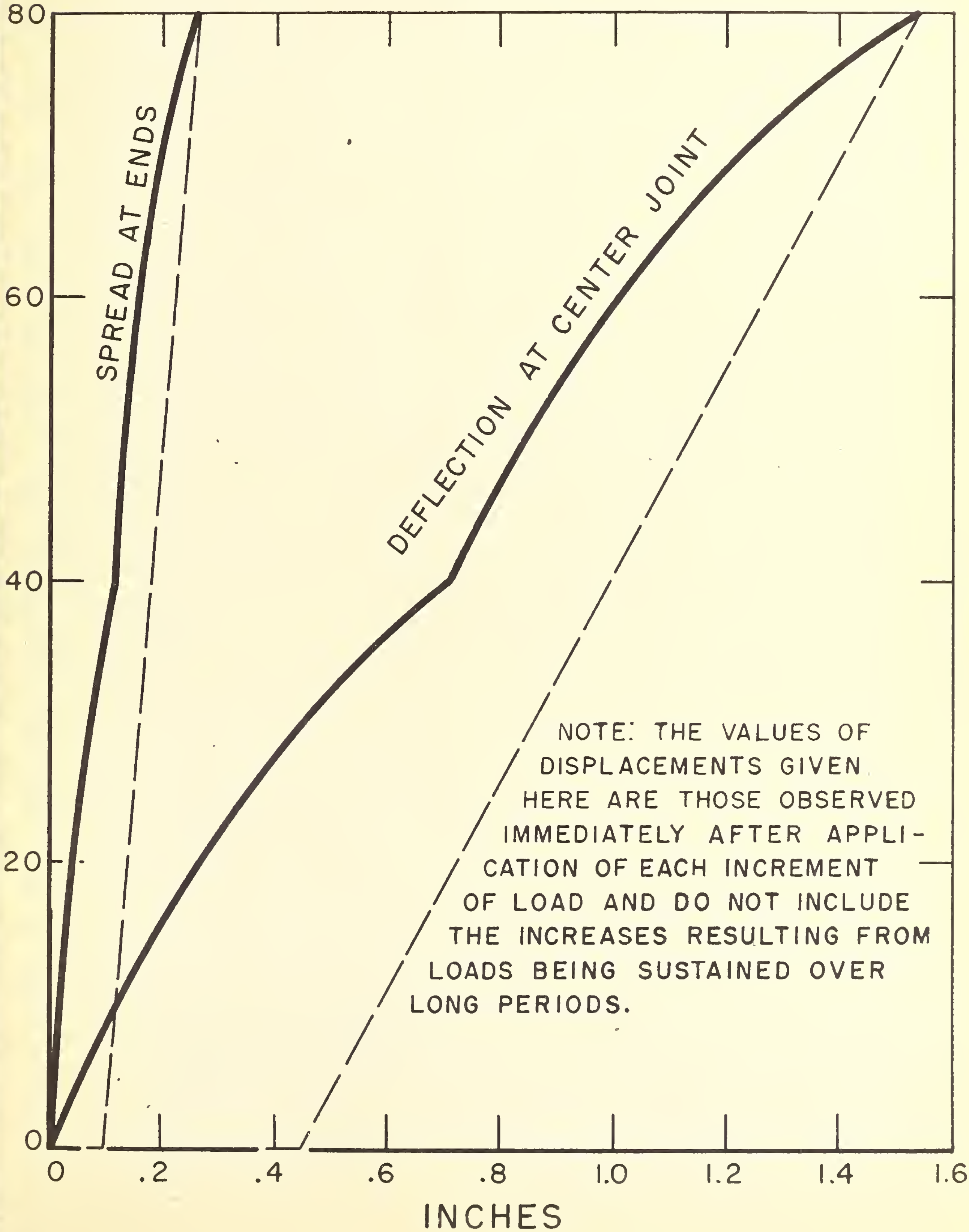
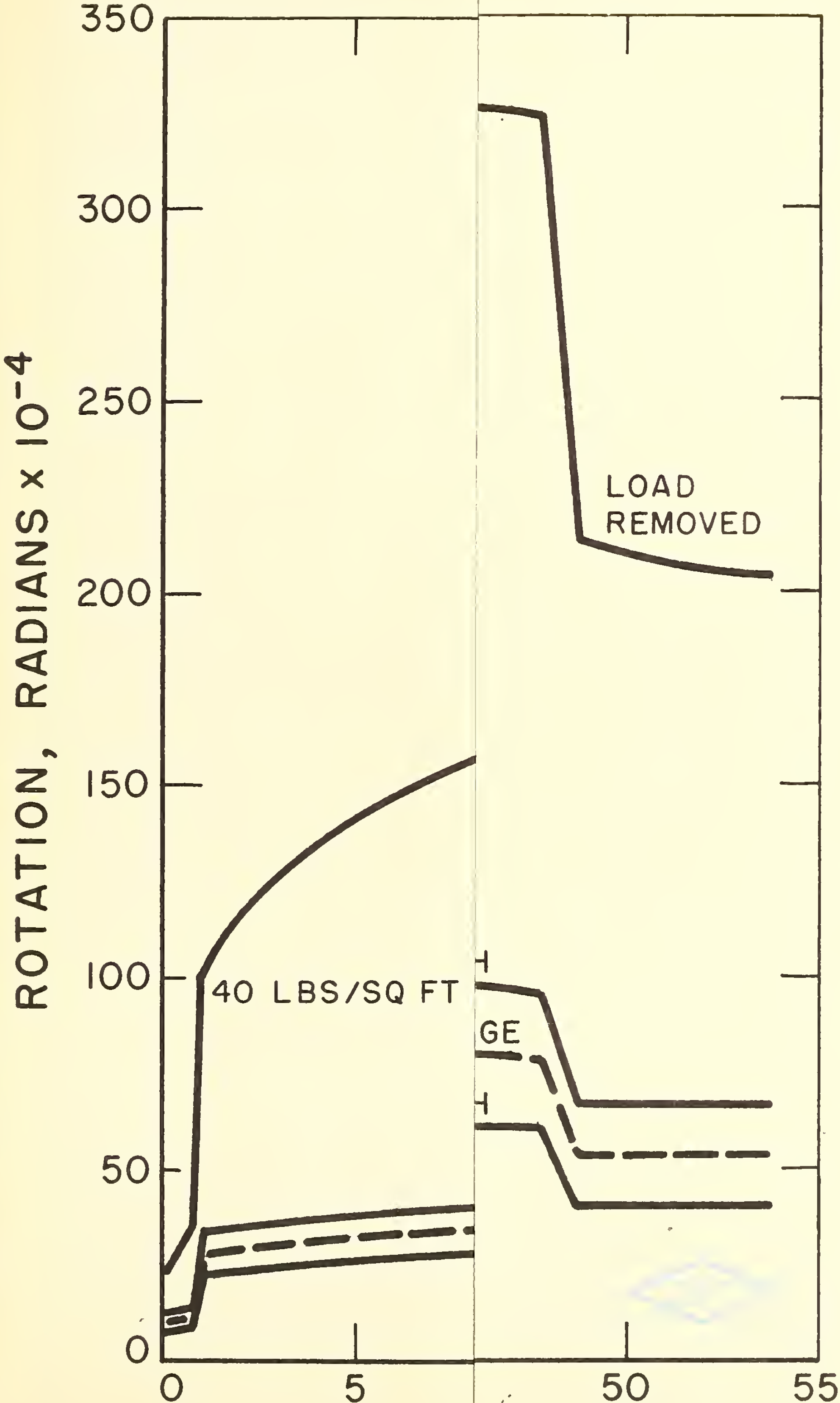

DISPLACEMENTS VS. TIME
DEFLECTION, INCHES
3.0
2.5
2.0
1.5
1.0
.5
0
80 LBS/SQ FT
LOAD
REMOVED
DEFLECTION AT CROWN
40 LBS/SQ FT
20 LBS/SQ FT
SPREAD AT ENDS
0
5
10
15
20
25
30
35
40
45
50
55
TIME, DAYS

FIG. 7

DISPLACEMENTS VS. APPLIED LOAD

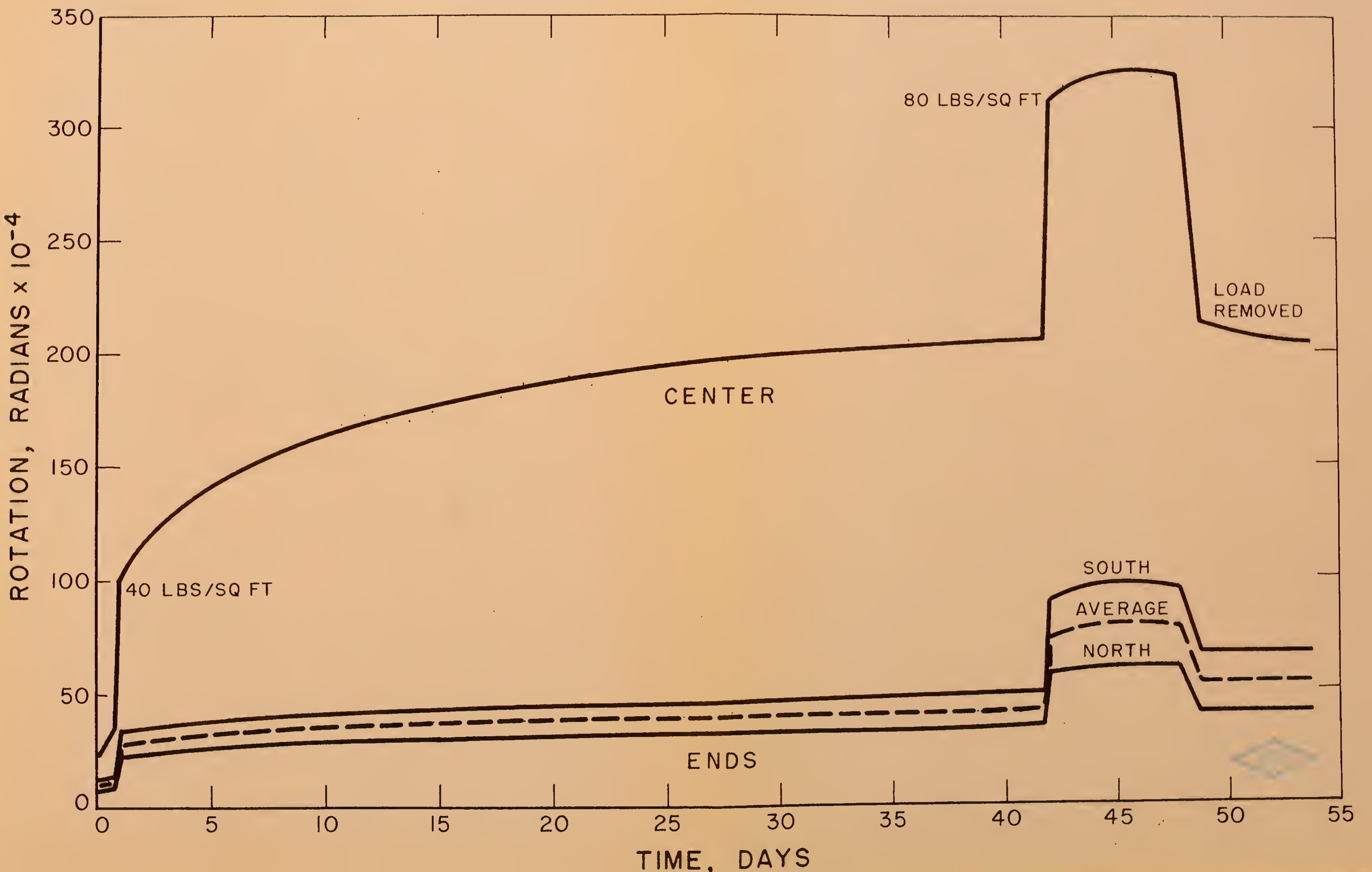


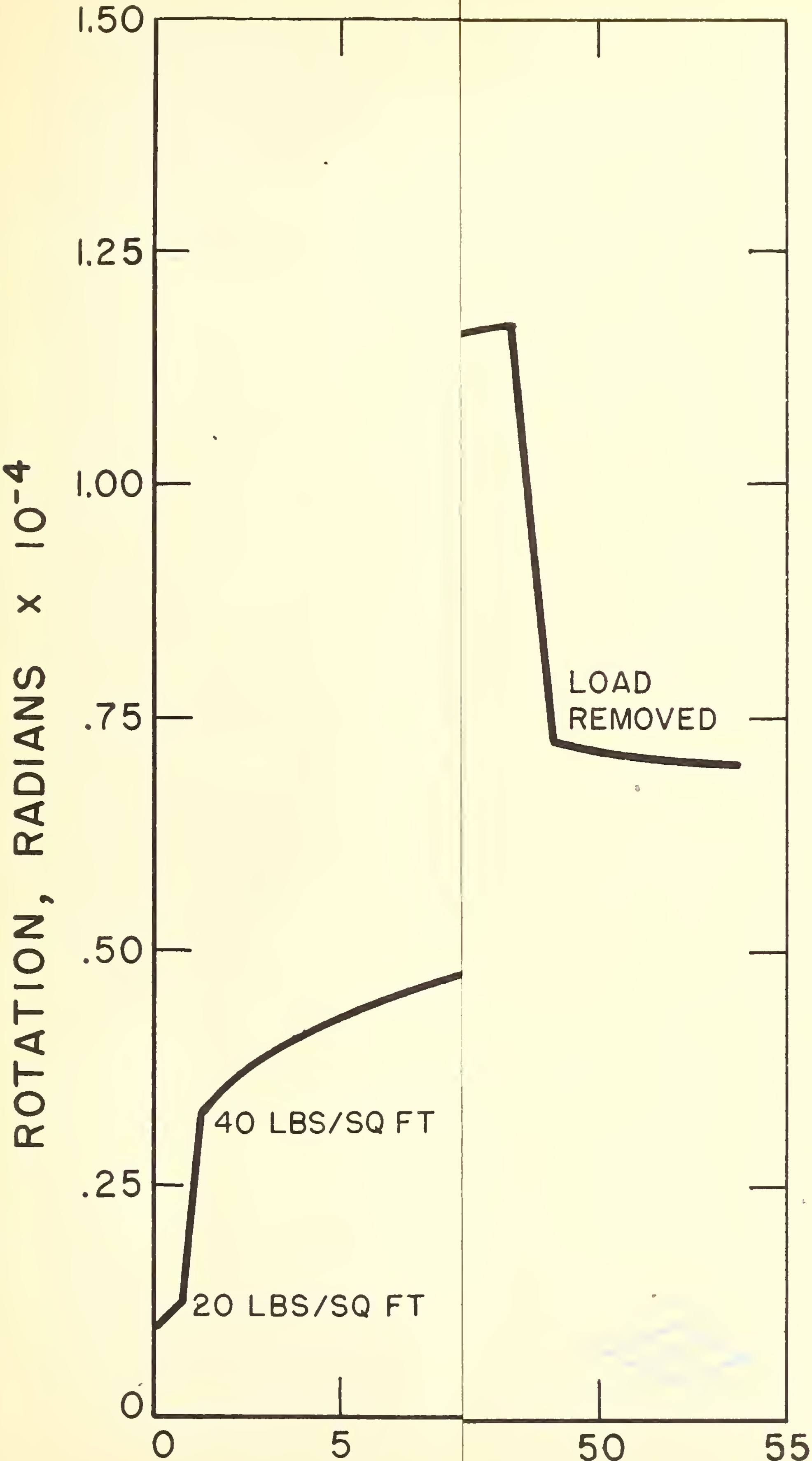


ROTATION, RADIANS x 10^{-4}
350
300
250
200
150
100
50
0
0
5
50
55
LOAD
REMOVED
40 LBS/SQ FT
GE

FIG. 8

ROTATION AT JOINTS VS. TIME

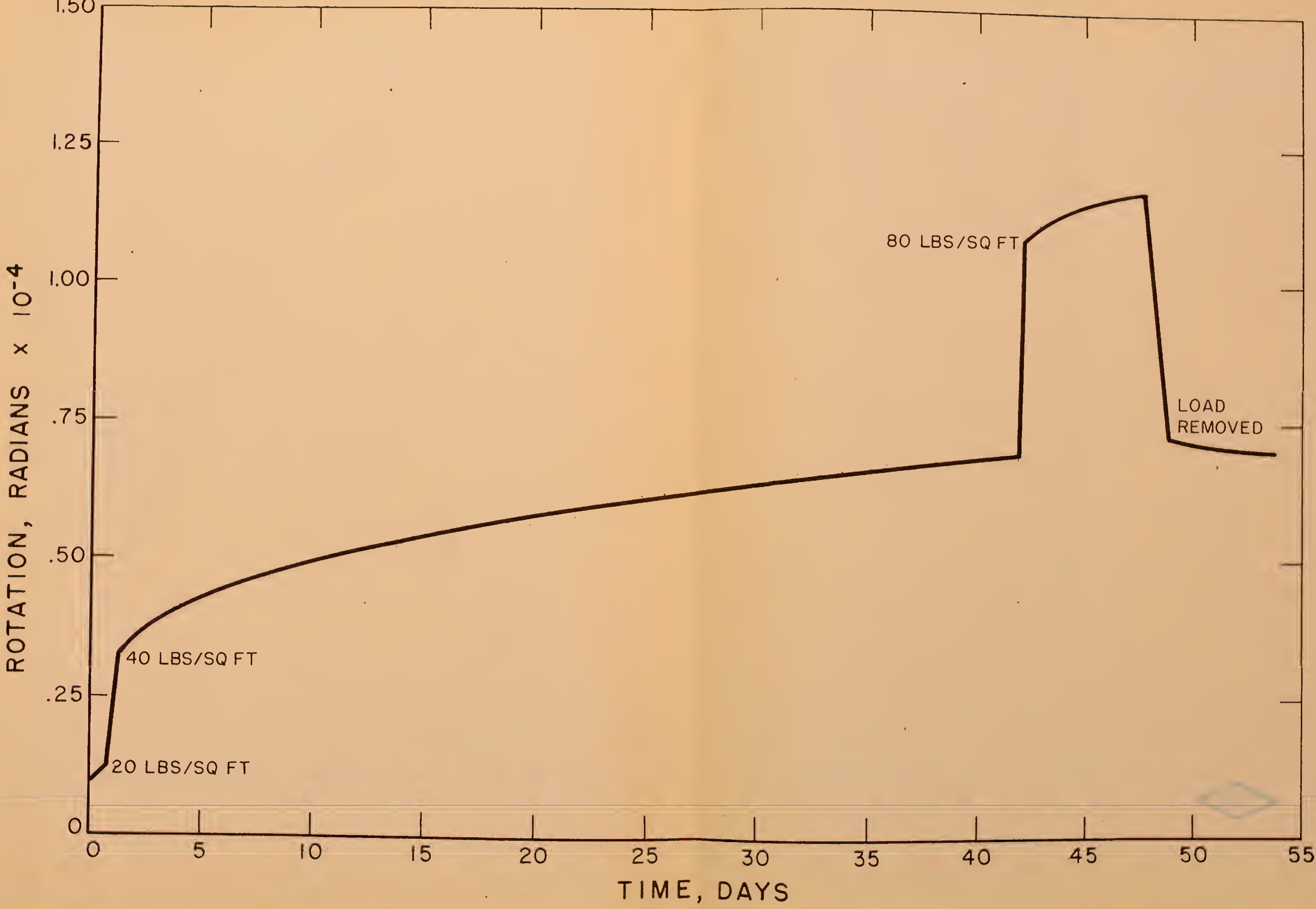




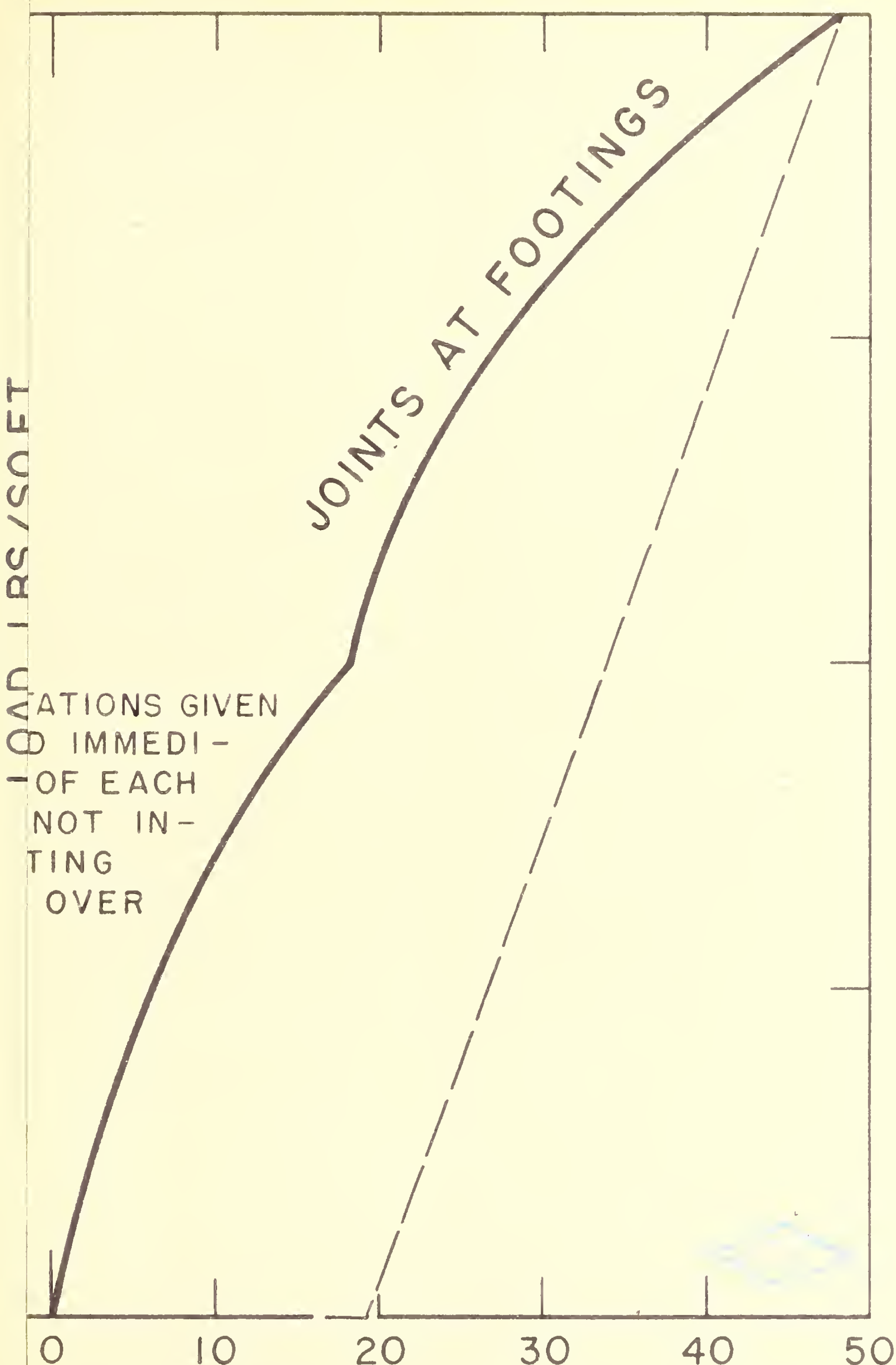
1.50
1.25
1.00
.75
.50
.25
0
ROTATION, RADIANS x 10^{-4}
LOAD
REMOVED
40 LBS/SQ FT
20 LBS/SQ FT
0
5
50
55

FIG. 9

ROTATION AT FOOTINGS VS. TIME



D



JOINTS AT FOOTINGS
ATIONS GIVEN
D IMMEDI-
OF EACH
NOT IN-
TING
OVER
0
10
20
30
40
50

FIG. 16
ROTATION AT JOINTS VS. APPLIED LOAD

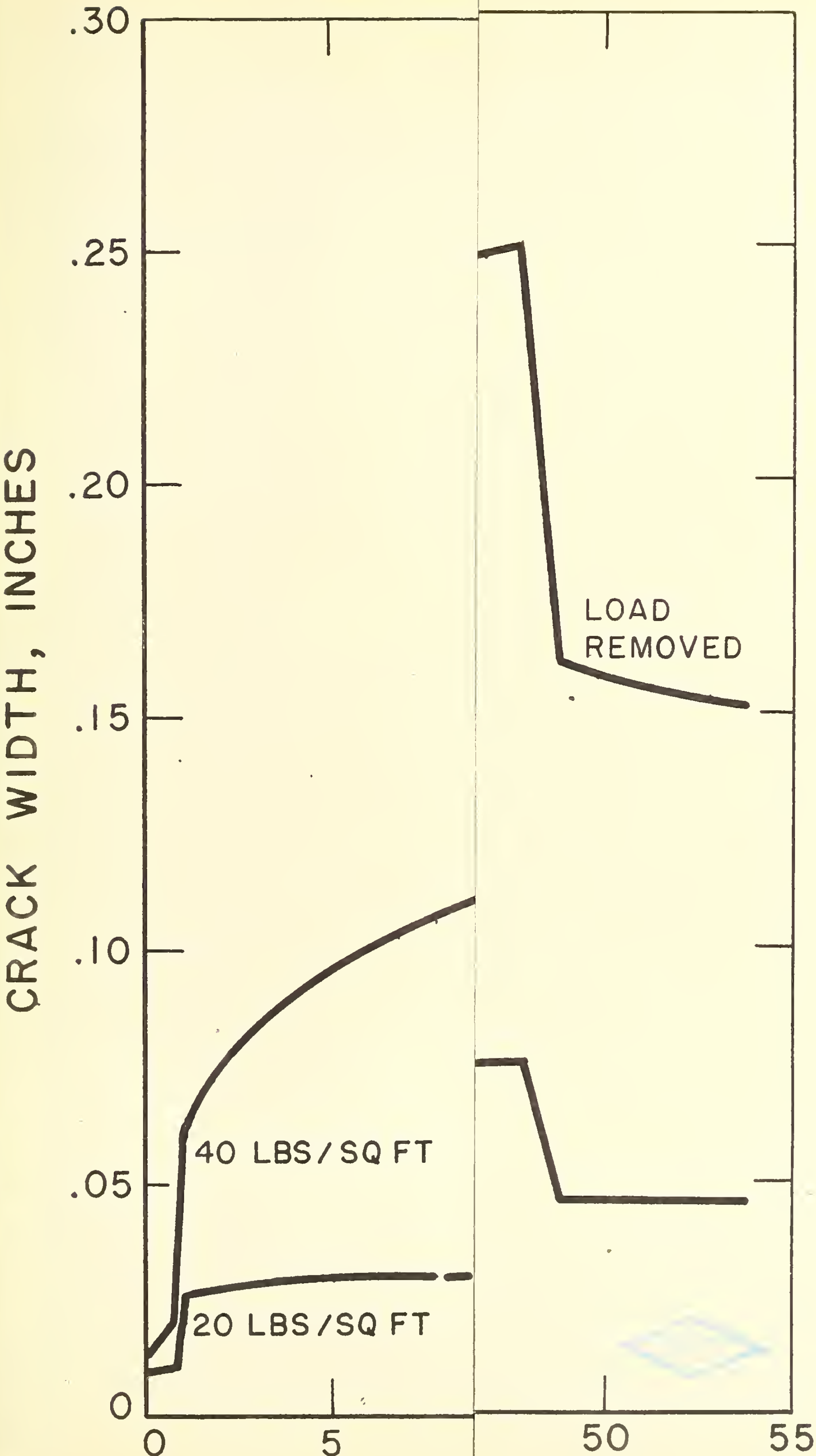

CRACK WIDTH, INCHES
.30
.25
.20
.15
.10
.05
0
0
5
50
55
LOAD
REMOVED
40 LBS/SQ FT
20 LBS/SQ FT

FIG. II

WIDTH OF CRACKS VS. TIME

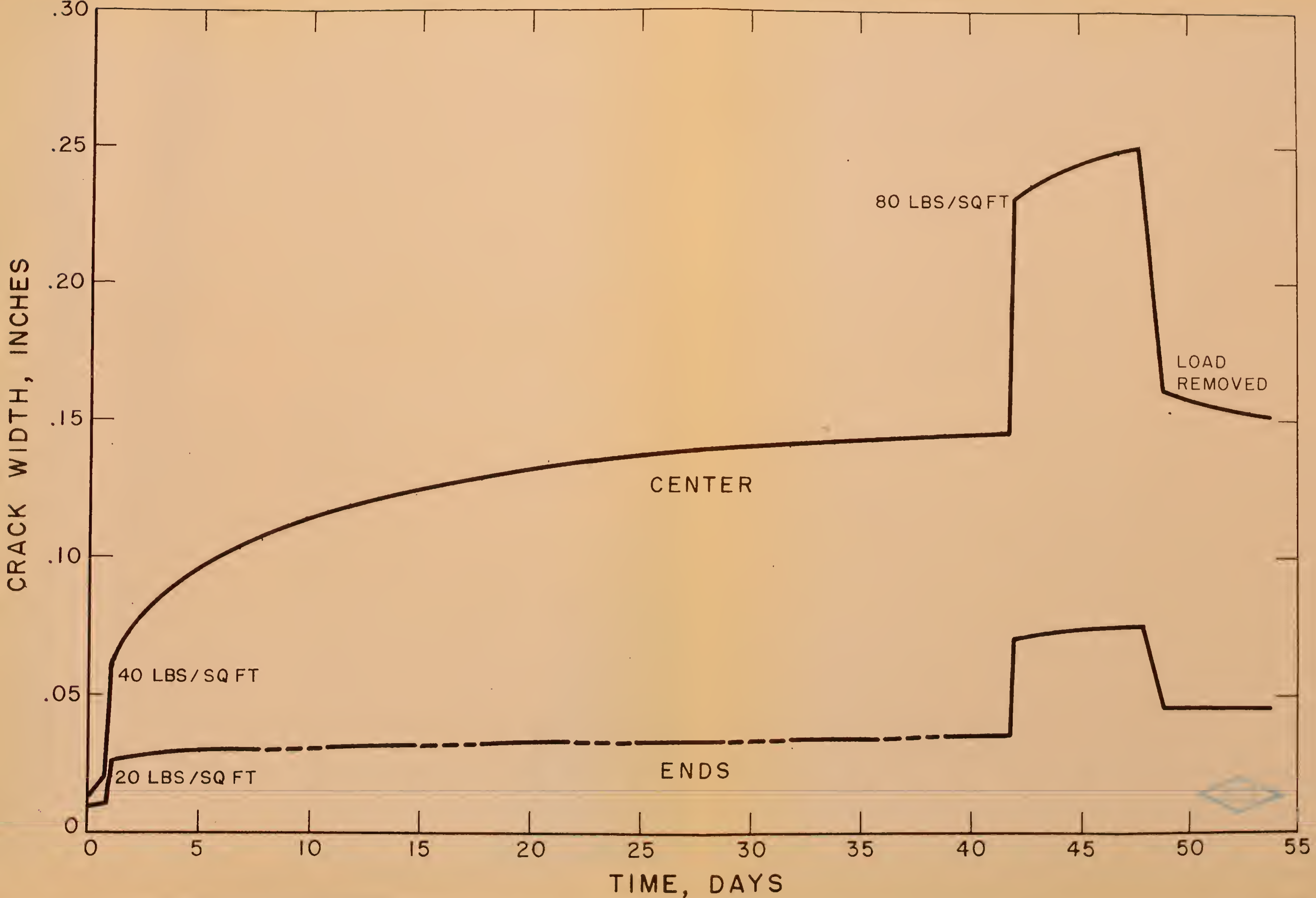


Fig. 12 - Crack pattern, under-side of north roof panel (load = 80 psf)

Fig. 13 - Crack pattern, north wall panel
(load = 80 psf)

Fig. 14 - Cracking at center joint (load = 80 psf)

THE NATIONAL BUREAU OF STANDARDS

Functions and Activities

The functions of the National Bureau of Standards are set forth in the Act of Congress, March 3, 1901, as amended by Congress in Public Law 619, 1950. These include the development and maintenance of the national standards of measurement and the provision of means and methods for making measurements consistent with these standards; the determination of physical constants and properties of materials; the development of methods and instruments for testing materials, devices, and structures; advisory services to Government Agencies on scientific and technical problems; invention and development of devices to serve special needs of the Government; and the development of standard practices, codes, and specifications. The work includes basic and applied research, development, engineering, instrumentation, testing, evaluation, calibration services, and various consultation and information services. A major portion of the Bureau's work is performed for other Government Agencies, particularly the Department of Defense and the Atomic Energy Commission. The scope of activities is suggested by the listing of divisions and sections on the inside of the front cover.

Reports and Publications

The results of the Bureau's work take the form of either actual equipment and devices or published papers and reports. Reports are issued to the sponsoring agency of a particular project or program. Published papers appear either in the Bureau's own series of publications or in the journals of professional and scientific societies. The Bureau itself publishes three monthly periodicals, available from the Government Printing Office: The Journal of Research, which presents complete papers reporting technical investigations; the Technical News Bulletin, which presents summary and preliminary reports on work in progress; and Basic Radio Propagation Predictions, which provides data for determining the best frequencies to use for radio communications throughout the world. There are also five series of nonperiodical publications: The Applied Mathematics Series, Circulars, Handbooks, Building Materials and Structures Reports, and Miscellaneous Publications.

Information on the Bureau's publications can be found in NBS Circular 460, Publications of the National Bureau of Standards ($1.25) and its Supplement ($0.75), available from the Superintendent of Documents, Government Printing Office. Inquiries regarding the Bureau's reports and publications should be addressed to the Office of Scientific Publications, National Bureau of Standards, Washington 25, D. C.

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