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NATIONAL BUREAU OF STANDARDS REPORT

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BEHAVIOR OF THIN-SHELL CONCRETE ROOF PANELS UNDER SUSTAINED LOADS

By

L. F. Skoda and A. F. Kirstein

Report to

Bureau of Yards and Docks
Department of the Navy

U. S. DEPARTMENT OF COMMERCE
NATIONAL BUREAU OF STANDARDS

THE NATIONAL BUREAU OF STANDARDS

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IMPORTANT NOTICE

NBS

U. S. DEPARTMENT OF COMMERCE

NATIONAL BUREAU OF STANDARDS

BEHAVIOR OF THIN-SHELL CONCRETE ROOF PANELS UNDER SUSTAINED LOADS

By

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Sustained load tests were performed on four thin-shell concrete roof panels to obtain information on their general behavior and deflection with time. Periodic time-deflection observations yielded data which indicated that the technique of casting initial camber into panels of this type is a satisfactory method of eliminating the deflection due to dead load. Comparisons of the time-deflection relationships of the panels investigated also indicated that the history of and transportation of these panels may cause initial differential displacements which in turn may cause distress in the calked joints necessary in this type of construction.

1. INTRODUCTION

The object of these tests was to observe time-deflection relationships of thin-shelled precast concrete roof panels subjected to various loading conditions. A major factor to be considered in the design of flat roof panels is the elimination of concave surfaces or pockets which would allow water to accumulate and possibly seep through the joints. Since thin-shelled concrete panels deflect under loads, the problem of the designer is to compensate for this deflection. One solution of this problem is to design panels having initial positive camber so that expected loads cannot produce these concave surfaces.

At the request of the Bureau of Yards and Docks, a cambered and a flat roof panel were cast and tested at the Structural Engineering Laboratories of the National Bureau of Standards for the purpose of studying this problem. In addition to these panels the report describes the behavior over an extended period of a previously cast prestressed and and a non-prestressed panel.

2. DESCRIPTION OF TEST SPECIMENS

Two types of roof panels designed by the Bureau of Yards and Docks were constructed for this test. Type A was a prestressed panel and Type C was a non-prestressed panel. The first panel to be put under test was a Type A panel. The succeeding panels Nos. 2, 3, and 4 were of the Type C design.

2.1 Description of prestressed panel

The prestressed panel was channel shaped in cross section with overall dimensions of 5 ft by 24 ft by 8 in. with a web thickness of one inch. The panel had transverse ribs at a distance of 9 in. from either end.

Two ten-wire Freyssinet prestressing assemblies were used to apply a prestressing force of 30,300 lb on each longitudinal rib. Each assembly consisted of two end anchorage cylinders, two end anchorage cones, one flexible steel conduit, and ten 0.196 in. diameter high yield point steel wires. In order to accurately determine the amount of prestress applied to each longitudinal rib, a dynamometer was placed between the anchorage cylinder and concrete at one end of each assembly.

In addition to the prestressing steel, 2- by 2-in. by 12/12 gage welded wire fabric was placed at the midplane of the cross section of the panel.

A detailed description of the reinforcement, casting, and prestressing of a duplicate specimen which was tested to destruction under uniform loading was presented in NBS Report No. 2295 entitled, "Test of Precast Prestressed Roof Panel No. 1." Report No. 2295 included the physical properties of the prestressing steel as well as photographs of the dynamometers and tensioning devices used in the investigation presented herein.

2.2 Description of reinforced precast panels

The non-prestressed precast roof panels Nos. 2, 3, and 4, were also essentially channel shaped in cross section with overall dimensions of 5 ft by 24 ft by 10 in. Each panel had six transverse ribs. Two of the ribs were located 9 in. from each end and the four intermediate ribs were equally spaced between them 4 ft 6 in. apart.

The major tensile reinforcement was located near the bottom of each longitudinal rib and consisted of a single No. 7 deformed reinforcing bar. All transverse ribs were reinforced with a pair of No. 3 deformed reinforcing bars located near the top and bottom of each rib. Welded wire fabric 2- by 2-in. by 12/12 gage was placed at the midplane of the cross section of the panels.

Panel No. 4 was similar to panels Nos. 2 and 3 in all respects except that a positive camber was cast into this panel such that the middle of the 24 ft length was 0.75 in. higher than the ends.

The deformed reinforcing bars used in all panels were of intermediate grade steel which met the requirements of ASTM A15-54T for billet steel bars and the deformations of these bars complied with the requirements of ASTM A305-53T.

2.3 Concrete mix

Type III cement, sand, and rounded pea gravel were proportioned 1:2.48:2.02, by weight, with a water-cement ratio of 0.53. The average 7 day compressive strength of 6- by 12-in. control cylinders was 6800 psi for this concrete mix.

3. TEST SETUP

3.1 Loading conditions

After each panel was cast and moist cured, it was transferred from the laboratory to the outdoor test site. The first two panels were transferred to the test site in the months of March and May 1953. The succeeding panels were moved to the test site in September 1954 and February 1955. All four panels were simply supported on a span of 22 ft 4 in.

A uniformly distributed live load of 45 and 20 psf was applied to panels Nos. 1 and 2, respectively. Panel No. 1 was loaded on the day following its transfer to the test site. Panel No. 2 was placed under test supporting its own weight for approximately one month before the required live load of 20 psf was applied to its top surface. In both cases the live load was applied with 1/4 in. pea gravel carefully screeded to the calculated height that would produce the required loads. Side boards were placed about the periphery of the panels in order to retain the gravel.

The construction of an addition to the building adjoining the test site necessitated moving the panels to a new location. In April 1954 both panels were unloaded and moved to their present location. The panels had been subjected to their particular live loads for approximately one year prior to the unloading and moving. After the panels were unloaded and moved to the new test site they remained unloaded for the duration of the test.

Panels Nos. 3 and 4 were only tested under dead load conditions. They were transferred to the test area directly from the curing area and observed periodically to determine their deflections.

3.2 Instrumentation

Since deflection measurements were to be made over a period of years, the measuring technique had to be independent of temperature and movement of the supporting piers. A taut wire system that enabled measurements to be taken with an accuracy of about 0.02 in. was decided upon.

The taut wire system consisted of an 0.011 in. diameter stainless steel wire and a stainless steel scale graduated in hundredths of an inch. The wire was draped over pivot points that were attached to the panel just over the panel supports. Weights were attached to the ends of the wire so that a fairly constant tension would be maintained. The stainless steel scale was attached to the edge beams of the panels and it was intersected by the wire at mid-span. The stainless steel scale and the pivot points were firmly anchored to the panel with lead expansion shields. This system was applied to each longitudinal rib of every panel.

Readings were also made with dynamometers that had been used to determine the applied prestressing forces on panel No. 1. These readings were taken for about 80 days; after this period, the resistance to ground of the gage elements was such that further readings were deemed unreliable.

4. TEST RESULTS

4.1 Load and time-deflection relationships

Figure 1 shows the variation of positive camber with time and different loading conditions in panel No. 1. Initial measurements were made prior to the application of prestress. The amount of immediate positive camber as a result of prestressing was 0.84 in. as indicated by point A of the legend. The upward movement continued to increase and was 1.10 in. at point B just prior to the transfer of the panel to the test site. As a result of lifting and moving the panel to the test site an immediate increase in camber of 0.12 in. occurred. The total positive camber after moving was then 1.22 in. as shown at point C. This upward movement continued for 24 hr and became 1.34 in. just prior to loading (point D). The reduction of positive camber due to the application of the 45 psf load was 0.60 in., leaving a net camber of 0.74 in. shown by point E.

After being subjected to the live load for approximately 400 days, the panel was unloaded and moved to the new test site. The camber just prior to unloading was 0.71 in. After unloading and moving, the total upward displacement was 1.3 in. (point G).

After about two months of a rather steady increase in upward displacement, the camber reached a value of 1.88 in. From this time on the camber increased more slowly and fluctuated between maximum and minimum values of 2.03 and 1.65 in., respectively. These fluctuations were apparently due to the temperature and moisture content of the panel.

Figure 2 shows the time vs. deflection relationship for panel No.2. The deflection, in inches, is the downward movement of the panel at mid-span. An immediate deflection of 0.14 in. resulted when the panel was moved from the casting table to the test site (point A). Prior to loading, the panel carried its own weight for approximately 30 days. The deflection just prior to application of a superimposed load of 20 psf was 0.36 in. (point B) and was 0.60 in. just after loading (point C). After a recovery of 0.20 in. which took about three weeks, the deflection progressed at a fairly linear rate for a period of about a year. The deflection (point D) was 0.56 in. just before the panel was unloaded. Immediately after the panel was unloaded the deflection was 0.45 in. (point E). The panel remained unloaded at the old site for two days before it was moved. During this time additional recovery occurred and is indicated by point F at 0.40 in.

The deflection was measured just after the panel was moved and no change was observed. From this point on the deflection increased slightly and fluctuated between maximum and minumum values of 0.69 and 0.43 in., respectively.

Figure 3 illustrates the time-deflection relationships for panels Nos. 3 and 4. These two panels were never subjected to live loads and the deflections were measured with respect to the original profiles of the panels as cast.

The deflection of panel No. 3 as it was placed on its outdoor piers was 0.58 in. Within the first ten days a recovery of 0.13 in. occurred. The average total deflection for the next 240 days was approximately 0.47 in. After 300 days the deflection increased slightly and fluctuated between maximum and minimum values of 0.72 and 0.56 in.

Panel No. 4 was cast with an initial camber of 0.75 in. and figure 3 illustrates the downward movement at mid-span. As the panel was placed under test a deflection of 0.28 in. was observed. For the next 160 days the rate of increase of deflection was rather rapid and the deflection reached 0.53 in. From this point on the deflection increased slightly and fluctuated between maximum and minimum values of 0.65 and 0.50 in. The curve shows that a small amount of camber is still present in the panel.

4.2 Variation of prestressing force

Figure 4 presents the variation of the amount of prestress in Panel No. 1 with load and time. The initial prestress was taken as 100 percent and the periodic dynamometer measurements indicated the variation of the prestressing force. Observations were discontinued after 80 days due to the low resistance to ground of the sensing elements of the dynamometers.

5. DISCUSSION

It is obvious from the increase in camber with time shown in figure 1, that the prestress applied to Panel No. 1 was in excess of that required to eliminate dead load deflections. Further observations indicate that the prestress was great enough to maintain a camber of about 0.7 in. for about 400 days while the panel was subjected to a uniform live load of 45 psf.

The panels required different lengths of time to reach a fairly constant rate of deflection. This was due to the fact that the loading conditions varied in each case and to the fact that the initial deflections of panels Nos. 2, 3, and 4, were different. These three panels were identical in design, but when they were placed on their respective outdoor piers a maximum difference in initial deflection of 0.44 in. was observed between panels Nos. 2 and 3. This large difference in initial deflection can be attributed to the manner in which each panel was handled and transported to the test site. However, the important point to observe is that all three panels reached equilibrium at approximately 0.6 in. of deflection regardless of their previous history of handling, transportation, and loading condition. Nevertheless, careful consideration should be given to the methods used in handling and transporting panels of this kind as they are easily damaged.

A comparative study of the time-deflection relationships of the various panels reveals that the deflection characteristics of all panels are quite similar after the initial deflections due to handling and loading. The variation in deflection after the uniform rate was reached can be attributed to temperature and moisture conditions. However, the magnitude of the changes due to the variations of temperature and moisture were not expected to be as great as they were. It is believed that in future tests of this kind it will be necessary either to make careful observations of the weather conditions at the site, or have a duplicate control specimen tested concurrently in the laboratory under controlled conditions.

The 0.75-in. camber that was cast into panel No. 4 adequately compensated for the dead load deflection, as the equilibrium deflection reached by this panel was approximately 0.65 in. This resulted in a mid-span elevation of 0.10 in. which is not excessive considering the span length of 22 ft 4 in.

When considering a structure composed of panels of the type investigated herein, it is also necessary to give consideration to the jointing material required to weatherproof the structure. It is evident that any deflection or movement of one panel with respect to the other will cause serious distress in the calked joints between them. Therefore, precision casting is essential in the production of these panels.

6. CONCLUSIONS

1. Caution must be exercised when handling and transporting thin-shell prestressed and non-prestressed panels as they can be damaged through careless or improper handling.

2. The effect of temperature and moisture on the panels was greater than anticipated indicating a need for careful observations of the weather conditions at the test site.

3. The technique of casting initial camber into a non-prestressed panel appeared to be a very satisfactory method of eliminating the deflection due to dead load.

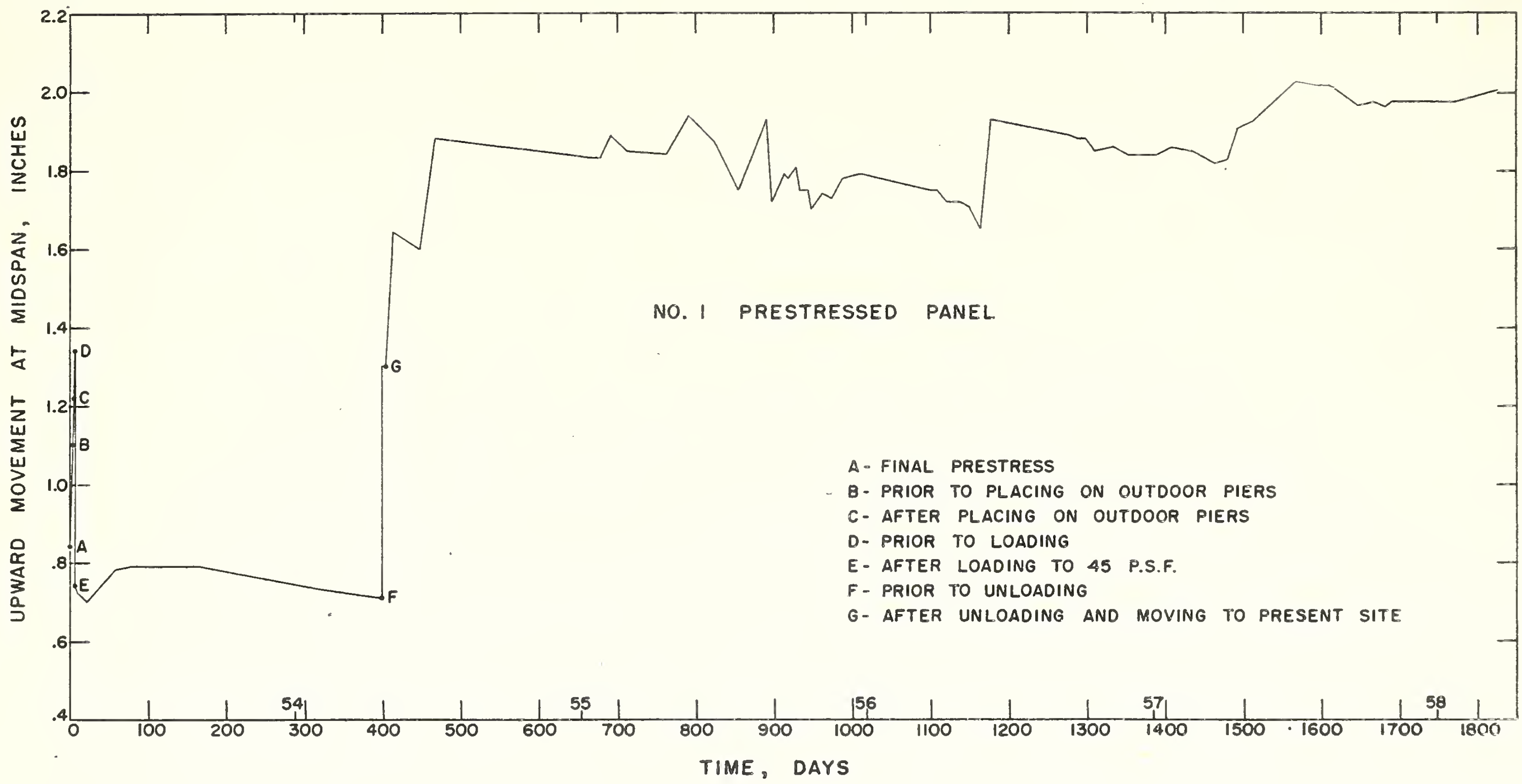

FIG. I VARIATION OF UPWARD CAMBER IN PRESTRESSED PANEL WITH TIME AND DIFFERENT CONDITIONS OF LOADING. DEFLECTIONS WERE MEASURED WITH RESPECT TO THE ORIGINAL PROFILE OF PANEL AS CAST.

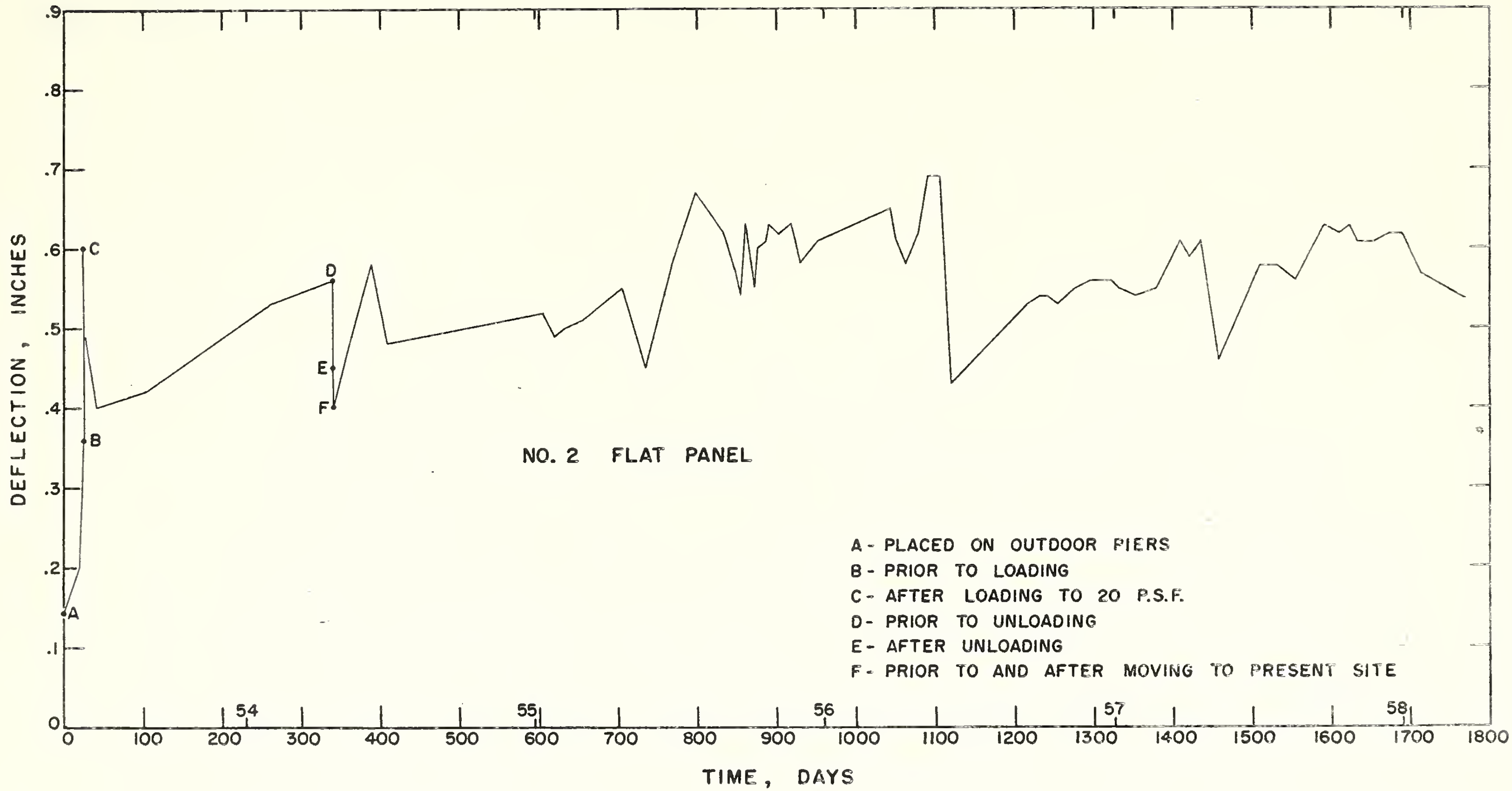

FIG. 2 VARIATION IN DEFLECTION AT MIDSPAN OF FLAT PANEL NO. 2 WITH DIFFERENT CONDITIONS OF LOADING. DEFLECTIONS WERE MEASURED WITH RESPECT TO THE ORIGINAL PROFILE OF PANEL AS CAST.

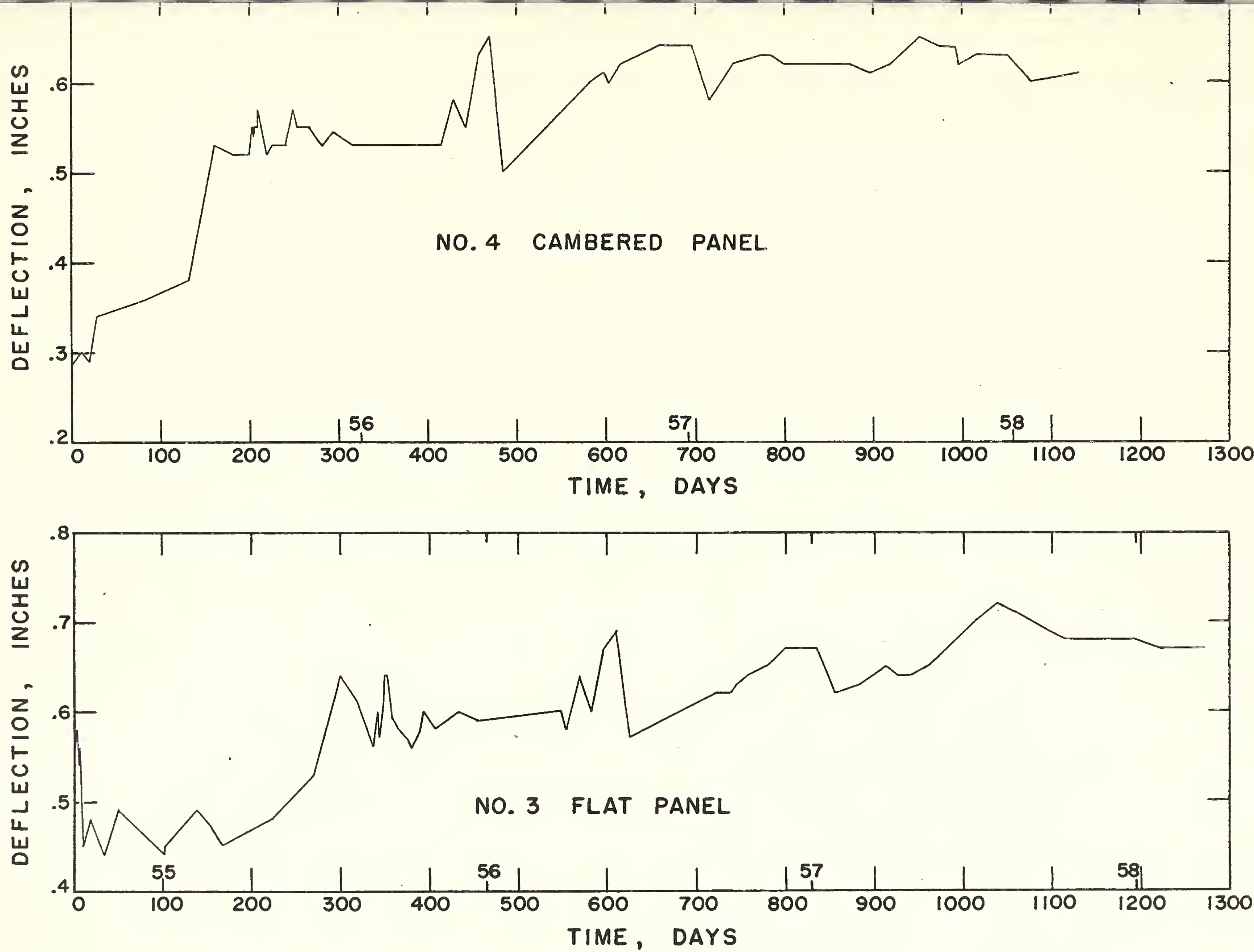

FIG. 3 DEFLECTION AT MIDSPAN OF CAMBERED AND FLAT ROOF PANELS. DEFLECTIONS WERE MEASURED WITH RESPECT TO THE ORIGINAL PROFILE OF PANELS AS CAST.

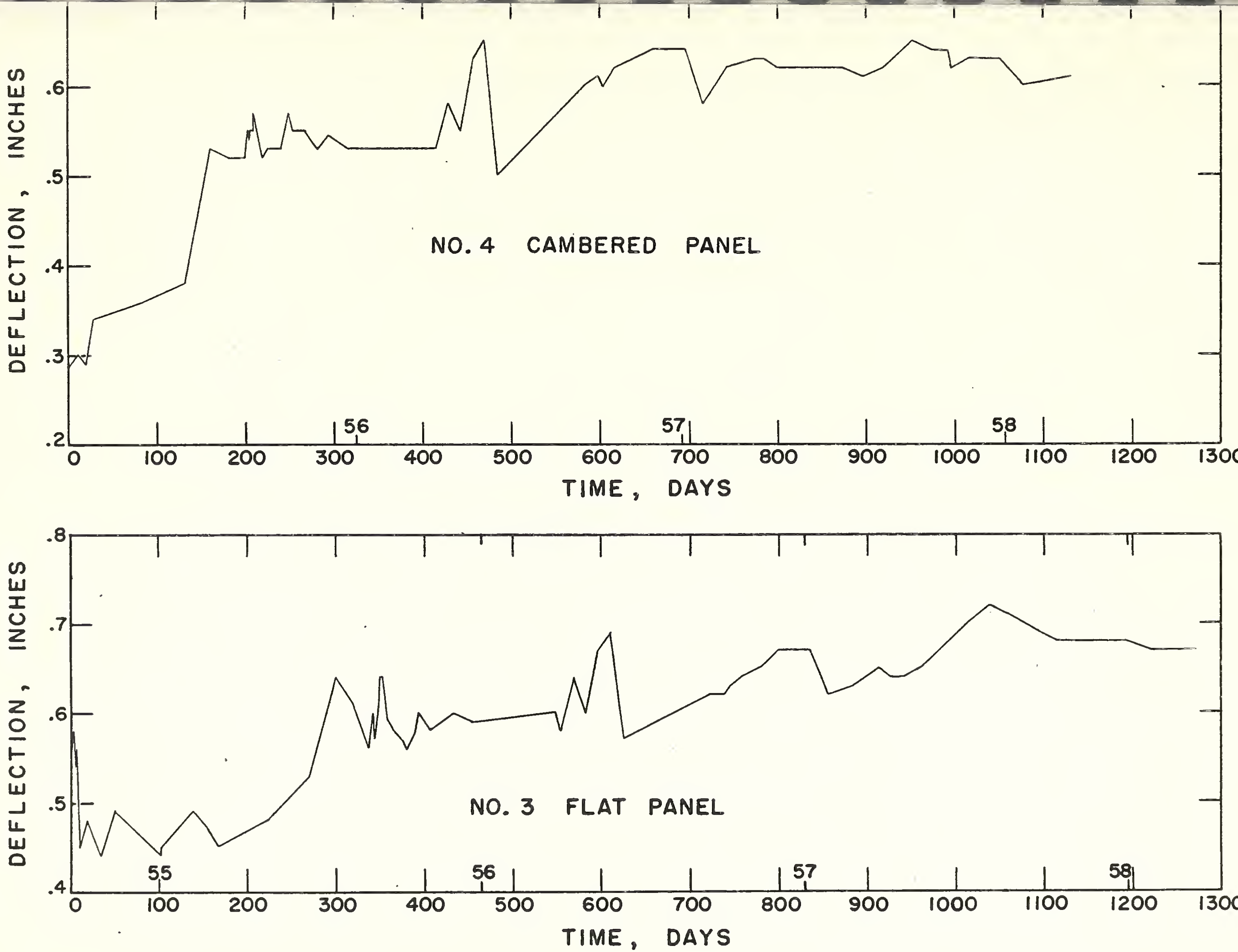

FIG. 3 DEFLECTION AT MIDSPAN OF CAMBERED AND FLAT ROOF PANELS. DEFLECTIONS WERE MEASURED WITH RESPECT TO THE ORIGINAL PROFILE OF PANELS AS CAST.

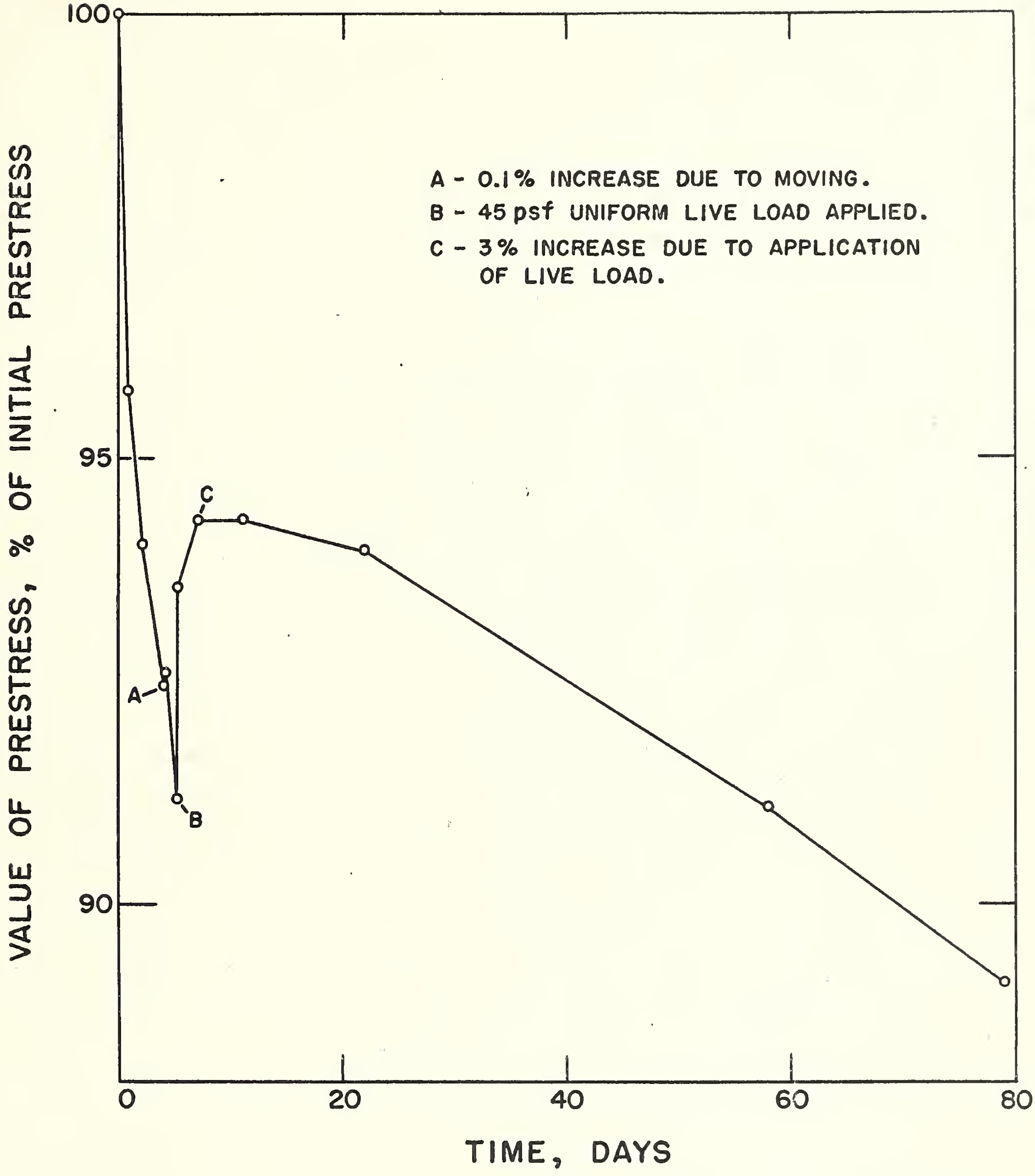

FIG. 4 VARIATION OF PRESTRESS WITH LOAD AND TIME.

U. S. DEPARTMENT OF COMMERCE
Sinclair Weeks, *Secretary*

NATIONAL BUREAU OF STANDARDS
A. V. Astin, *Director*

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