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CYCLIC LOAD TESTS OF COMPOSITE PRESTRESSED-REINFORCED CONCRETE PANELS

in cooperation with the
Department of Transportation
Federal Highway Administration

RESEARCH REPORT 145-4F

STUDY 2-5-70-145

PRESTRESSED CONCRETE

CYCLIC LOAD TESTS

OF

COMPOSITE PRESTRESSED-REINFORCED CONCRETE PANELS

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Research Report No. 145-4F

A Study Prestressed Panels and Composite Action in Concrete Bridges Made of Prestressed Beams, Prestressed Sub-deck Panels, and Cast-in-place Deck

Research Study No. 2-5-70-145

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16. Abstract

Static and cyclic load tests made on seven composite panels of prestressed concrete and cast-in-place concrete are described. The Prestressed panels are used as stay-in-place forms for concrete bridge decks. Reinforced concrete bonds to the top surface of the prestresse panels. The unit acts compositely under traffic loads.
Some of the test panels contained an interlocking shear lug which was cast in the prestressed panel and was engaged by the cast-in-place concrete. Others had no such lugs. Panels with the shear lugs showed a slight advantage in stiffness over the others at high loads. Panels with shear lugs were cycled under 210 percent of design load for 11.9 million load cycles without failure. Panels without shear lugs reached failure deflection, 1/4 in. at 2.25 million cycles under 210 percent of design load.

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Bridge deck, composite concrete, cyclic load, endurance limit, load test, prestressed concrete, stay-in-place forms	

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FOREWORD

This is the final report on research performed under Research Project No. 2-5-70-145. Three previous reports covering different phases of the research have been published. Those reports are:

"Study of In-Service Bridges Constructed with Prestressed Panel Sub-Decks", Harry L. Jones and Howard L. Furr, TTI Report 145-1, July 1970.

"Development Length of Strands in Prestressed Panel Sub-Decks", Harry L. Jones and Howard L. Furr, TTI Report 145-2, December 1970.

"Evaluation of a Prestressed Panel, Cast-in-Place Concrete Bridge", Eugene Buth, Howard L. Furr, Harry L. Jones, and A. A. Toprac, a joint report of CFHR and TTI, TTI Report No. 145-3, September 1972.

The major work was reported in Report 145-3 in which the results of tests on a full scale bridge model were presented. Two million applications of a simulated heavy axle load of the AASHO H-20 truck were applied in each of three locations on the bridge. In another loading, two million cycles of load were alternately applied on opposite sides of a panel to simulate a wheel rolling across the joint. Finally, static failure loading was applied to the deck. No failure of any kind occurred during the cyclic loading; performance was in accordance with the design. The static load cracked the deck at 3.8 times the design wheel load, and punch through occurred at 12.5 times design wheel load.

The field evaluation of three 10-year old bridges revealed transverse cracks in the top side of the deck over a large portion of the butt joints in the prestressed sub-panels. Those bridges were performing well under heavy traffic, and they showed no indications of failure.

The tests and evaluations showed the composite bridge-deck made of prestressed panels and cast-in-place concrete, as designed by the Texas Highway Department, to be sound.

ABSTRACT

Static and cyclic load tests made on seven composite panels of prestressed concrete and cast-in-place concrete are described. The prestressed panels are used as stay-in-place forms for concrete bridge decks. Reinforced concrete bonds to the top surface of the prestressed panels and the unit acts compositely under traffic loads.

Some of the test panels contained an interlocking shear lug which was cast in the prestressed panel and was engaged by the cast-in-place concrete. Others had no such lugs. Panels with the shear lugs showed a slight advantage in stiffness over the others at high loads. One panel with shear lugs was cycled under 210 percent of design load for 11.9 million load cycles without failure. One panel without shear lugs reached failure deflection of 1/4 in. at 2.25 million cycles under 210 percent of design load.

Key Words:

Bridge deck, composite concrete, cyclic load, endurance limit, load test, prestressed concrete, stay-in-place forms.

SUMMARY

The use of stay-in-place prestressed concrete panels as forms for an upper layer of bridge deck concrete has raised questions about bonding of the cast-in-place concrete to the prestressed stay-in-place elements. Static and cyclic load tests were made on seven of such composite panels to determine how they would fail under repeated loads.

Prestressed panels, 3 1/2 x 22 x 92 in., were prestressed with 7-wire, 270 ksi strands to produce 835 psi prestress after 20 percent loss. Interlocking shear lugs made of number 4 reinforcing bar bent to a curved Z shape were cast in three of the panels. The Z-bars, spaced 18 in. on centers, were later engaged by the cast-in-place concrete to form mechanical interlock. Four other panels were made without the shear lugs.

Panels were tested as simple beams with 86 in. span. They were loaded at midspan, and loads, deflections, and strains on top, middle and bottom surfaces were monitored. Failure was taken to be any condition which would render the panels unservicable or 1/4 in. deflection, whichever occurred first. Load, strain, and deflection data were monitored to detect distress. All specimens under cyclic loading failed by deflecting 1/4 in. with the exception of one which was loaded through 11.9 million cycles at 210 percent of design load without failure.

There was no indication in any panel of bond failure at the interface of prestressed and cast-in-place concrete. No prestress strand failure by fracture or slipping was indicated.

Both panel types were loaded in excess of 200 percent of design load before they deflected 1/4 in. which was designated a failure condition. In static testing, both panels had the same stiffness up to approximately the design load; beyond that load, the panel with shear lugs was stiffer.

Curves of load versus number of load cycles at failure, S-N curves, were developed from fatigue tests. The panel with shear connectors consistently took more load cycles to failure for loads ranging from 210 percent of design load to 260 percent of design load, although the differences were not very great at higher loads. At 210 percent of design load the panel with shear lugs was cycled 11.9 million cycles without failure. The specimen without shear lugs failed by deflection, 1/4 in., at 2.25 million cycles under 210 percent of design load.

IMPLEMENTATION

The test panels of this study showed no evidence whatsoever of failure of bond at the interface of prestressed and cast-in-place concretes, and no failure of bond of prestressing strands was evident. The very high number of load applications at more than 200 percent of design load shows that this composite type of bridge deck is adequate for field service. Either the panel with or without shear connectors, Z-bars, may be used with assurance of good service. The panel with Z-bar shear lugs has the advantage of greater stiffness at higher loads. That stiffness endured, under cyclic load at about 200 percent of design load, longer than it did for the panel without shear connectors.

Particular care should be taken in the fabricating yard and the bridge construction site to insure that the top surfaces of the prestressed panels do not receive curing compounds. The top surface must be free of debris, dust, and grease for good bonding of the cast-in-place deck concrete. Bonding of these surfaces is not only important for stress transfer, but also to prevent accumulations of moisture which might freeze and break the two concretes apart at the bond line.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

INTRODUCTION

Precast prestressed concrete panels are used in some bridges as stay-in-place forms which serve later as elements in the structural deck. They are bonded to the cast-in-place deck during construction. Wheel loads crossing the bridge develop shear and bending moment in the prestressed panels. The bending moments cause additional tension in the prestressing steel, and the shears cause horizontal shear at the bonded interface of the panels and cast-in-place concrete. There has been considerable research to determine the effect of repeated loads on bond of prestressing steel, but little has been reported on the behavior of the bond at the interface of the prestressed panels and cast-in-place concrete. This report gives details and results of laboratory tests to determine the effect of repeated loads on interface bond of prestressed panels overlaid with cast-in-place concrete.

TEST SPECIMENS

The precast prestressed concrete panels, 7 ft-8 in. long, 22 in. wide, and 3 1/2 in. thick, were cast in a commercial precasting yard. An upper layer of concrete was cast later on top of these panels. Prestressing strands were 270 ksi, 3/8 in. diameter, 7 wire cables. They were placed at mid-depth of the prestressed panels, and were tensioned initially to 16.1 kips. The cables extended 3 in. beyond each end of concrete to later engage the cast-in-place material. The calculated prestress, after 20 percent loss, was 835 psi.

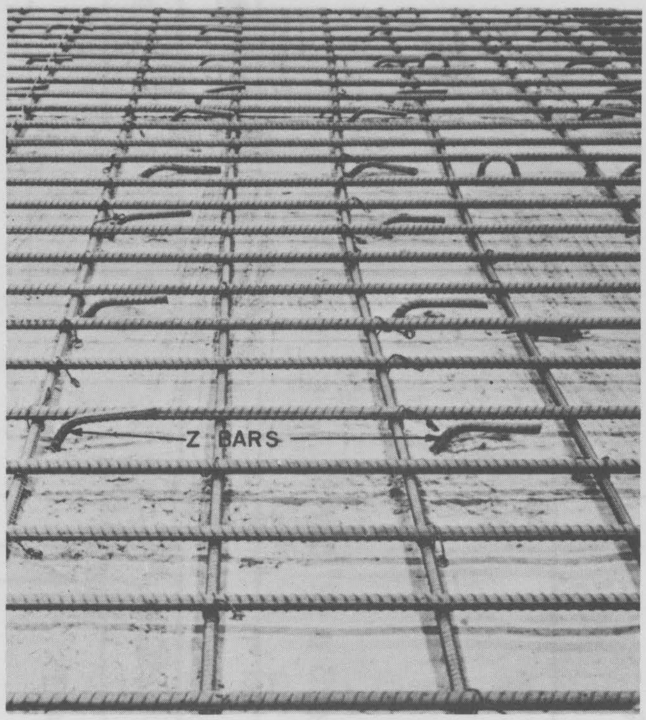
Mechanical interlock was provided between the prestressed panel and the cast-in-place concrete in 3 of the 7 panels. Number 4 reinforcing rods, bent to a rounded Z configuration, were used for the interlock. They were cast in the prestressed panel and were later engaged by the cast-in-place concrete. Details of the interlocking steel elements are shown in Figures 1 and 2. The prestressed panels were given a rough surface finish with a stiff broom. They were cured under wet mat for 6 days after which they were stored in the yard prior to being hauled some 170 miles to the testing laboratory. No curing compound was used on the panels.

The panels were overlaid with a 3 1/2 inch layer of cast-in-place reinforced concrete when they were 90 days old. The overlay was cured under wet mat for 6 days, and the specimens then remained in open storage until they were tested -- approximately three months after the top layer of concrete was placed. Details of the reinforcing for the cast-in-place concrete are shown in Figure 2. Material properties are shown in Table 1.

All concrete was normal weight, using Type III cement for prestressed panels and Type I cement for the overlay. Crushed limestone was used for coarse aggregate in the prestressed concrete, and gravel was used in the cast-in-place material. Natural sand was used as fine aggregate in both materials.

INSTRUMENTATION

Longitudinal strains were taken from electrical resistance gages. These were located at the bottom surface of the prestressed panel, at



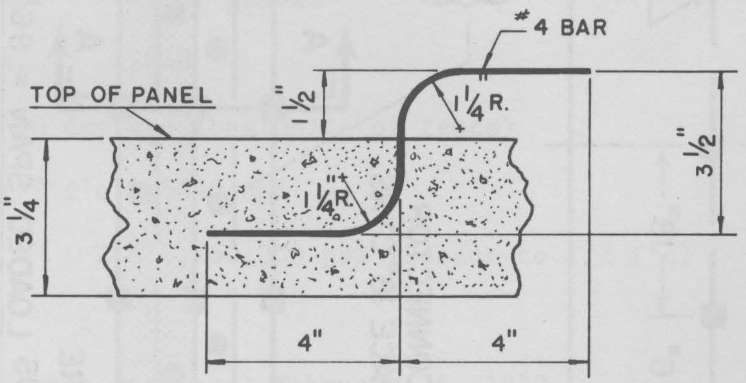


Figure 1. Z-Bars Used in Selected Panels to Aid in Providing Structural Connection Between Panel and Cast-in-Place Deck.

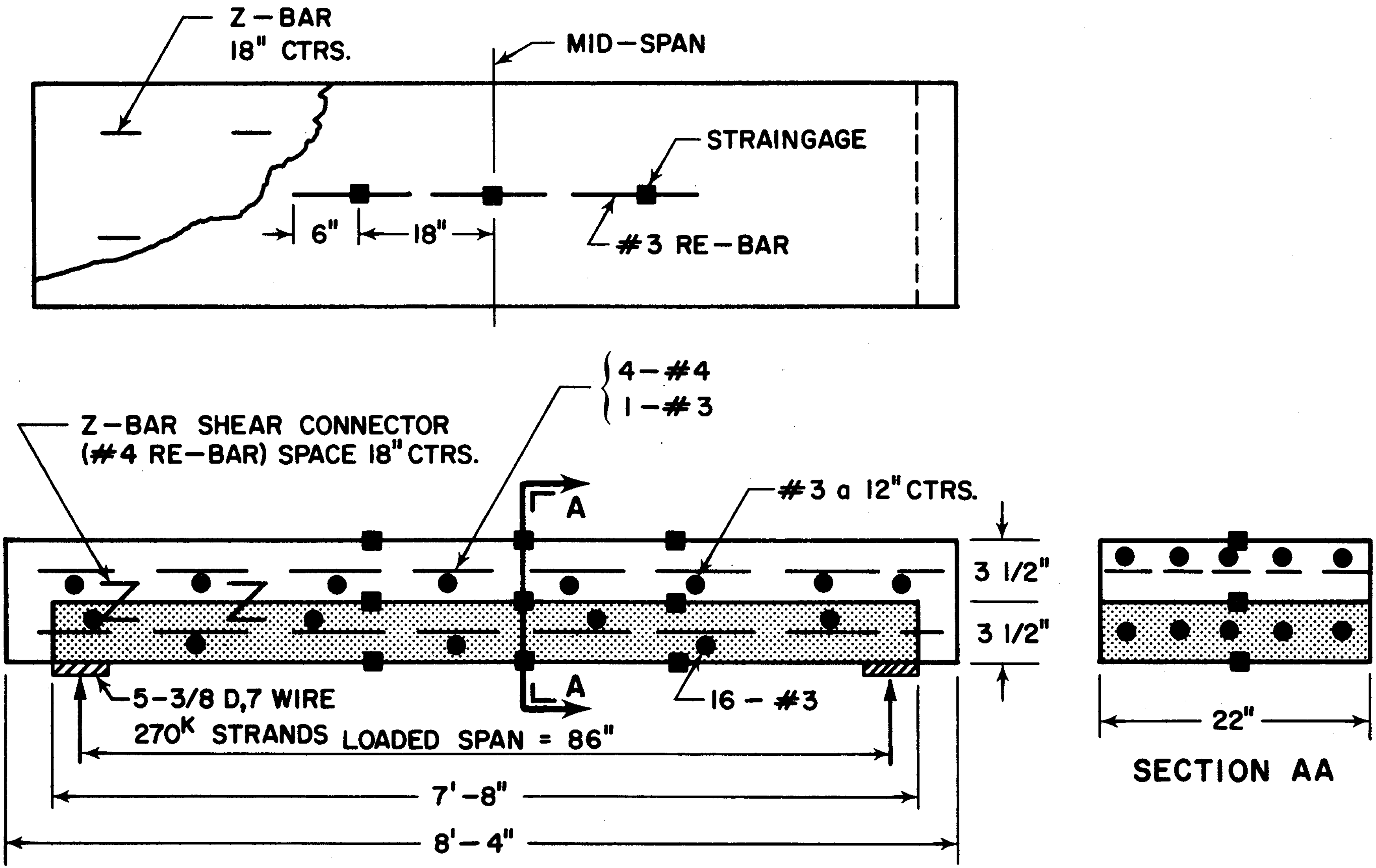


Figure 2. Test Panel Details.

TABLE 1 MATERIAL PROPERTIES

Prestressed Concrete:

Cement	Type III
C.A.	Crushed limestone
F.A.	Sand

At 160 day age (beginning of tests)

$f'c = 8670$ psi

$E = 5{,}130{,}000$ psi (Secant to 1/2 f'c)

Cast-in-place Concrete (overlay)

Cement	Type I
C.A.	River gravel
F.A.	Sand

At 70 day age (beginning of tests)

$f'c = 5540$ psi

$E = 5{,}250{,}000$ psi (Secant to 1/2 f'c)

Prestressed panels were cast February 25, 1972.

Cast-in-place concrete was cast May 26, 1972.

Panel testing began August, 1972.

the interface of prestressed and cast-in-place concrete and at the top surface of the cast-in-place concrete. They were made by casting a 4 ft long number 3 reinforcing bar in the concrete at these levels. After the concrete cured the embedded rod was ground smooth at its center and 6 in. from each end. The strain gages (6 mm, TML Type FLA-6-11) were then attached and waterproofed, as shown in Figure 2. The output data from the gages were recorded with a Visicorder Recorder.

Loads, load rates, and deflections at midspan were monitored through a load cell and a deflection transducer in the Gilmore loading machine. Load and deflection signals were displayed continuously on an oscilloscope screen. Strain readings were made at intervals of four hours of running time.

TESTS

Eight load tests were made on seven composite panels. They were tested in sets of two panels, one with Z bars and one without. Identical tests, except as noted below for the static tests, were made on the two panels of each set. Six panels were tested under cyclic load and two were tested statically. One of the static load tests was made on a panel that had already completed its cyclic testing, test number 3b Table 2, because of a shortage of panels.

Panels were uniformly supported across their full width by bearing on 1/2 in. x 5 in. steel plates which were supported on rockers. They were clamped down at the ends to prevent rebound impact during cycling. The load was applied through a steel beam extending across the full width of the panel. A steel plate, 1/2 in. x 5 in., was set in plaster-of-paris to transmit the load from the beam to the top of the panel. The same system of support and load application was used in all tests, static and cyclic.

Static loads were applied in 500 lb increments at midspan. Simultaneous readings of load and midspan deflection were made until the beams deflected about 1/4 inch. No indication of rupture or of bond failure were observed in these tests. Tensile cracks closed in both specimens when the load was removed.

Cyclic loads of a one-half sine wave form were programmed at the maximum rate that the machine could handle with the load requirements set for the particular test. At the lower loads those rates were about 16 cps. At the highest loads, the rates had to be reduced to about 10 cps. Specimens were mounted in the frame in the manner described for static loading. All strain gages were set to zero readings under an initial 500-pound load. This load was sustained throughout the test to prevent possible impact hammering due to rebound of the specimens.

Cyclic loads, in increments of 6,950 pounds, the design load, were scheduled for three sets of panels. Each set was load cycled until failure occurred. Excessive deflection (greater than 1/4 in.), or any condition which would make the panel unservicable constituted failure. The durability of bond between the prestressed panel and the cast-in-place concrete was of primary interest.

Panel 1a, with no Z bars, was tested first. At two million load cycles under 65 percent of design load it showed no evidence whatsoever of failure. No visible cracks developed anywhere in the panel, and

TABLE 2 SCHEDULE OF TESTS

Test Number	Test Specimen	Type of Test	Load Schedule			Remarks
			Load (lb)	Load ÷ Design Load	Number of Load Cycles	
1a	No Z bar	Repeated Load	4500	0.65	2,000,000	
			6750	0.97	2,000,000	
			9000	1.3	"	
			14000	2.0	"	
			18000	2.6	145,000	failed*
1b	Z bar	Repeated Load	4500	0.65	2,000,000	
			6750	0.97	2,000,000	
			9000	1.3	2,000,000	
			14000	2.0	2,000,000	
			18000	2.6	370,000	failed*
2a	No Z bar	Repeated Load	15750	2.3	1,314,000	failed*
2b	Z bar	Repeated Load	15750	2.3	2,000,000	failed*
3a	No Z bar	Repeated Load	14675	2.1	2,250,000	failed*
3b	Z bar	Repeated Load	14675	2.1	11,900,000**	
4a	No Z bar	Static	Figure 3			
4b	Z bar used in test #3b	Static	Figure 3			

* Failure condition 1/4 in. deflection at midspan.

** The test was discontinued at 11,900,000 cycles without failure.

neither strain nor deflection readings indicated that trouble had developed. The load was increased to approximately 100 percent of design load and carried through 2 million cycles and the slab again showed no evidence of trouble. This continued with 2 million load cycles at 130 percent of design load then with 2 million load cycle at 200 percent of design load. Small cracks developed at midspan in the tension zone under the latter load. Having 8 million cycles of load already completed on the specimen, the load was increased to 260 percent of design load. Cracks began to open up wider under that load, and the cyclic rate had to be decreased from 16 to 12 cps because of increased deflection. The specimen reached the 1/4 in. limit of deflection at 145,000 cycles under 260 percent of design load, and it was rated as a failure at that condition. No failure of bond of concrete to concrete, steel to concrete, nor crushing of concrete was found at the end of this test.

The second test, 1b in Table 2, was made on the Z-bar panel companion of panel 1a. The procedure was identical to that of the test described above, and the results were almost the same. Failure, 1/4 in. deflection, occurred under 260 percent of design load at 370,000 cycles after it had gone through 8 million cycles at lower loads.

Loads were reduced to 230 percent of design for the next set of two panels. Failure occurred in those panels by 1/4 in. deflection at 1,314,000 cycles for the panel without Z-bars, and at 2,000,000 cycles for the panel with Z-bars.

The third, and last, cyclic test was made at 210 percent of design load. The panel with Z-bars never failed. It was withdrawn at 11,900,000 cycles because of time. That panel was tested statically, test number 4b, before it was removed from the test frame. In the static test it failed by excessive deflection at a load of approximately 25 kips. No other trouble was found in the specimen.

The last test on non Z-bar panels failed in deflection at 2,250,000 cycles under 210 percent of design load. No other evidence of failure was found in the panel.

RESULTS AND DISCUSSION

Both panel types behaved essentially the same in static loading up to approximately the design load, Figure 3. At higher loads the panel with Z-bars displayed greater stiffness than the one without Z-bars. The Z-bar slab had previously undergone almost 12 million load cycles at 210 percent of design load. The panel without Z bars had never been loaded before the static tests. Cracks in both panels closed when the load was removed.

Closure of tension cracks in the panels indicates that the steel was still elastic. Some additional load could have been carried before a rupture type of failure, but it is not likely that the designer would permit more deflection than the span-to-deflection ratio of $86 \div 1/4 = 344$ that was developed in these tests. The panels would be much stiffer if they were continuous over end supports with other deck panels. Under this latter condition, greater load could be carried for the same deflection, but the horizontal shear at the bonded interface would increase with the load.

In all of the cyclic tests performed, the deflection of the panels limited their load capacity as defined in these tests. If greater

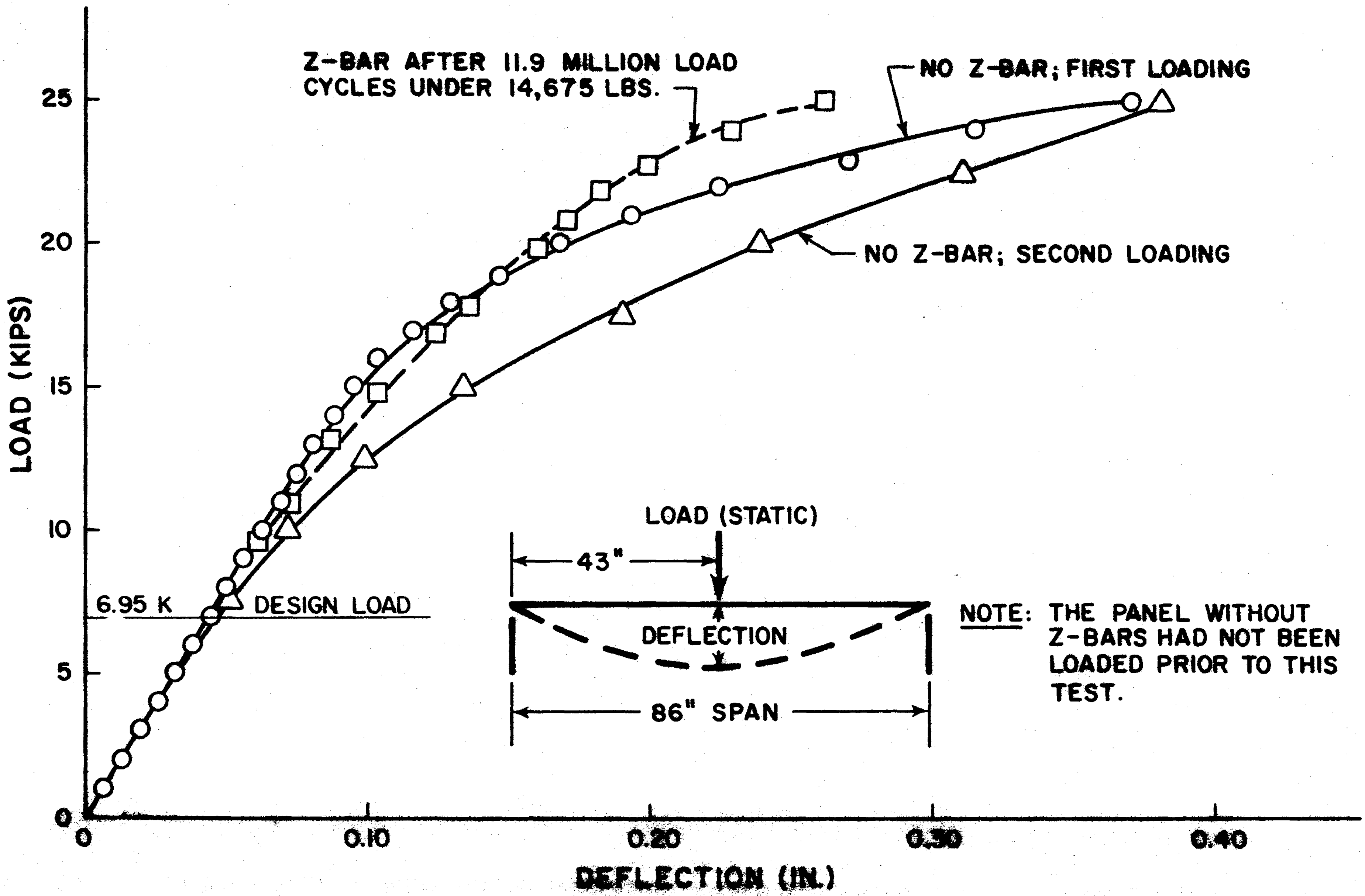


Figure 3. Static Load-Defelction Curves.

deflection had been permitted, greater loads could very likely have been carried. The load deflection curve of the static tests appear to have reached the typical "after cracking" stage (1) with at least some reserve strength for additional load. But, behavior under heavier cyclic loading has not been carried out to a sufficient number of cycles to determine if failure of bond at the interface of the prestressed and cast-in-place concrete will develop.

The plot of load versus number of load cycles at failure, Figure 4, indicates that the endurance limit of the panel with Z-bars is a little greater than for the other panel. The Z-bar panel has a limit at some value near 225 percent of its design load whereas the limit for the no Z-bar panel is near 200 percent of design load.

Miner's theory (2) was used to determine cumulative fatigue damage. In the Z-bar panel, test 1b, Table 2, the cumulative damage is negligible since all load values, except the failure load, fall below the endurance limit. However, the panel without Z-bars, test 1a, Table 2, has a cumulated damage development in its loadings up to 200 percent of design load. If the S-N curve for the panels with Z-bars is projected out along the abscissa, it will level out at about 10 million cycles at 200 percent of design load. No load lower than about 200 percent of design load, on that basis, will damage the specimen. The 2 million cycles at 200 percent of design load accounts for about 20 percent of the total damage value of the specimen. The 145,000 cycles at 260 percent of design load accounts for the remaining 80 percent of its life. The full cyclic life at 260 percent of design load would then be 1.2 x 145,000 or 174,000 cycles. On the scale of the curve of Figure 4 the difference

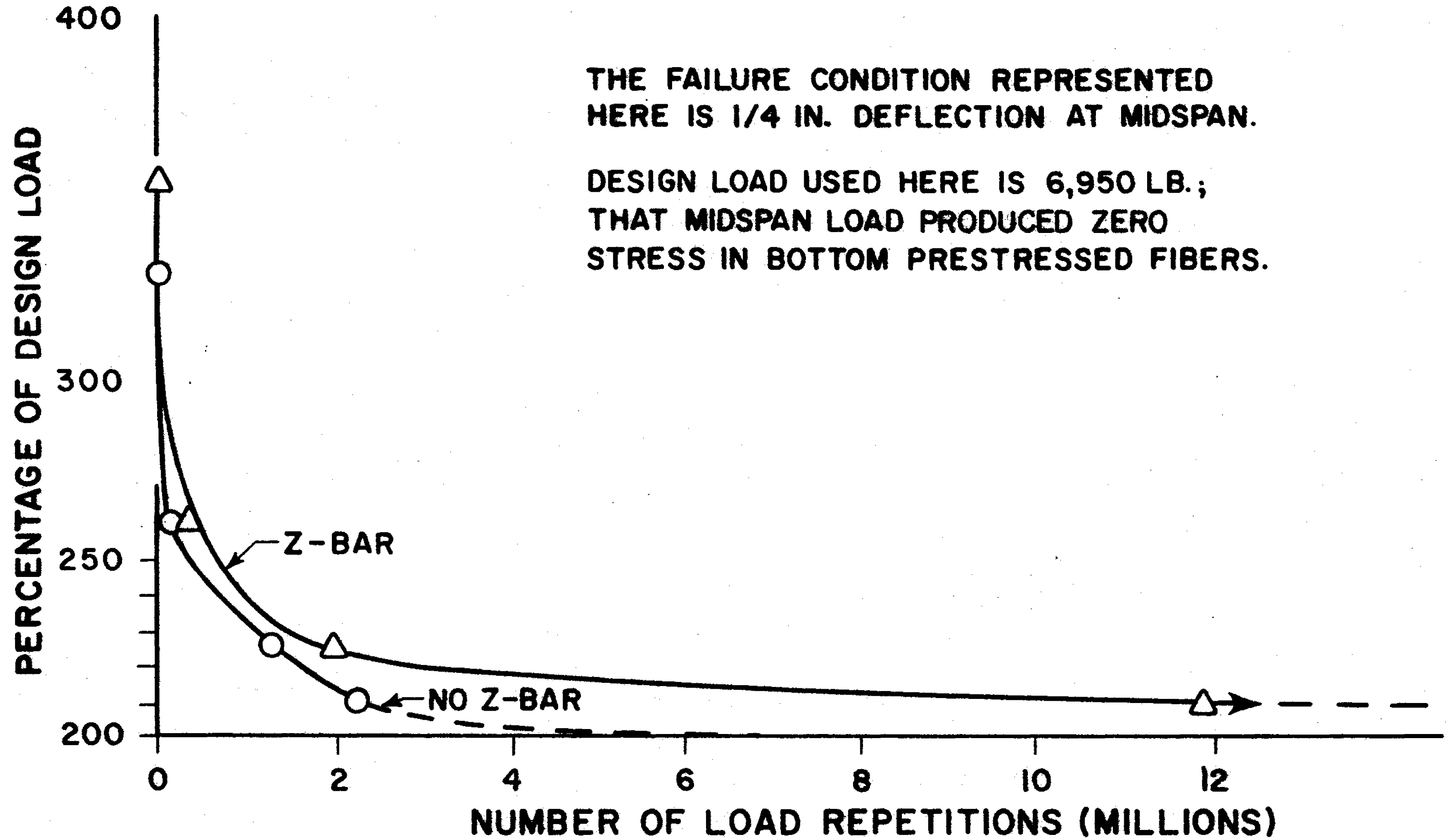


Figure 4. Load versus Number of Load Repetitions at Failure.

in 145,000 and 174,000 is hardly perceptible. One panel without Z-bars was prepared for a test at 200 percent of design load but it could not be tested for lack of time. One, or possibly two, additional points for each panel on the S-N curve are desirable, but the curve is reasonably well defined by those values that were developed.

CONCLUSIONS

1. No indication of bond failure between prestressed concrete and cast-in-place concrete was detected in any specimen.

2. There was no apparent loss of bond of prestressing strands during the tests.

3. There was a gradual loss of stiffness in the panels under cyclic load. That loss of stiffness resulted in a deflection of 1/4 inch, termed failure deflection, in each specimen except Number 3b. No other type of failure was detected.

4. The specimens with shear connectors underwent more load cycles to failure (1/4 inch deflection) than did the specimens with no shear connectors.

5. The specimen with shear connectors did not fail under 210 percent design load at 11.9 million cycles. (Design load is that load which theoretically causes tension to develop in the bottom fibers of the prestressed element of the composite panel.)

6. The specimen with no shear connectors failed (1/4 inch deflections) at 2.25 million cycles under 210 percent of design load.

7. The panels with shear connectors have a slightly greater load-to-deflection ratio than the panels without the connectors at loads higher than design load.

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