Executive Summary

EXPERIMENTAL STUDY OF TWO PRESTRESSED STEEL BEAM-CONCRETE SLAB BRIDGE UNITS

by

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Sponsored by

Oklahoma Department of Transportation
Research and Development Division

in cooperation with the

Federal Highway Administration

Report No. FSEL/ODOT 87-02

May 1987

This publication was printed and issued by the Oklahoma Department of Transportation as authorized by Neal A. McCaleb, Director. 155 copies have been prepared and distributed at a cost of $208.40.

TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
FHWA/OK 87 (1)		

4. TITLE AND SUBTITLE
Experimental Study of Two Prestressed Steel Beam-Concrete Slab Bridge Units

5. REPORT DATE
May 1987

6. PERFORMING ORGANIZATION CODE

7. AUTHOR(S)
Thomas A. Densford and Thomas M. Murray

8. PERFORMING ORGANIZATION REPORT

9. PERFORMING ORGANIZATION NAME AND ADDRESS
Fears Structural Engineering Laboratory
University of Oklahoma
303 E. Chesapeake St.
Norman, OK 73019

10. WORK UNIT NO.

11. CONTRACT OR GRANT NO.
Agreement #28, 82-02-2

12. SPONSORING AGENCY NAME AND ADDRESS
Research and Development Division
Oklahoma Department of Transportation
200 NE 21st
Oklahoma City, OK 73105

13. TYPE OF REPORT AND PERIOD COVERED
Final
Jan '85 - May '87

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES in cooperation with

Federal Highway Administration, US Department of Transportation

16. ABSTRACT The behavior of two 55 ft. long prestressed, composite steel beam-concrete slab bridge units was studied. The type of unit tested is currently used in county road bridge construction, where the use of prefabricated units is especially economical.

In primary test phases, the first unit was subjected to 3 years of sustained loading, over 2,000,000 cycles of fatigue loading and statically loaded to failure. The second unit underwent 500,000 cycles of fatigue loading and was statically loaded to its yield level.

In supplementary test phases, pushout-type specimens with channel and stud shear connectors, identical to those in the bridge units, were studied to determine the difference between the two connector types under sustained and ultimate loading conditions. In addition, transverse slab strength tests were performed at six locations on the first unit, and on six similar, simply supported, control slabs. The transverse slab strength tests were performed to verify that arching action occurs in the bridge slab. The presence of arching action in the bridge slab changed the mode of slab failure from a relatively ductile flexural failure, to a sudden punching failure at a much higher concentrated load.

Test results were compared to theoretical predictions and AASHTO Specification limitations. It was found that the behavior of the unit was reasonably predictable, and that with a minor connection detail change, the prestressed, composite steel beam design concept is suitable for county road bridge use.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
bridge, pre-stressed, composite, pre-fabricated, creep, shear connector, fatigue	no restrictions

19. SECURITY CLASSIF. (OF THIS REPORT)	20. SECURITY CLASSIF. (OF THIS PAGE)	21. NO. OF PAGES	22. PRICE
none	none	45	

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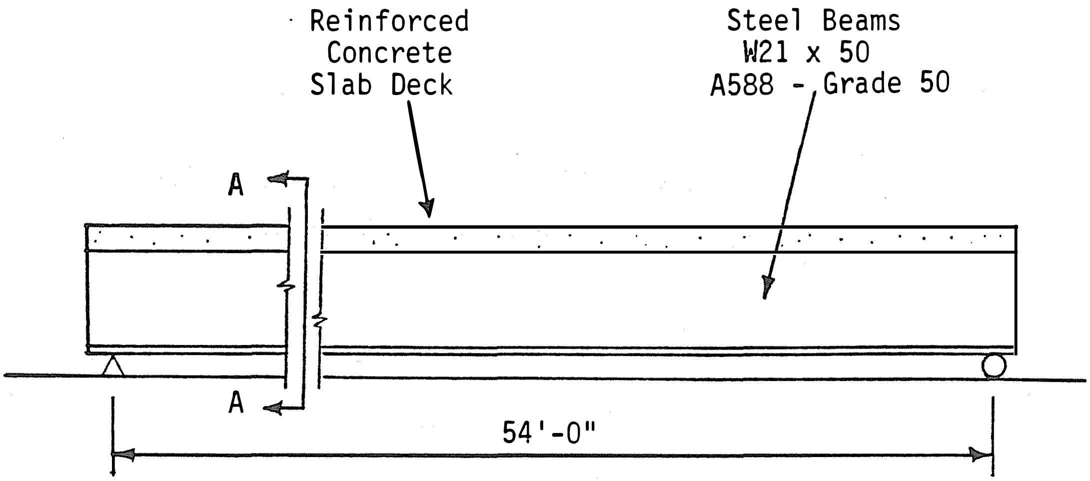
CHAPTER 1

INTRODUCTION

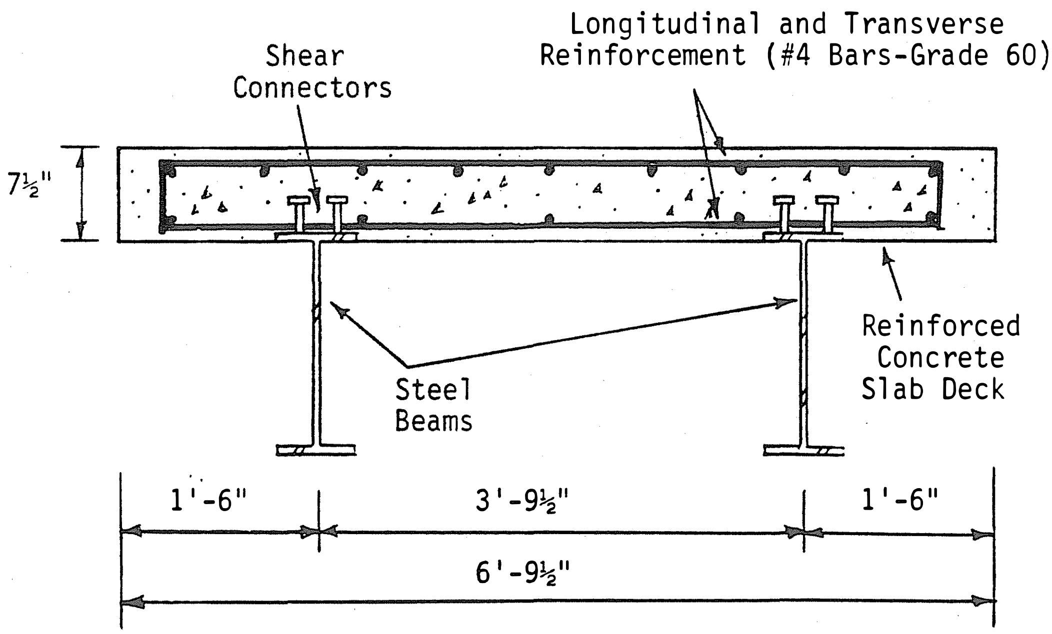
1.1 General

This report is a summary of a four year research program involving the experimental study of two prototype precast, prestressed steel beam-concrete slab bridge units. Typical units consist of a concrete slab attached to two steel beams by shear connectors as shown in Figure 1.1. The units are usually prefabricated and transported to a site, where a bridge is constructed by placing two or more units on abutments and connecting individual units with angle X-brace steel diaphragms. These bridge units are now being used for county road bridges, but the possibility of use in state highway bridges exists.

The method of construction used to produce the bridge units is unique and patented. Shear connectors are welded to two steel beams which are inverted and simply supported above a form containing a mat of concrete reinforcing steel. Concrete forms are then hung from the steel beams as shown in Figure 1.2 and the bridge deck concrete is poured into the forms. Additional dead load may be applied to the beams to increase the unit deflection to a predetermined amount so that the desired prestress level in the steel beams is obtained. When the concrete has cured and the unit



(a) Elevation



(b) Section A-A

Figure 1.1 Typical Bridge Unit Details

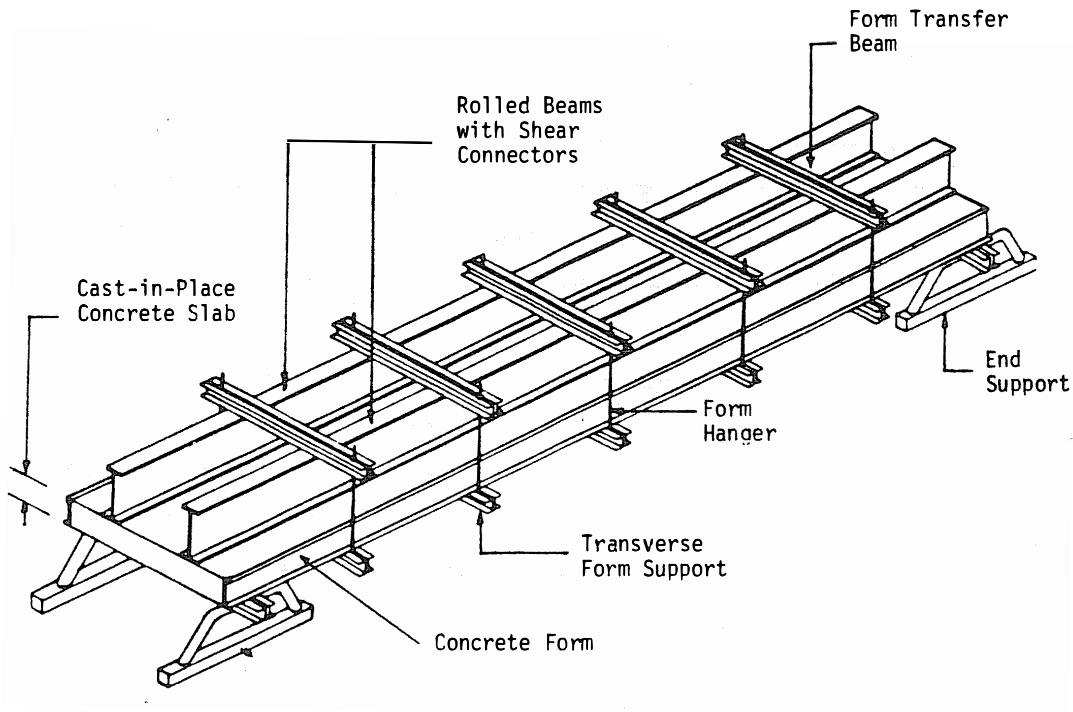


Figure 1.2 Method of Fabrication of Bridge Unit

is unloaded, forms are stripped, and the unit turned over. The resulting composite beam is similar to that obtained using shored construction methods, but with additional stressing of the steel beam in the direction opposite to in-place gravity stresses.

This prestressing extends the service load range of the units as illustrated in Figure 1.3, which also shows the behavior of a conventional composite beam constructed without shores. Because of the method of construction, dead loads are resisted by the full capacity of the composite beam, resulting in substantially reduced dead load deflection and tension flange stresses when compared with unshored composite construction values. The net result is an increased service load range for the unit. However, as Figure 1.3 shows, the ultimate moment capacity of the cross-section is not affected by the choice of construction method.

Another advantage of the prestressed composite bridge unit is that the permeability of the deck may be reduced. Since the slab is cast in an inverted position, a reduction in concrete deck permeability is possibly obtained because the bleedwater capillaries in the curing concrete open toward the bottom of the in-place unit. The resulting possible resistance to water penetration may reduce corrosion of the deck reinforcing steel and accompanying maintenance problems.

A disadvantage of this method of construction is that mild steel is used as the prestressing element as opposed to very high strength steels (prestressing strands) that are used in the construction of conventional prestressed concrete beams. Since the service load capacity of the bridge units is dependent on a sustained level of prestressing, the Research and Development Division of the

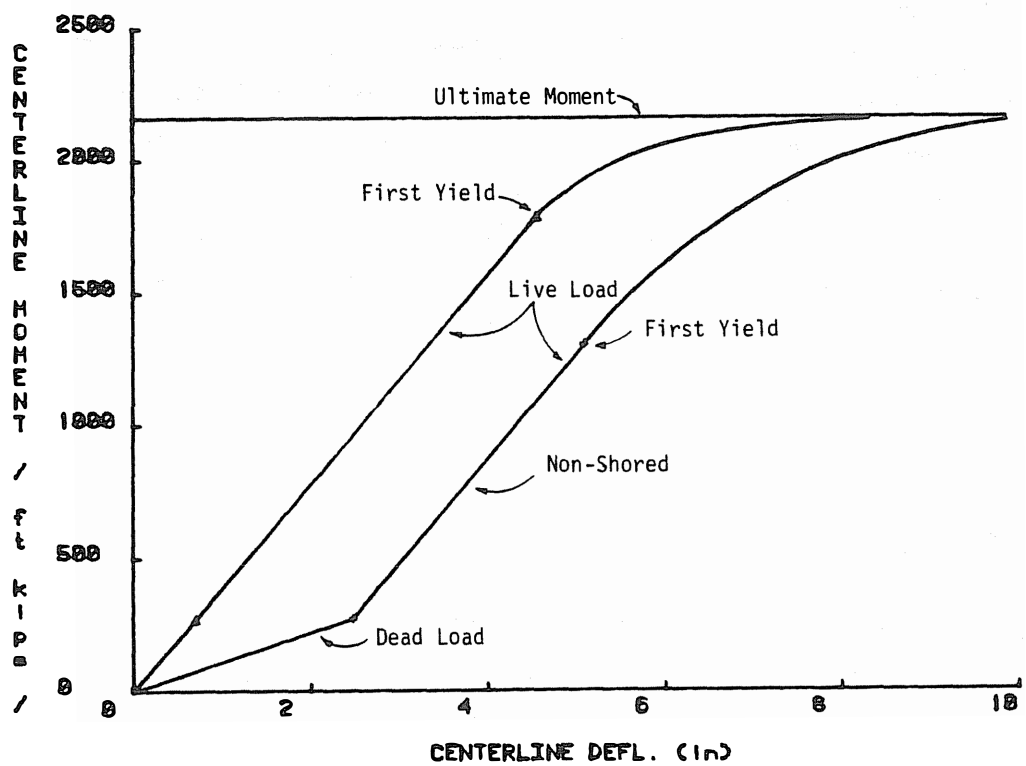


Figure 1.3 Bridge Unit Moment vs. Centerline Deflection

Oklahoma Department of Transportation commissioned an extensive study of the behavior of bridge units under sustained, repeated and static failure loadings. Long term sustained loading was used to study the effects of temperature change and concrete creep; repeated loading was used to determine the adequacy of the bridge unit design under a lifetime of truck loading; and the ultimate strengths of the unit in both longitudinal and transverse directions were determined under static loading. In addition, supplementary test series were conducted to investigate other aspects of the structural behavior of the units. Complete details of the study are found in Reference 1.

1.2 Overview of Testing Program

1.2.1 General

The testing program was divided into the phases shown in Table 1.1 and conducted in the order shown in Table 1.2. Two nearly identical bridge units were used to conduct the tests with the research phases separated into primary and supplementary tests. In the primary test phases, one of the units was subjected to alternating periods of sustained loading and repeated loading to simulate typical service life conditions. This unit was also subjected to overloading and to ultimate strength tests in the primary phases. The first unit was accidentally dropped between Phases IV and V (see Table 1.1) and as a consequence, the results of the static flexural test to failure (Phase VIII) are questionable. A second unit was then constructed and used for Phases IX thru XI.

In the two supplementary test phases, tests were conducted on the first bridge unit to determine the ultimate strength of the concrete deck in the transverse direction,

Table 1.1

Research Phases

Phase	Description
	Unit 1
I.	First bridge unit preparation and one year of observation under sustained loading.
II.	Repeated (HS-20) loading of 500,000 cycles.
III.	Operating rating (HS-30) loading test.
IV.	Two years of observation under sustained loading (totaling three years of sustained loading).
V.	An additional 1,500,000 cycles of repeated (HS-20) loading.
VI.	Repeated operating rating (HS-30) loading of 2,000 cycles.
VII.	Repeated unbalanced loading of 100,000 cycles.
VIII.	Static flexural test to failure of first unit.
	Unit 2
IX.	Second bridge unit preparation and 500,000 cycles of repeated (HS-20) loading.
X.	Static flexural test to first yield of second unit.
XI.	Observation of second bridge unit under sustained loading.
	Supplementary Tests
XII.	Transverse slab strength tests on first bridge unit.
XIII.	Shear connector specimen observation and strength tests.

Table 1.2

Chronological Summary of Research

Dates	Comments
1 April 1982	Concrete poured for first bridge unit.
8 April 1982	First bridge unit placed outside Fears Structural Engineering Laboratory (FSEL).
22 April 1982- 11 May 1983	Phase I, observation of first bridge unit under long term (one year) sustained loading.
3 March 1983- 19 July 1985	Phase XIII A, observation of shear connector specimens under long term sustained loading (810 days).
2 June 1983- 15 Sept 1983	Phase II, first unit moved into FSEL and subjected to 500,000 cycles of repeated (HS-20) loading.
23 Sept 1983	Phase III, first unit tested under operating rating (HS-30) loading.
30 Sept 1983- 4 Sept 1985	Phase IV, first unit moved outside FSEL and observed under two years of sustained loading (700 days).
5 Sept 1985	First unit accidentally dropped when transport was attempted.
6 Sept 1985- 2 Oct 1985	Repair and curing of damaged portion concrete slab of first unit.
21 Sept 1985	Phase XIII B, shear connector specimen failure tests.
3 Oct 1985- 20 Nov 1985	Phase V A, first unit brought into FSEL and subjected to 600,000 cycles of repeated (HS-20) loading.
21 Nov 1985	Phase VI, first unit subjected to 2,000 cycles of operating rating (HS-30) loading.

Table 1.2, Continued

Chronological Summary of Research

Dates	Comments
25 Nov 1985- 1 Jan 1986	Phase V B, first unit subjected to 900,000 cycles of repeated (HS-20) loading.
8 Jan 1986- 20 Jan 1986	Phase VII, first unit subjected to 100,000 cycles of repeated unbalanced loading.
6 Feb 1986	Phase VIII, static flexural test to failure of first unit.
19 March 1986	Concrete poured for second bridge unit.
21 March 1986- 15 April 1986	Phase XII, transverse slab strength tests using the first unit.
17 April 1986	First bridge unit removed from FSEL.
18 April 1986	Second bridge unit brought into FSEL.
22 April 1986- 22 May 1986	Phase IX, second unit subjected to 500,000 cycles of repeated (HS-20) loading.
28 May 1986	Phase X, test on second unit to determine first yield of cross section.
2 June 1986	Second bridge unit removed from FSEL.
3 June 1986- July 1986	Phase XI, observation of second bridge unit under sustained loading.

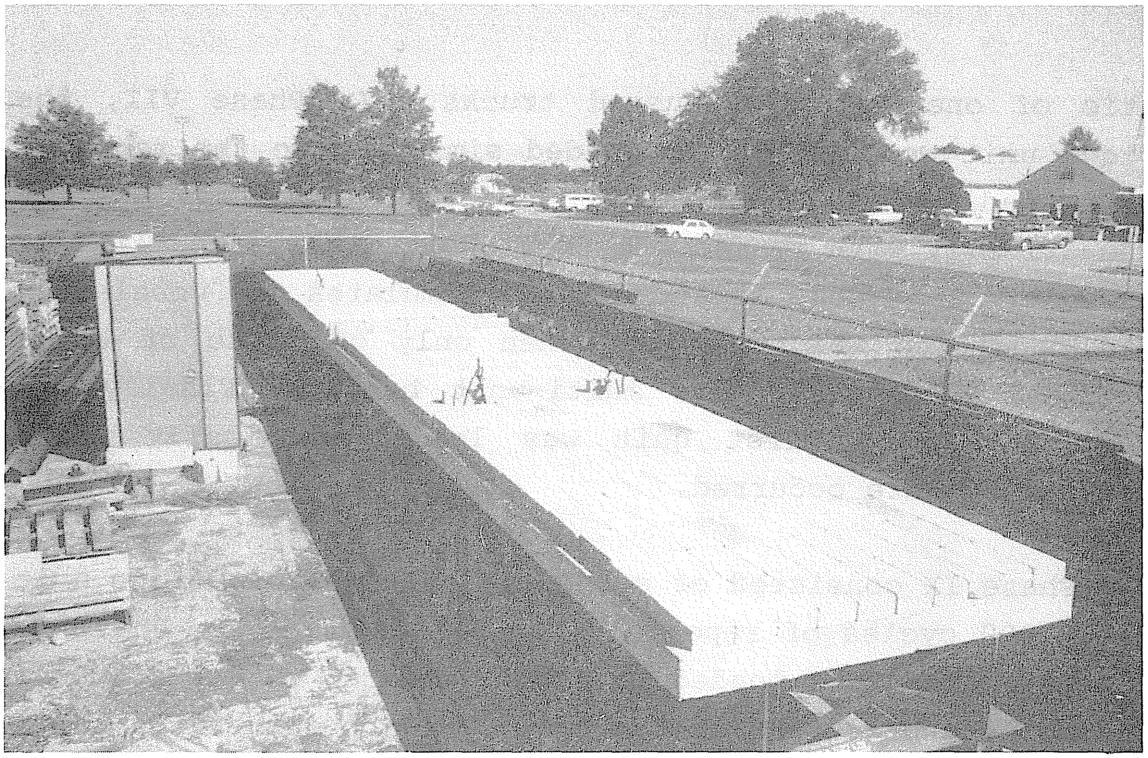
and on separately constructed shear connector specimens to study possible sustained loading effects for two types of shear connectors.

Testing details for all phases are found in Reference 1.

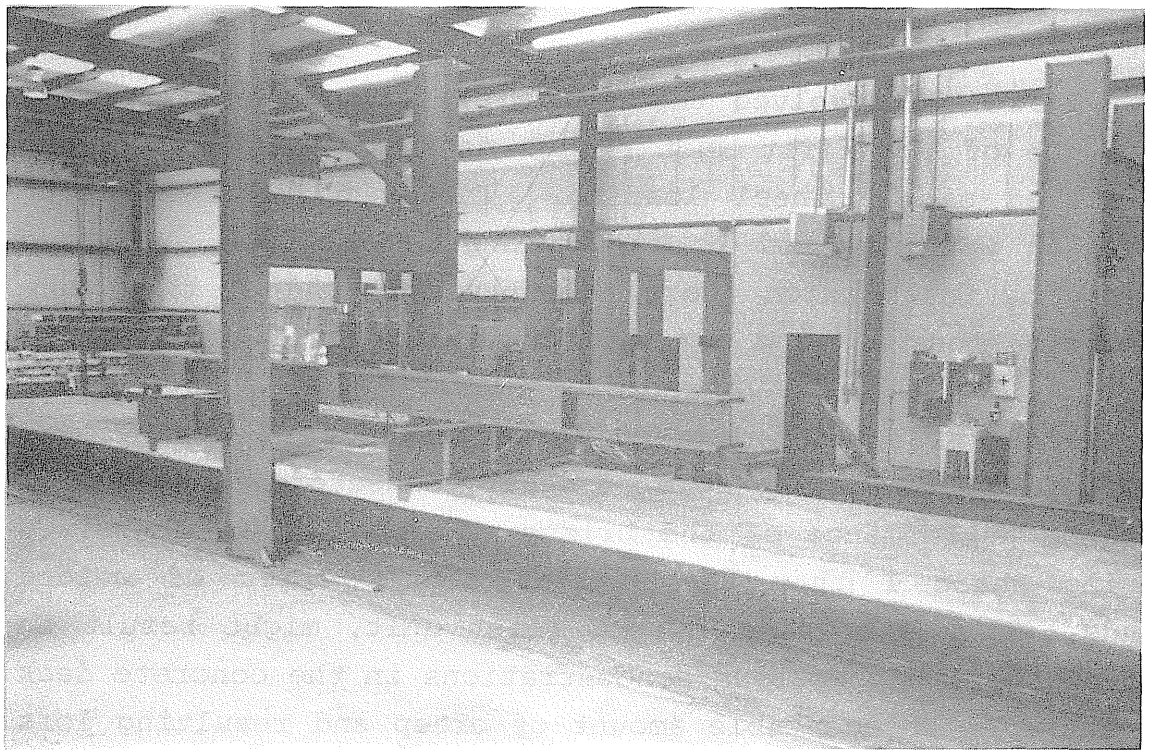
1.2.2 Primary Tests

Phases I through XI were considered to be primary test phases. Photographs of the two primary loading configurations are shown in Figure 1.4. Phase I consisted of one year of observation of the first bridge unit under sustained loading. The goal of this phase was to determine the response of the bridge unit to sustained loading and its response to temperature fluctuation. In Phase II, the bridge unit was subjected to a simulated truck traffic volume in the form of 500,000 cycles of repeated loading. The load magnitude corresponded to AASHTO Specification [2] HS-20 loading, adjusted by axle fraction and impact coefficients. Phase III consisted of subjecting the unit to a static overload which produced a maximum tension flange stress equal to 75% of the material yield stress. This loading corresponds to an operating rating load as defined in the AASHTO Specification [2] and is equal to 1.5 times the HS-20 load magnitude. It is referred to herein as an HS-30 loading. The unit was then observed under sustained loading, similar to Phase I, for two additional years which comprised Phase IV.

Phase V consisted of cycling the same bridge unit an additional 1,500,000 times under HS-20 loading (for a total of 2,000,000 cycles, the requirement for an interstate highway rating for the bridge design). Phase VI consisted of subjecting the bridge unit to 2000 cycles of operating rating (HS-30) loading, which represented a permit overload



(a) Sustained Loading Configuration



(b) Fatigue and Static Loading Configuration

Figure 1.4 Primary Test Loading Configurations ratio of one in one thousand trucks. In Phase VII, the bridge unit was cyclically loaded similarly to the repeated HS-20 loading of Phase V, except that the load was applied eccentrically with respect to the longitudinal centerline of the unit. This test conservatively simulates the unbalanced load condition which results when only one line of wheel loads is on a unit in a multi-unit bridge. Finally, in Phase VIII, the first unit was loaded statically until flexural failure occurred.

Phase IX consisted of subjecting the second bridge unit to 500,000 cycles of repeated (HS-20) loading. In Phase X, the second unit was loaded to first yield so that the amount of remaining prestress in the unit could be quantified after the repeated loading of Phase IX. Phase XI was a short observation period under sustained loading.

1.2.3 <u>Supplementary Tests</u>

Phase XII involved the determination of the transverse strength of the first unit bridge deck when subjected to a simulated single wheel loading. The in-situ bridge slab strength was compared to the strength of simply supported slab sections which were constructed using the same specifications as used for the test unit deck.

Phase XIII was initiated during Phase I of the primary tests to determine the role of shear connectors on sustained loading performance of the bridge units. It was theorized during Phase I that the smaller contact area of welded studs, which were used in the first unit, might result in sufficiently high stress concentrations in the concrete deck to cause an unacceptable amount of creep and resulting loss of prestress. One set of pushout-type specimens was constructed using welded shear connectors identical to those in the first unit. A second set was constructed using channel-type shear connectors. The specimens were observed under long term sustained loading so that creep and slip effects could be evaluated. On completion of the observation period, the failure strength of the shear connector specimens was experimentally determined (Phase XII B).

1.2.4 Bridge Unit Test Specimens

Two composite girder bridge units of nearly identical configuration were tested. Each unit consisted of two upright, parallel, 55 ft. long W21x50 steel beams of A588 Grade 50 steel, connected by $3x3x\frac{1}{4}$ in. steel angle cross-frame diaphragms, located at the ends and third points of the beams. Pairs of 3/4 in. diameter by 4 in. high welded stud shear connectors, spaced along the beam flanges in accordance with the AASHTO Specification [2] were welded to the beams prior to casting the concrete deck. For each unit, a full length, reinforced concrete slab of 6 ft. 9 1/2 in. width was cast against the top flanges of the parallel steel beams. Slab thicknesses were 7 1/2 in. and 7 in. for the first and second units, respectively. The slabs were cast using 5000 psi design strength concrete, reinforced with longitudinal and transverse, top and bottom, number 4 bars of Grade 60 yield strength steel. Specimen dimensions and details are shown in Figures 1.1, 1.5 and 1.6. Measured material properties for each unit are found in Table 1.3.

Instrumentation was similar for both units. Electrical resistance strain gages were mounted on selected longitudinal reinforcing steel bars and on the top and bottom flanges of the steel beams before the concrete slabs were cast. After the concrete slabs had cured and the units were stripped from formwork and turned upright, additional electrical resistance strain gages were mounted on the top

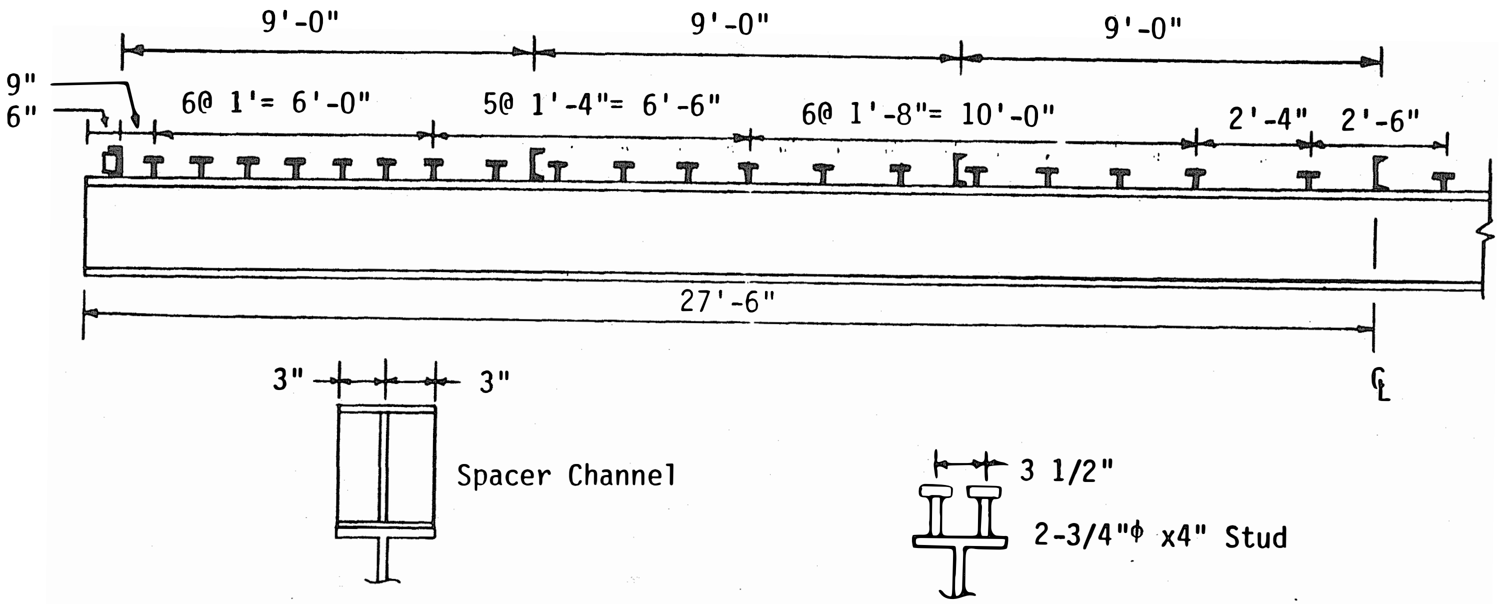


Figure 1.5 Bridge Unit Shear Connector Details

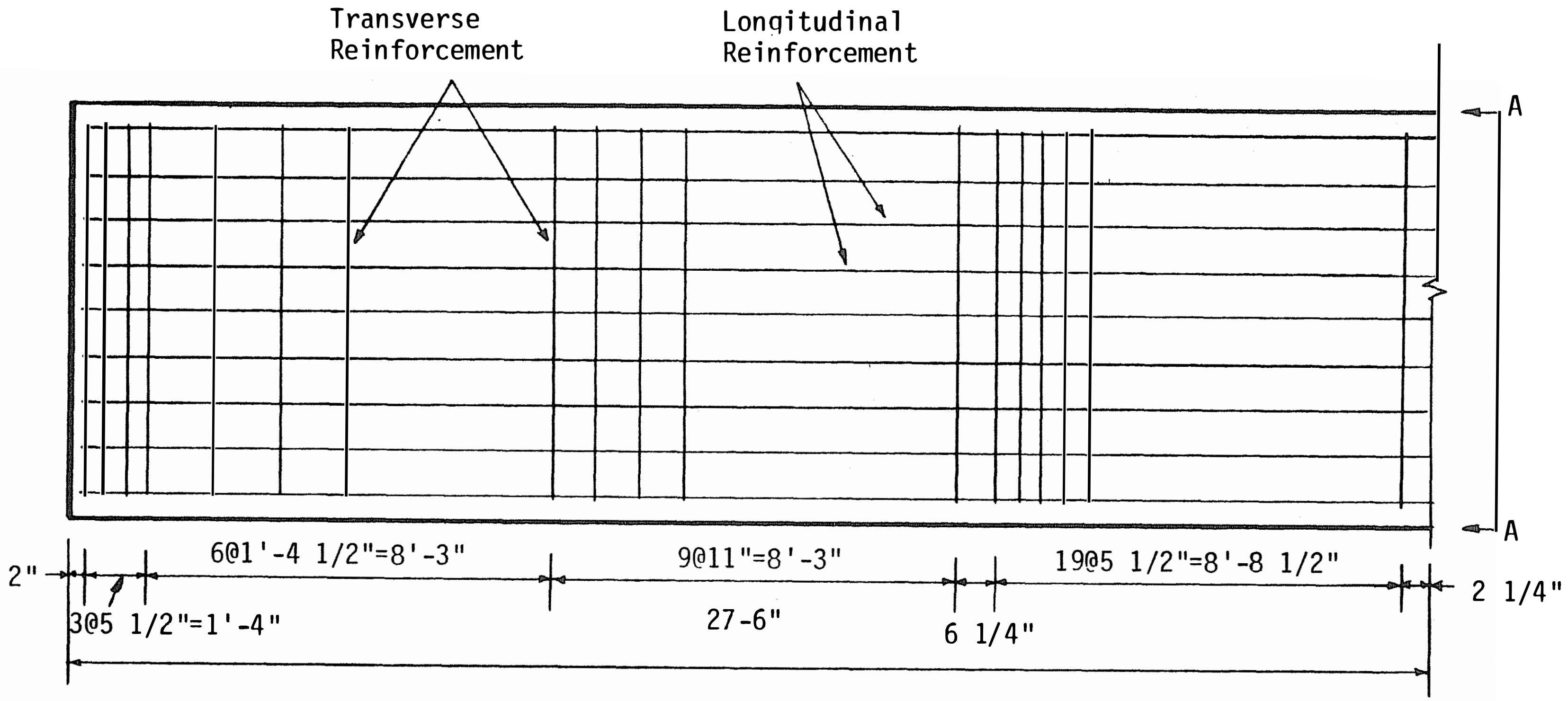


Figure 1.6 Bridge Unit Reinforcing Bar Details

Table 1.3

Measured Material Properties

(a) Steel Beams (W21x50, A588 Grade 50 Steel)

Test Specimen	Tensile Strength (ksi)	Elastic Modulus (ksi)
First Unit	56.0	29000.0*
Second Unit	58.0	29000.0*

(b) Reinforcement (#4 Bar - Grade 60)

Test Specimen	Tensile Strength (ksi)	Elastic Modulus (ksi)
First Unit	67.2	29000.0*
Second Unit	-	29000.0*
Control Slabs	79.5	-
Shear Connector Specimens	-	-

(c) Concrete (5.0 ksi Design Strength)

Test Specimen	Age at Cylinder Test (days)	Compressive Strength (ksi)	Elastic Modulus (ksi)
First Unit	28	5.30	4394.0*
	1408	7.40	4365.0
Second Unit	51	6.45	5335.0
Control Slabs	120	6.54	-
Shear Connector Specimens	28	5.74	-

* Assumed or Calculated
\- Not Required

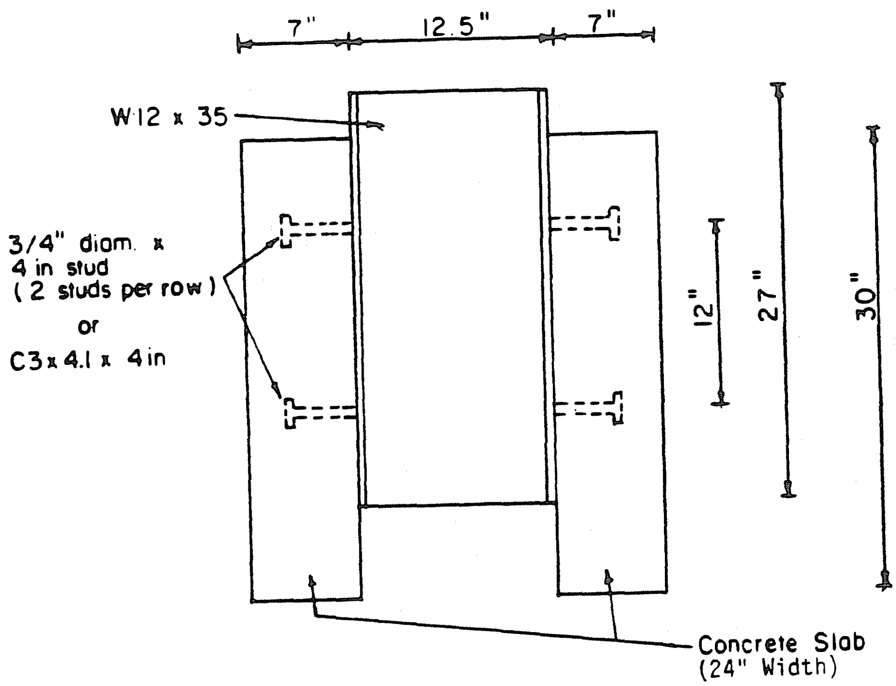


Figure 1.7 Overall Dimensions of Shear Connector Specimens

surface of the concrete slabs. All strain gages were located at the midspans of the units. Dial gages were used to measure relative movement of the concrete slabs with respect to the steel beams for the fatigue static loading phases of the research. Displacement transducers were used to measure support and midspan vertical movements. The test setups, instrumentation details and testing procedures are described in Reference 1. A summary of the results and significant observations are found in Chapter II of this report.

1.2.5 Supplementary Test Specimens

Control specimens for the transverse slab strength tests were six approximately square slabs constructed to match each of the three transverse reinforcing bar spacings in the first bridge unit. The only intended difference between the bridge unit slab and the control slabs was that the control slabs were tested when simply supported on steel pipe sections, whereas, the unit slab was constrained from axial displacement and rotation in both longitudinal and transverse directions due to the slab attachments to the steel beams and the longitudinal deck continuity.

The other supplementary test phases consisted of observing shear connector specimens under sustained loading followed by loading the specimens to failure. A total of four specimens were constructed; two with steel shear connectors and two with channel shear connectors. Specimen details are shown in Figure 1.7.

Complete descriptions of the testing procedure, instrumentation and results for both supplementary test series are found in Reference 1. A condensed summary of the results is found in Chapter II of this report.

CHAPTER II

SUMMARY AND OBSERVATIONS

2.1 Primary Tests

2.1.1 Sustained Loading Tests

In the sustained loading test phases, the first bridge unit was observed for a total of four years of sustained loading of 40 psf plus its own weight. The observation period for the second unit was less than 100 days including 500,000 cycles of repeated loading. The following observations were made concerning sustained loading behavior of the two bridge units:

1. Sustained loading phenomena is typified by increases in bottom flange stress and loss in camber of the bridge unit (see Figures 2.1, 2.2 and 2.3).

2. The effects of sustained loading phenomena on the first unit, characterized by creep and shrinkage of the concrete slab, reached a relatively asymptotic level after approximately 100 days of sustained loading. After that time, the strain and camber change of the unit varied inversely with the temperature change of the testing environment without a long term trend (see Figures 2.1 and 2.2).

3. The effects of sustained loading in the second unit were accelerated by the application of fatigue loading, but reached an asymptotic level upon completion of fatigue loading (see Figure 2.3).

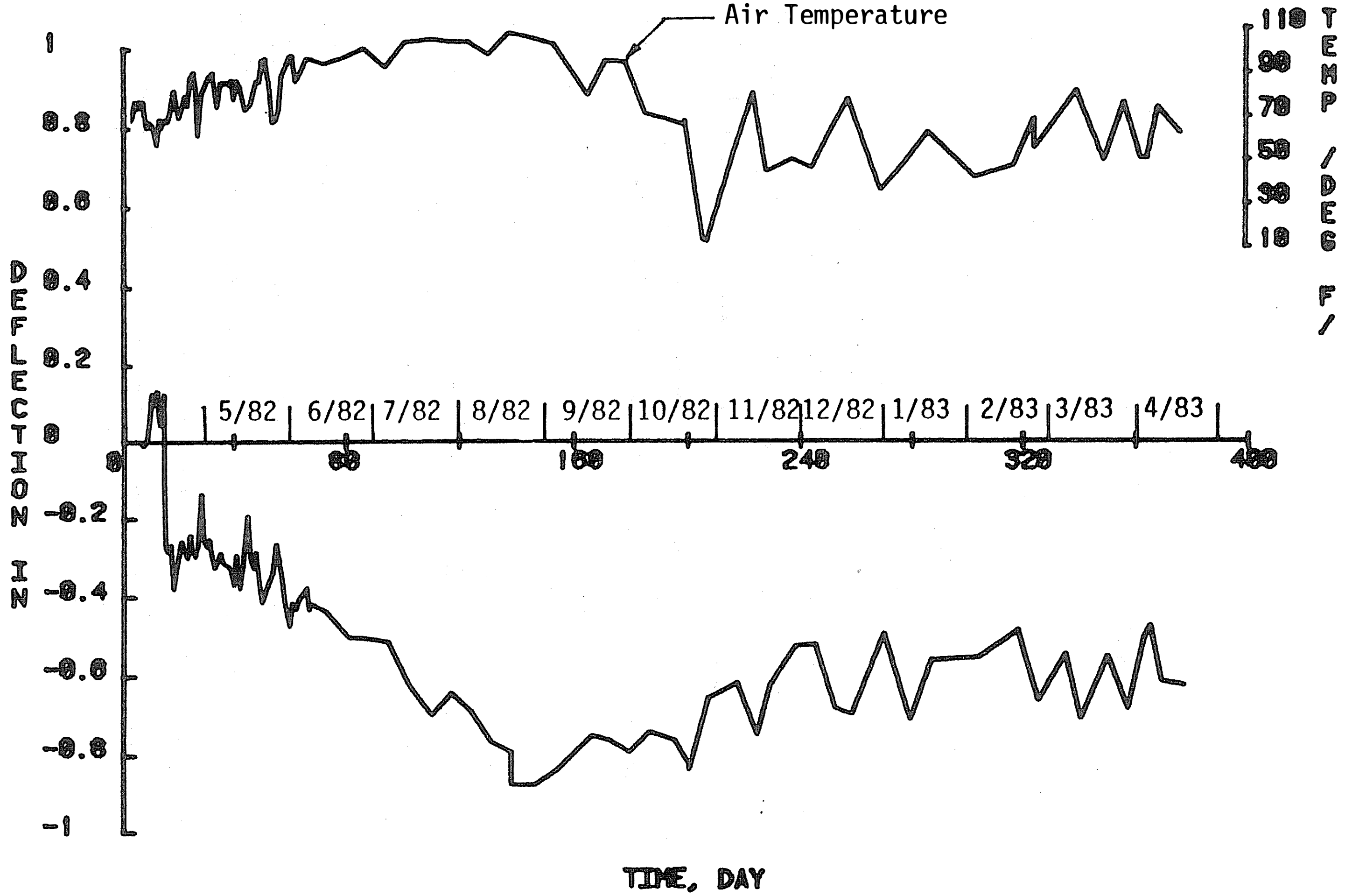


Figure 2.1 Change in Vertical Deflection vs. Time, Phase I Sustained Loading

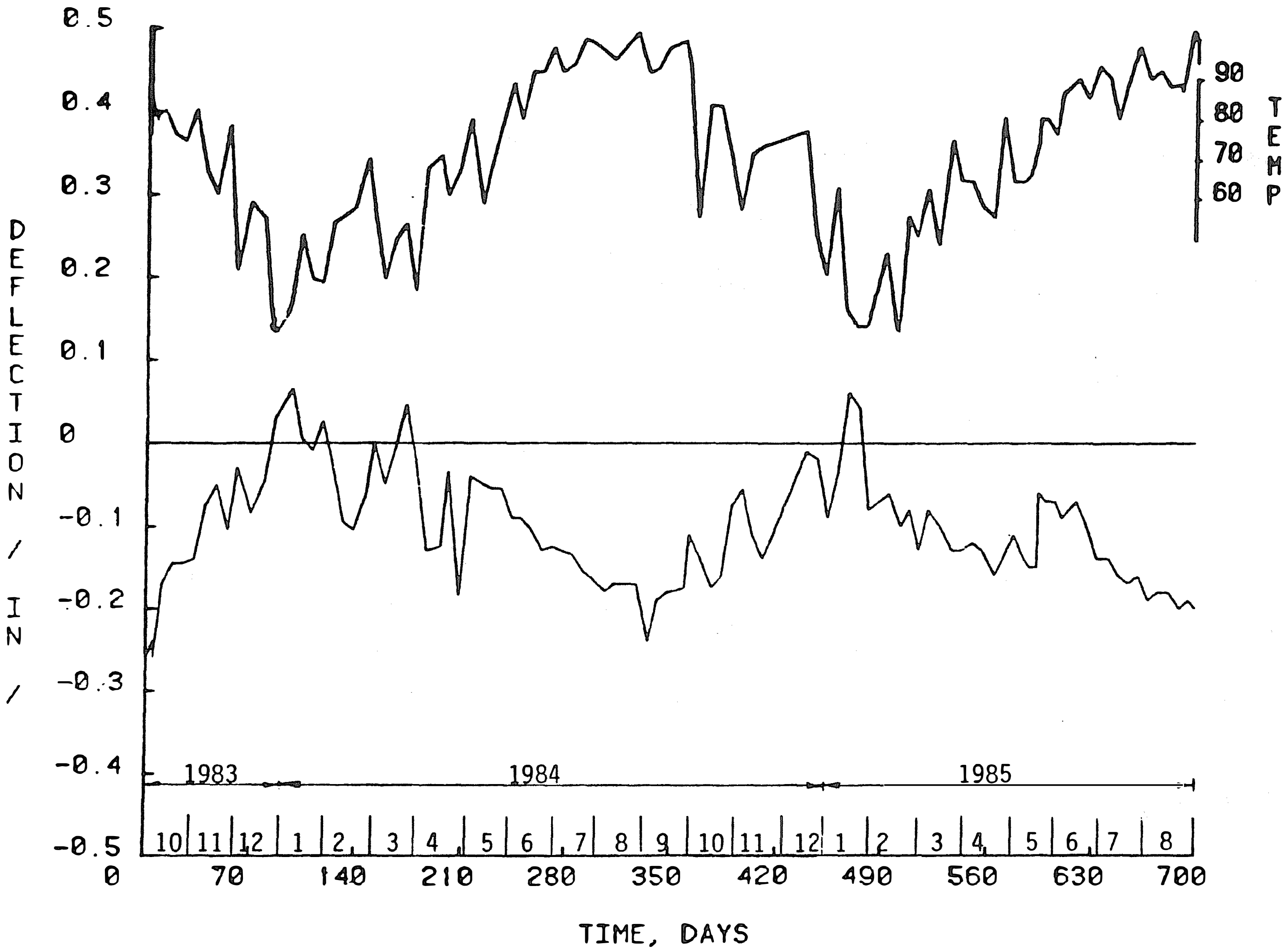


Figure 2.2 Change in Vertical Deflection vs. Time, Phase IV Sustained Loading

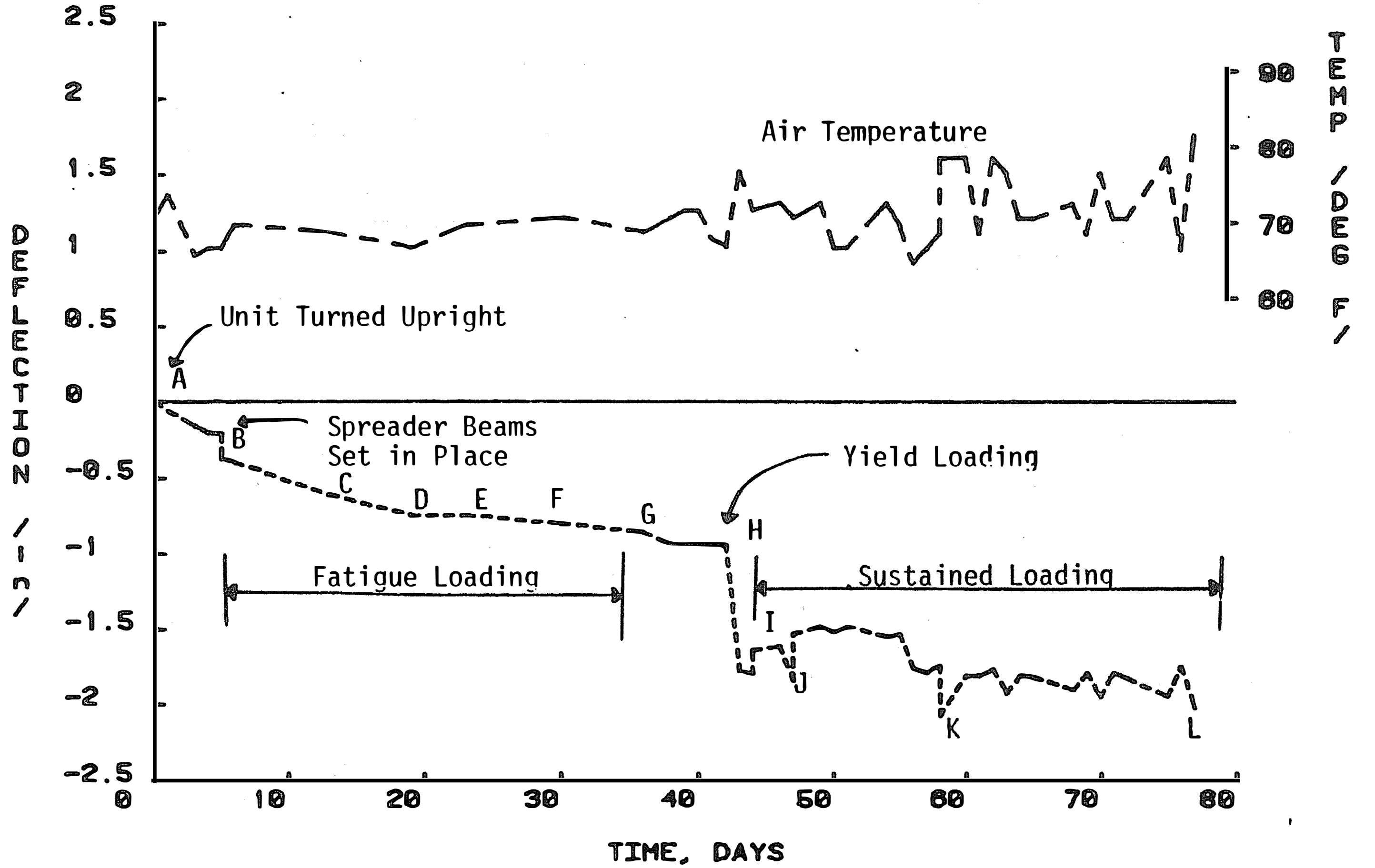


Figure 2.3 Change in Vertical Deflection vs. Time, Phase XI Sustained Loading Effects on Second Unit

4. The sustained loading induced relatively minor increases in bottom flange stresses; however, these increases reduce the yield strength of the bridge units.

5. The effective concrete elastic modulus method [2] for determining increased flange stresses and camber losses due to sustained loading reasonably predicted the measured behavior. This method resulted in a predicted increase in bottom flange stress of 3.8 ksi for the first unit, and 3.0 ksi for the second unit, as compared to measured values of 5.4 ksi and 3.8 ksi for the respective units. The predicted camber losses were 0.61 in. for the first unit and 0.58 in. for the second unit, versus measured values of 0.40 in. and 0.94 in. for the respective units.

6. Branson's method for estimating combined shrinkage and creep effects, as described in Reference 3, resulted in accurate predictions of bottom flange stress change in both units and in camber loss in the second unit. Camber loss in the first unit was overpredicted (see Figure 2.4). The predicted flange stress changes were 4.9 ksi for the first unit and 3.1 ksi for the second, versus 5.4 ksi and 3.8 ksi measured stress changes in the respective units. The predicted camber losses were 1.61 in. for the first unit and 0.96 in. for the second unit, and the measured sustained loading camber losses were 0.4 in. for the first unit and 0.94 in. for the second unit.

7. Branson's method [3] for estimating sustained loading effects was extended in Reference 1 for prediction of creep effects alone (without shrinkage). The extension gave qualitatively correct predictions of flange stress changes and camber loss. The predicted changes in bottom flange stress were 4.9 ksi for the first unit and 6.2 ksi for the second unit. The measured changes were 5.4 ksi and 3.8 ksi for the respective units. The predicted losses

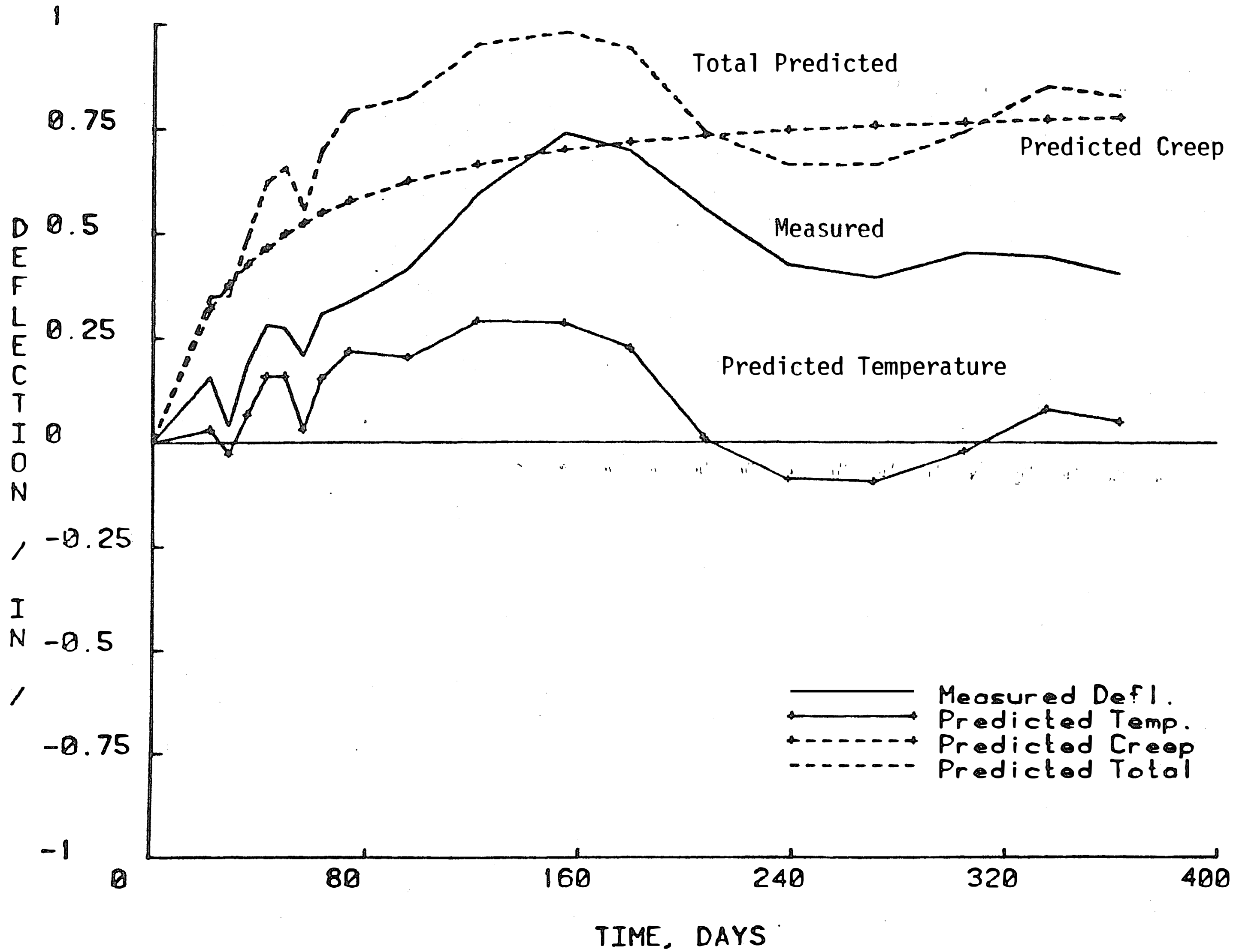


Figure 2.4 Comparison of Measured Midspan Deflection Due to Shrinkage, Creep, and Temperature Effects with Predictions by Branson Method, Phase I Sustained Loading

of camber were 0.8 in. for the first unit and 0.92 in. for the second unit, versus 0.4 in. and 0.94 in. measured for the respective units.

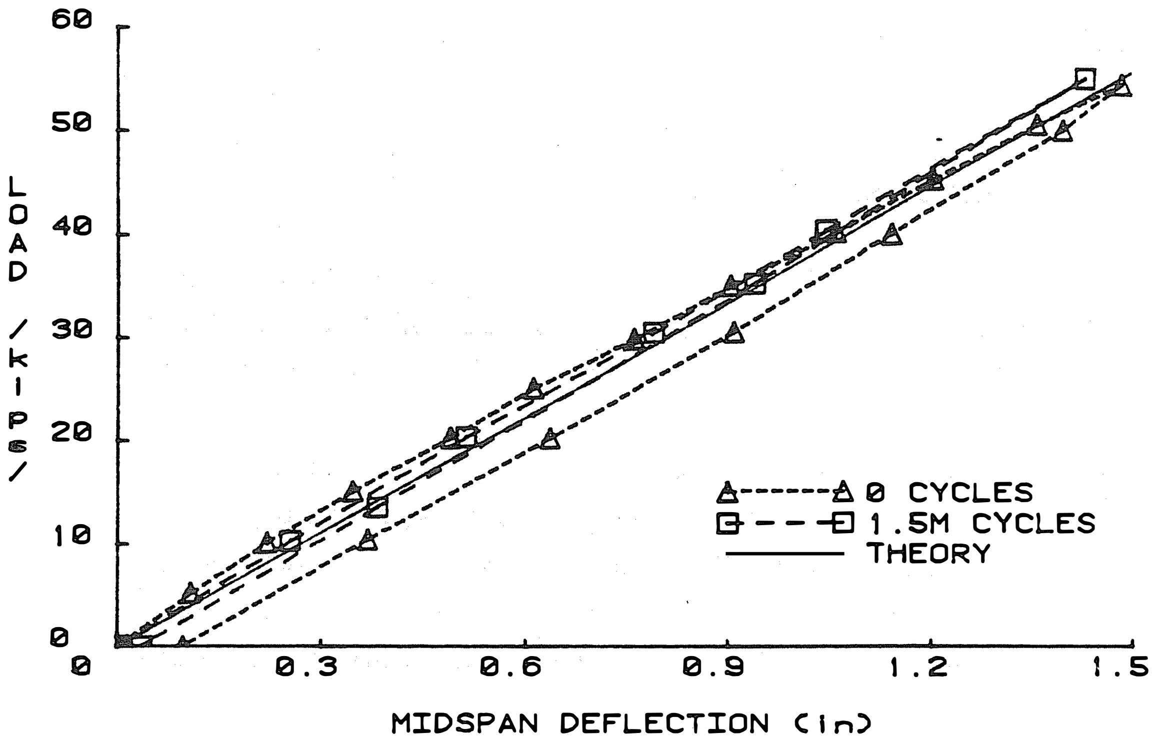
2.1.2 Fatigue Loading Tests

In the fatigue loading test phases, the first unit was subjected to 2,100,000 cycles of simulated AASHTO HS-20 truck loading and 2000 cycles of HS-30 truck loading. Of the HS-20 cycles, 2,000,000 cycles were applied symmetrically with respect to the longitudinal centerline of the unit, and 100,000 cycles were unsymmetrical with respect to this centerline. The second unit was subjected to 500,000 cycles of HS-20 loading. The following observations were made concerning the fatigue characteristics of the bridge units tested:

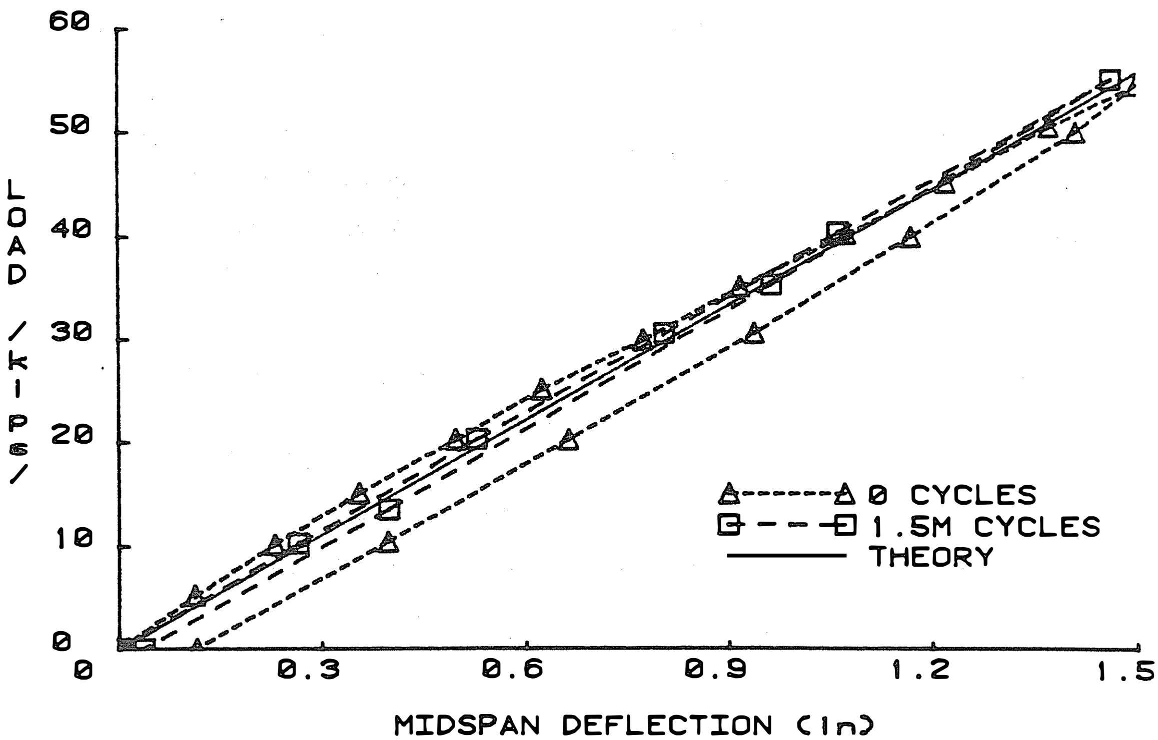
1. After 2,000,000 cycles of repeated loading and before the 100,000 cycles of unbalanced fatigue loading were applied, the first unit did not exhibit significant changes in stiffness (as shown in Figure 2.5, which is a plot of load vs. midspan deflection for the second series of fatigue loadings). Also, slip at the shear connectors was insignificant.

2. The first unit developed cracks along three interior cross-frame welds during the unbalanced fatigue loading tests. However, the unit was designed for 100,000 cycles of loading and the stress range at the welds was higher than allowed by AASHTO for a 2,000,000 cycle design life, which the unit exceeded.

3. The second unit was subjected to 500,000 cycles of repeated loading with no observed changes in stiffness, strength, or slip at the shear connectors (see Figure 2.6).

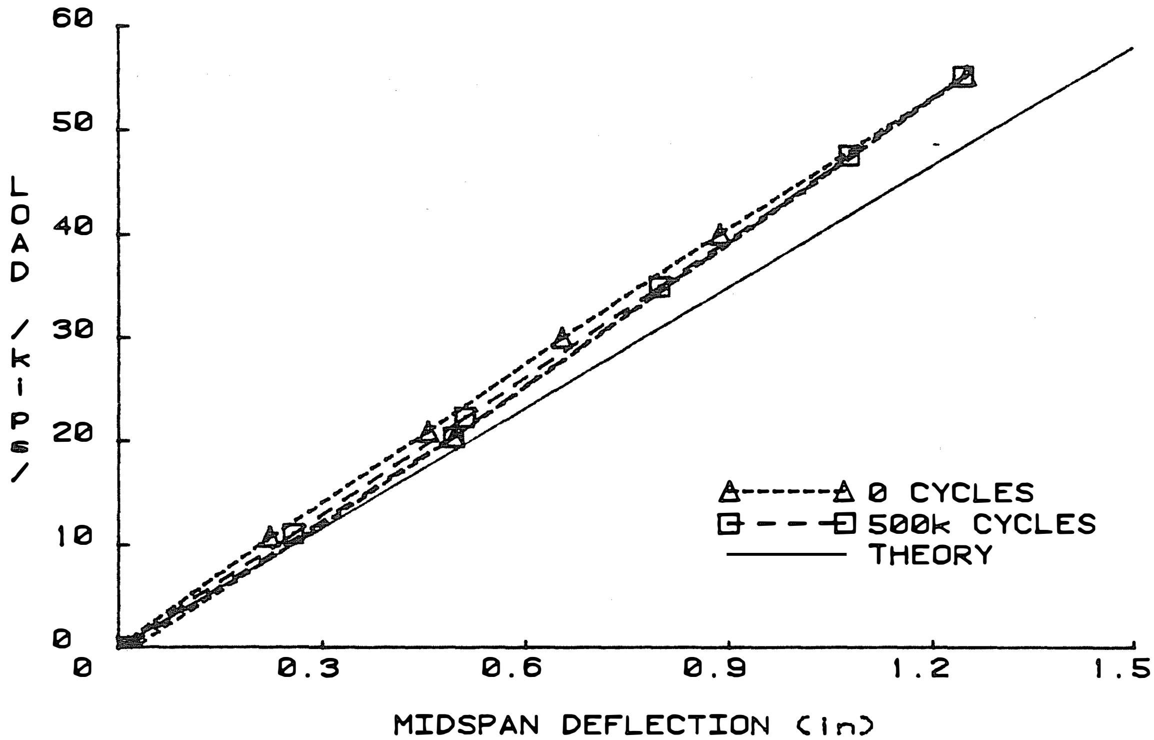


(a) East Beam

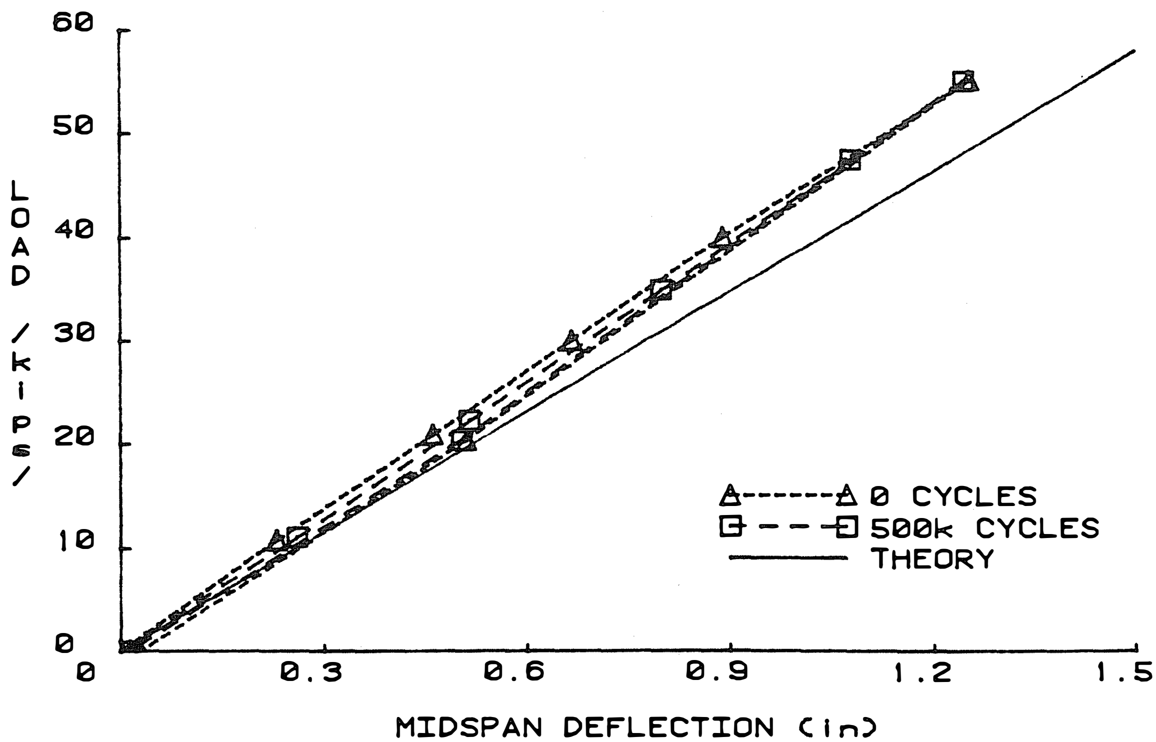


(b) West Beam

Figure 2.5 Load vs. Midspan Deflection, Phase V Repeated Loading



(a) East Beam



(b) West Beam

Figure 2.6 Load vs. Midspan Deflection, Phase IX Repeated Loading of Second Unit

2.1.3 Static Loading Tests

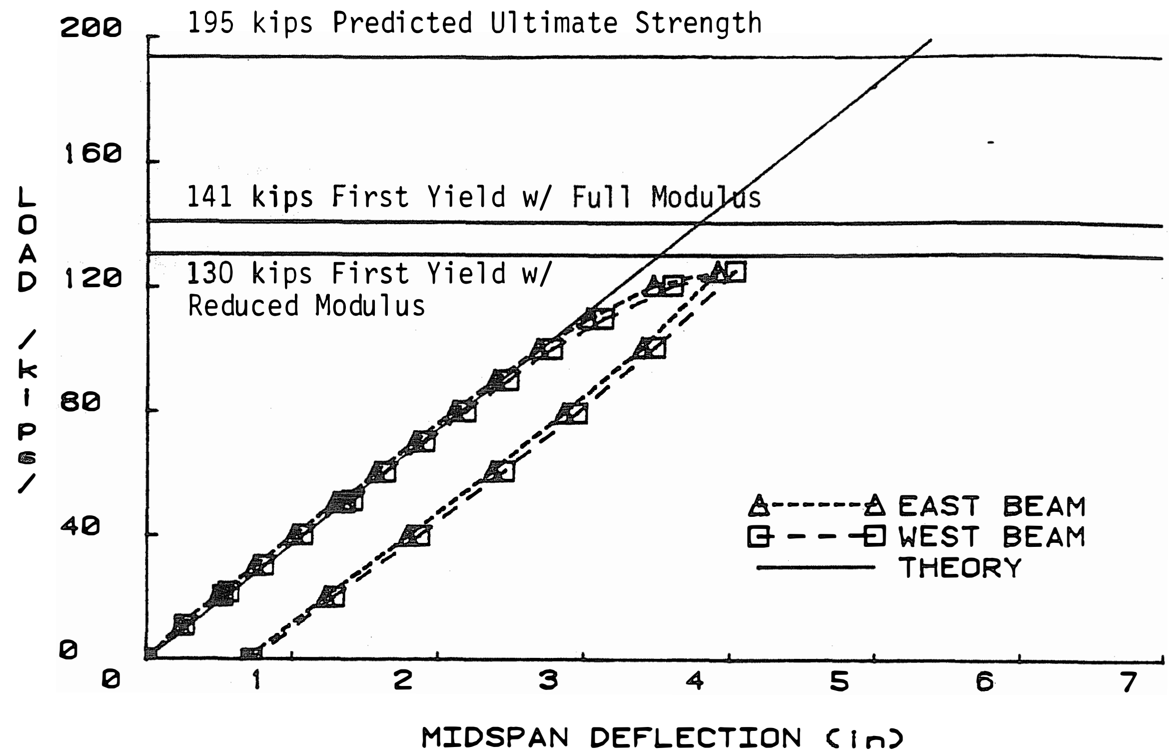
In the static test phases, the first unit was subjected to one HS-30 overload cycle after the first 500,000 HS-20 fatigue loading cycles and was loaded to failure after completion of all the fatigue loading phases. After the 500,000 fatigue loading cycles were applied, the second unit was loaded to determine its yield point. In addition, a static cycle test was conducted after each 50,000 cycles of fatigue loading. The following observations are drawn from the static loading test results:

1. Unit stiffness and stresses are predictable by classical elastic flexure theory if experimentally obtained material properties are used (see Figures 2.6 and 2.7).

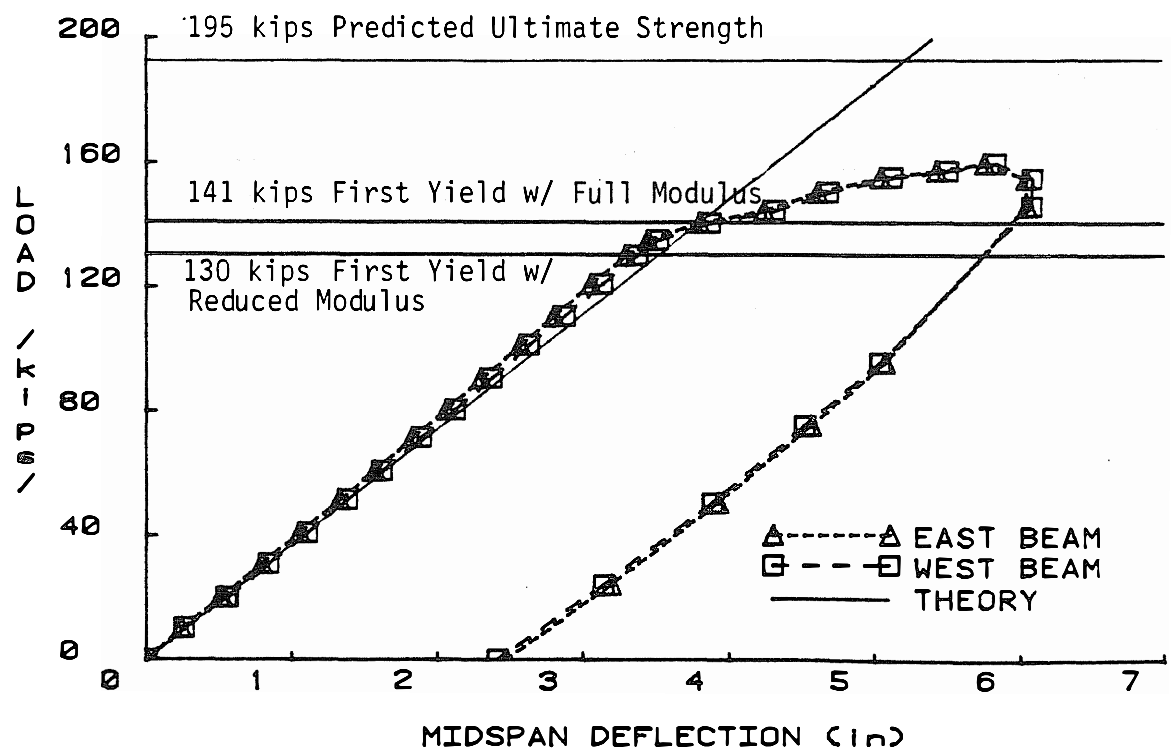
2. The experimental concrete modulus of elasticity, obtained using four year old cylinders, was very close to the AASHTO prediction of concrete elastic modulus based on the 28 day concrete strength. This indicates that the modulus of elasticity of concrete does not increase over time as does compressive strength. As a result, the stiffness of first bridge unit remained constant during the four year testing program.

3. Prestress losses reduce the yield capacity of the units. The losses in bottom flange prestress due to sustained loading effects were 5.4 ksi for the first unit and 3.8 ksi for the second unit. Due to accumulated error in estimating prestressing load magnitudes which directly affects prestress levels, the bottom flange of the second unit had an additional 2.4 ksi less prestress than specified in the design.

4. The first unit reached 94% of its predicted yield moment, which was computed considering the theoretical loss



(a) First Test



(b) Second Test

Figure 2.7 Load vs. Midspan Deflection, Phase VIII Flexural Failure Test in prestress noted above. The unit reached 84% of its ultimate moment before fracture occurred at a welded flange repair (see Figure 2.7).

5. The second unit reached 90% of the calculated yield moment. Part of this apparent undercapacity is due to differences in estimated and actual prestressing loads, and the rest resulted from the under-prediction of sustained loading effects, differences between actual and measured flange yield strengths, and observed slip at shear connectors (see Figure 2.8).

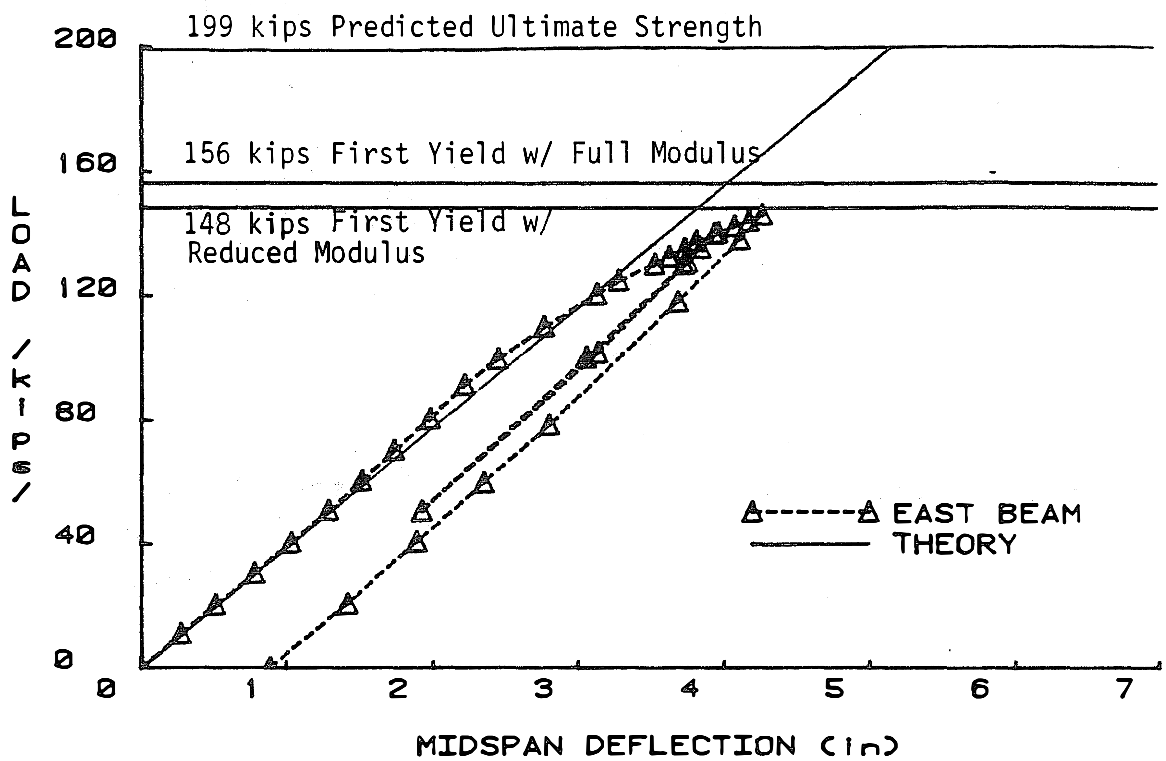
6. The yield strength of the unit is dependent upon the level of prestress in the bottom flange at the time of loading, which is a function of the magnitude of prestressing loads and prestress losses due to sustained loading effects. For optimum design, prestressing loads which result in the highest AASHTO allowable flange stresses should be used, and these loads should be applied accurately. Prestress loss due to sustained loading effects is predicted reasonably well by the effective concrete elastic modulus method. Branson's method is qualitatively correct, but is dependent upon assumed ultimate concrete creep and shrinkage strains which are not always predictable.

7. To account for construction inaccuracies in developing the calculated prestress only 85% to 90% of the calculated yield load is recommended for design.

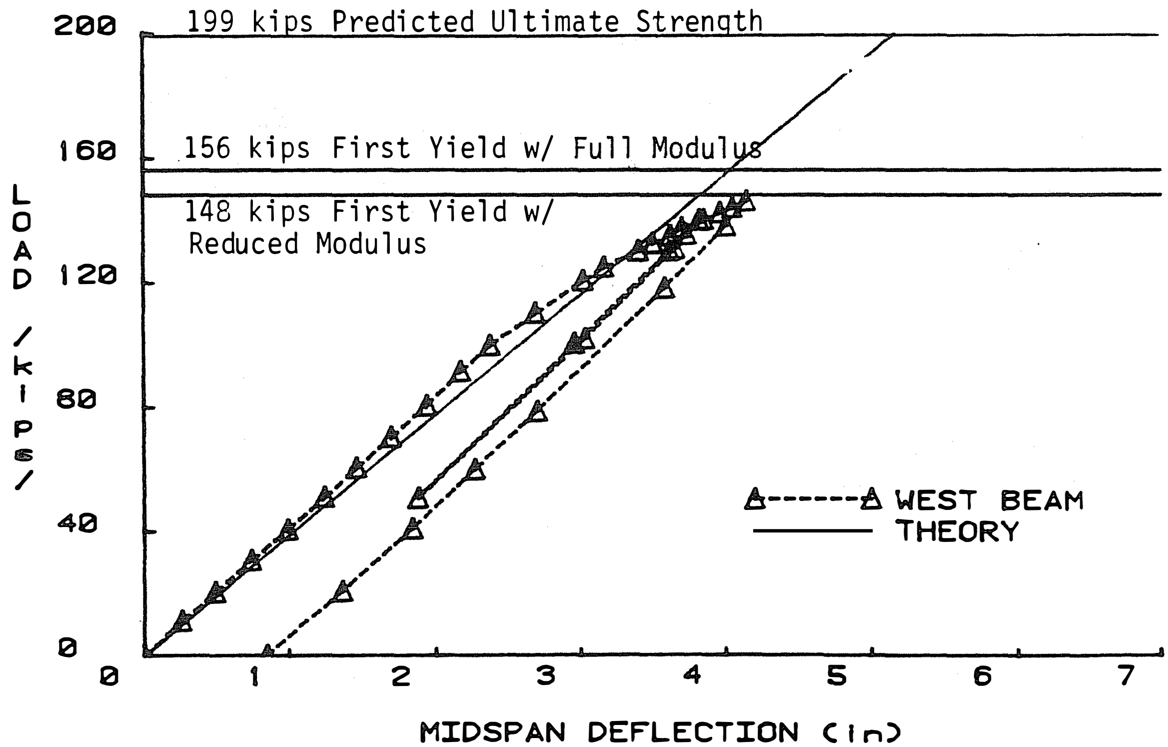
2.2 Supplementary Tests

2.2.1 Transverse Slab Strength Tests

In the transverse slab strength tests, the first bridge unit concrete slab was failed at six locations by the



(a) East Beam



(b) West Beam

Figure 2.8 Load vs. Midspan Deflection, Phase X Flexural First Yield Test of Second Unit application of a concentrated load. All bridge unit deck failures were by sudden punching of the concentrated load through the deck. Six simply supported, square, control slabs of the same transverse dimension and reinforcement ratios as the bridge deck were tested under similar loading conditions. The failure modes of the slabs ranged from ductile flexural failure to sudden punching failure, depending on the reinforcement ratio.

The relative fixity of the bridge unit slab in both longitudinal and transverse directions caused the limiting strength of the slab to be governed by punching strength rather than flexure strength, regardless of the reinforcement ratio.

This behavior is caused by arching action, a description of which is quoted from Reference 4:

> "A simple explanation of this behavior is that in pure bending of reinforced concrete with small steel proportions, the neutral axes at failure are close to the surface. Thus pure bending is accompanied by extensions of the middle surface. If such deformations are incompatible with the support conditions, collapse with pure bending cannot occur."

Thus, the flexural strength of the bridge unit slabs was increased above that of the smaller control slabs due to edge restraint, in addition to two-way action. While arching action is easily understood qualitatively, closed form mathematical solutions are not readily available due to the actual complexity of the phenomenon.

Observations from the transverse slab strength tests are as follows:

1. Due to the degree of axial boundary restraint provided by the slab/beam connection, the bridge unit slabs

behaved as if fixed boundary conditions existed, rather than simply supported conditions, and all failed in punching. The degree of restraint is indicated in Figures 2.9 and 2.10. These figures show the applied load vs. concrete top fiber effective strain and the applied load vs. slab displacement measured between supports for the bridge unit slabs (denoted B5 and B6), and for the small control slabs (denoted S5 and S6), both with the medium (0.29%) reinforcement ratio. The relatively soft curves for the control slabs, which were allowed to rotate and translate at their supports, are indicative of the more ductile bending failure mode observed for these slabs. Whereas, the curves for the bridge unit slabs show the very stiff behavior of the slabs which caused sudden punching failure at much higher loads.

2. The strength of both slab types increased almost linearly with increase in slab transverse reinforcement ratio for the range of ratios tested (see Figure 2.11). Thus, the flexural and punching shear capacity are believed to be interdependent.

3. The control slabs with the smallest reinforcement ratio (0.19%) failed in flexure, while the slabs with medium reinforcement ratio (0.29%) failed in combined flexure and punching. Even though the control slabs were simply supported, the slabs with the largest reinforcement ratio (0.57%) failed by punching. Thus, increased reinforcement caused the failure mode to change from purely flexural to punching, with the possibility that arching action is caused by internal, as well as, external lateral restraint.

4. Punching capacity predicted using AASHTO rules is a conservative lower limit strength for the bridge unit slabs tested, as shown in Figure 2.11.

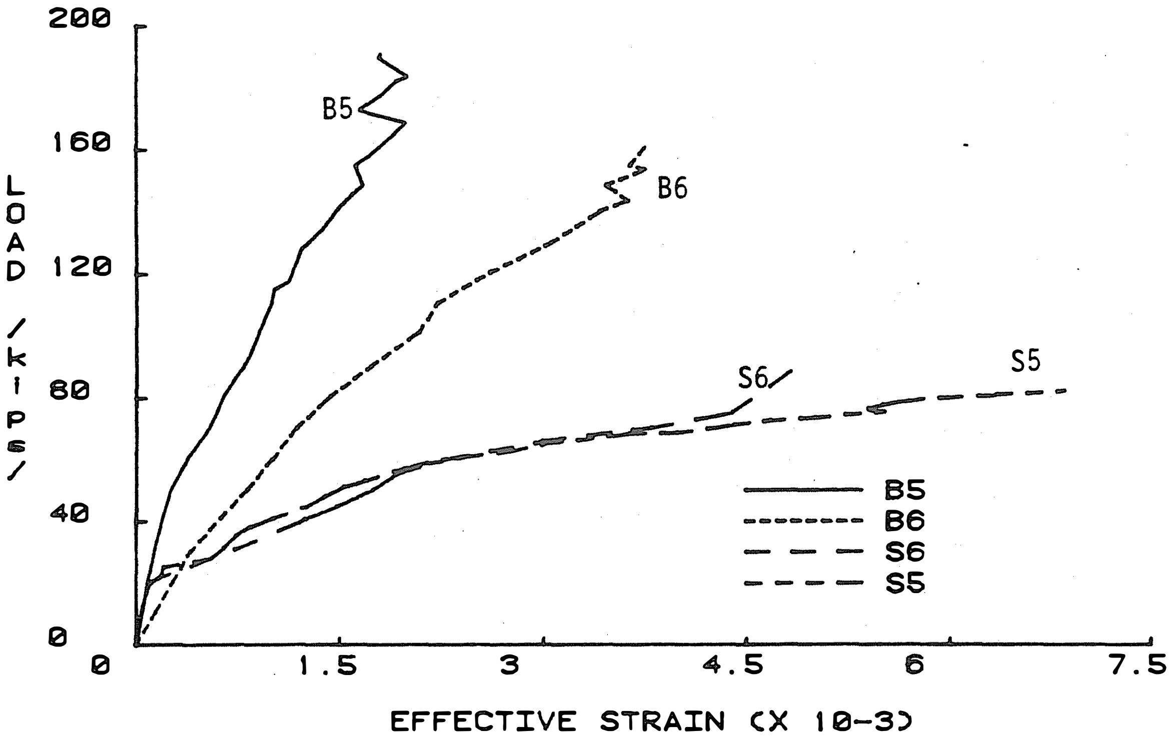


Figure 2.9 Load vs. Effective Strain for Slabs with 0.29% Reinforcement Ratio, Phase XII Transverse Slab Strength Tests

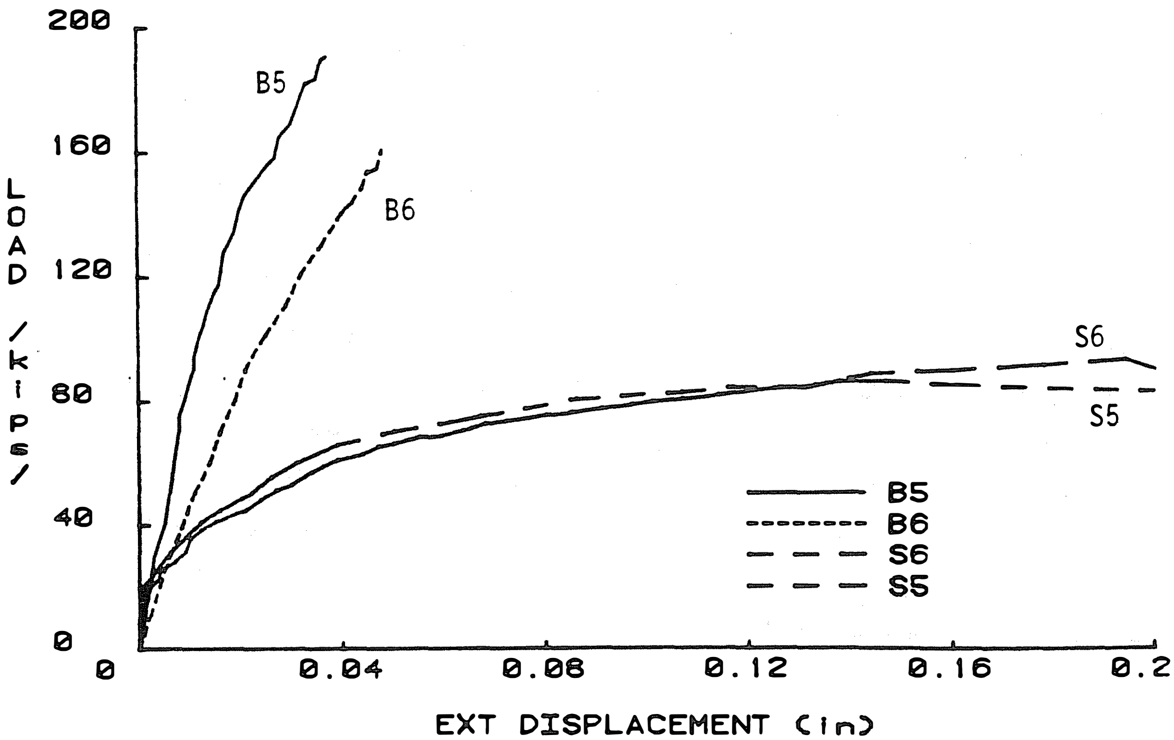
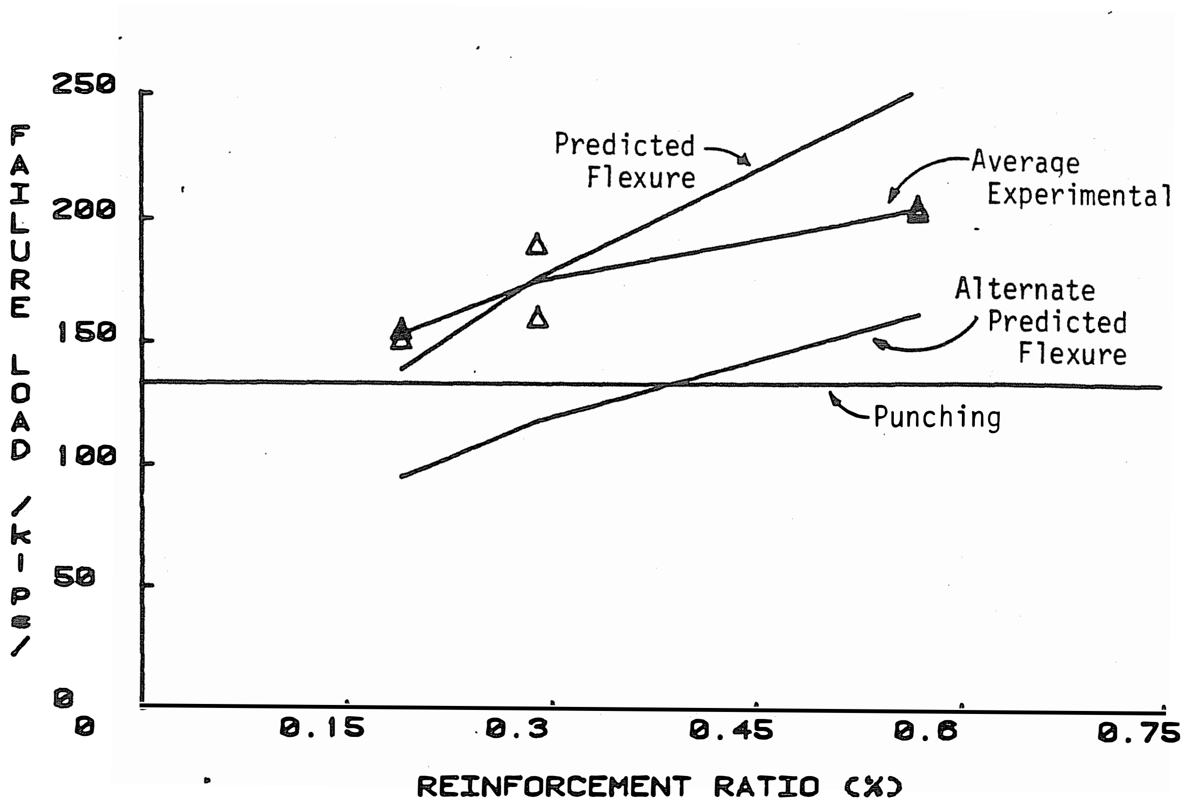
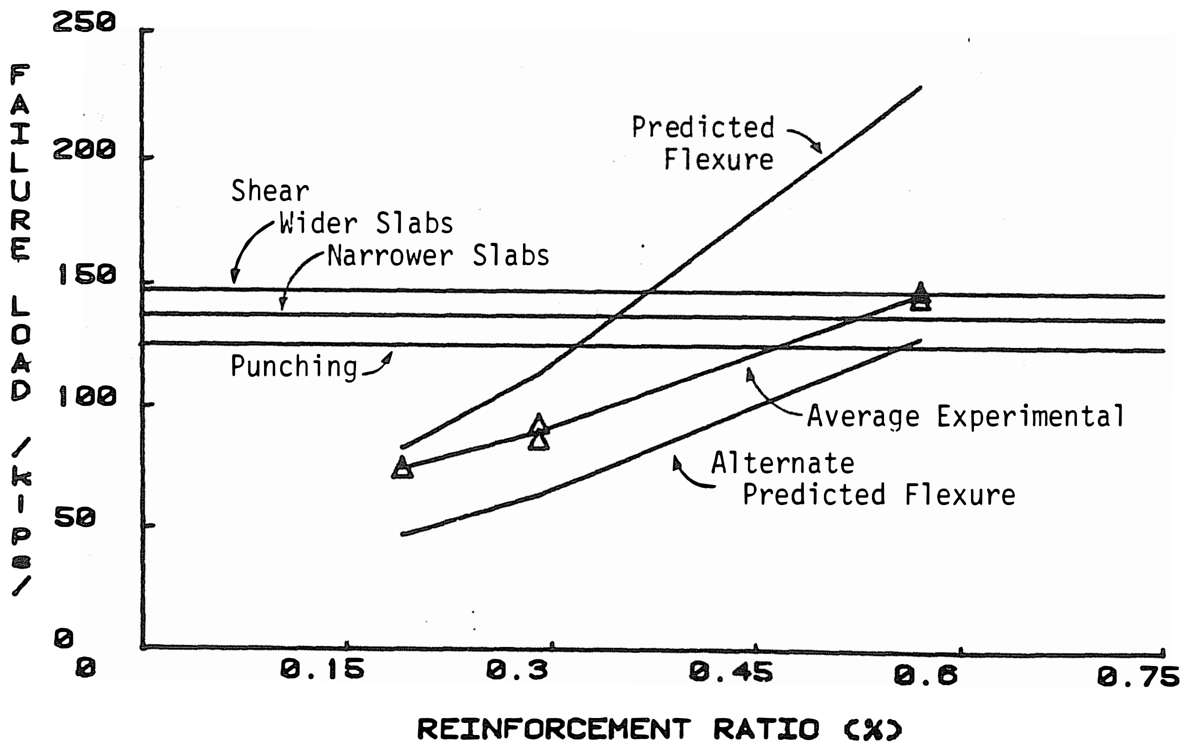


Figure 2.10 Load vs. Restraint Displacement for Slabs with 0.29% Reinforcement Ratio, Phase XII Transverse Slab Strength Tests



(a) First Bridge Unit Slabs



(b) Small Control Slabs

Figure 2.11 Failure Load vs. Slab Transverse Reinforcement Ratio, Phase XII Transverse Slab Strength Tests

5. Predicted slab strengths in flexure were determined using yield line theory. The yield patterns assumed provided failure loads which bracketed the experimental failure loads. These loads are labeled "predicted" and "alternate predicted flexure" in Figure 2.11.

6. In the design of bridge unit slabs, conservative strengths in punching and flexure are obtained from the punching equation given in the AASHTO Specification [2], and from yield line analysis. However, several yield line solutions must be developed so that a least upper bound solution is obtained.

2.2.2 Shear Connector Specimen Tests

During the initial sustained loading period of the first unit, it was surmised that creep at welded stud shear connectors would be greater than at channel shear connectors because of the difference in aspect ratio. To study this hypothesis, four pushout-type specimens were constructed of similar materials as the bridge unit (see Figure 1.7). Two specimens had channel connectors and two had welded stud connectors identical to those used in the bridge units.

Each specimen was loaded for 810 days under 48 kips sustained loading, so that creep and slip could be observed. After this sustained loading period, the specimens were loaded to failure to quantify the strength of the shear connectors. Observations from the shear connector tests are as follows:

1. During sustained loading, slip was slightly higher at the channel connectors than at the stud connectors (see Figure 2.12), but no distinct differences were found between the stud connector specimens and the channel

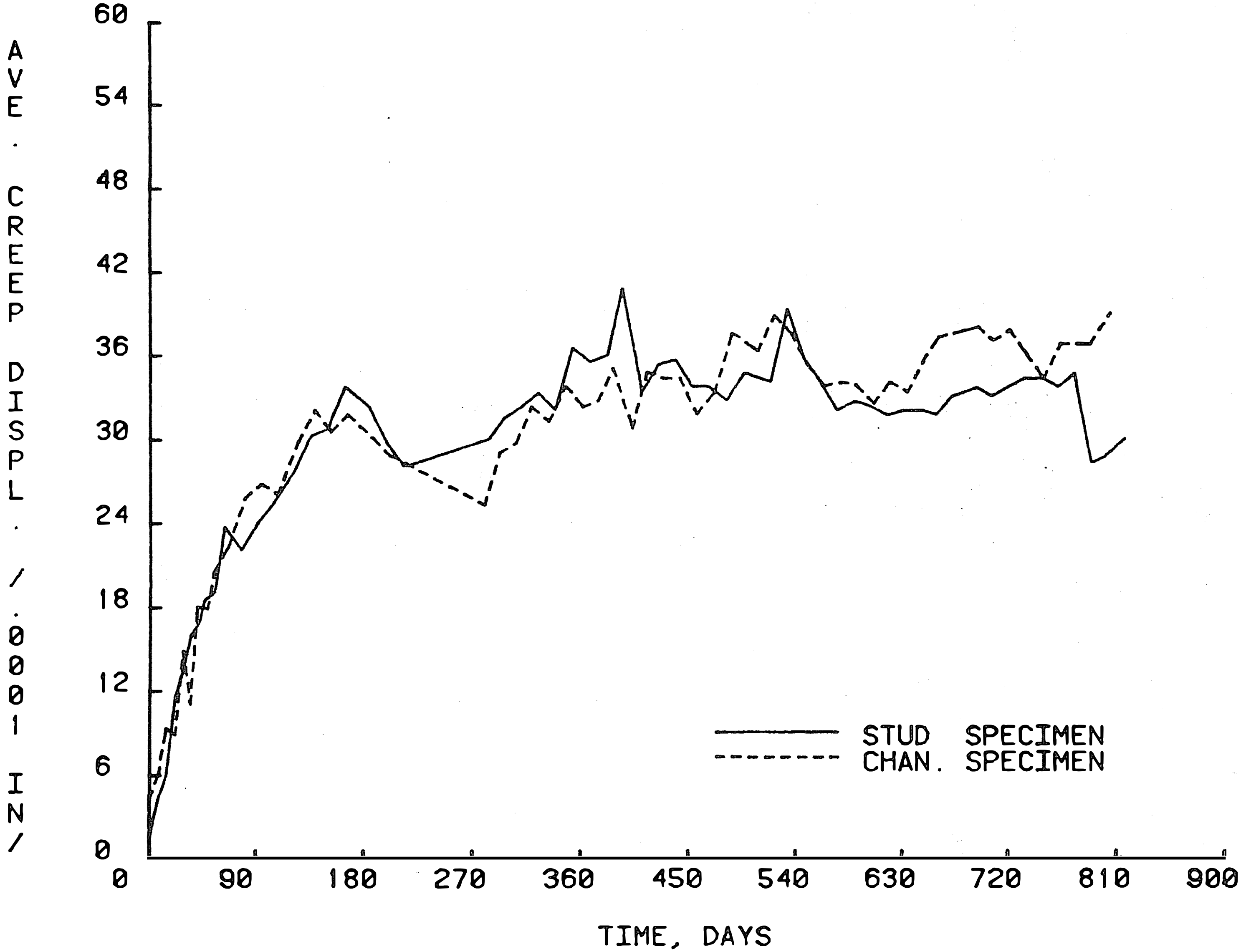


Figure 2.12 Comparison of Average Creep Values At Shear Connectors, Phase XIII A Sustained Loading of Shear Connector Specimens

connector specimens. However, the flanges of the beam sections were slightly embedded in the concrete and the resulting effects are unknown.

2. In ultimate strength tests, the channel and stud shear connectors failed by shear of the steel cross-section, with little damage to the surrounding concrete. Thus, as was noted in Reference 5, the strength of stud shear connections used in concrete of strengths greater than 4000 psi may not be limited by the concrete strength, but by connector strength itself. However, the AASHTO Specification does not consider failure of a shear connector without adjacent concrete crushing. This assumption may result in unconservative shear connector design, when high strength concrete is used in composite girders. Table 2.1 shows the experimental and predicted shear connector strengths.

3. The strength of the channel shear connectors was accurately predicted by AASHTO rules, using the 28 day concrete compressive strength.

4. The strengths of the stud shear connectors were also predictable by AASHTO rules if a limiting concrete compressive strength of 4000 psi is used.

5. The 28 day compressive strength used in the shear connector capacity equation provided by AASHTO should possibly be limited to 4000 psi. Based on the test data of this study, this limitation will result in an accurate estimate of stud type connector strength and a conservative result for channel type connectors.

Table 2.1

Experimental and Predicted Ultimate Strengths of Shear Connectors

(a) Stud Type Shear Connectors

Ultimate Load per Connector (kips)	
Experimental:	
Specimen #1	25.1
Specimen #4	25.9
Predicted:	
AASHTO Eqn. 10-66	34.8
AASHTO Eqn. 10-66*	27.9

*Results for $f'_c = 4000$ psi

(b) Channel Type Shear Connectors

Ultimate Load per Connector (kips)	
Experimental:	
Specimen #2	57.8
Specimen #3	68.8
Predicted:	
AASHTO Eqn. 10-65	57.7

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