Short Span Prestressed Steel Bridges

THOMAS A. DENSFORD, THOMAS L. HENDRICK and THOMAS M. MURRAY

INTRODUCTION

The concept of prestressed steel flexural members was extensively studied in the 1960s[1]; however, the actual use of such members for bridge construction is a recent development in the United States. The recently completed Bonners Ferry Bridge in Idaho and the Muddy Boggy River Bridge in Oklahoma are two examples of cable-stressed plate girder bridges. The INVERSET system of fabricating small river crossing bridges, which is gaining wide use in Oklahoma and other southern states, is another example of prestressed-steel bridge construction. The principal advantage of prestressing is, of course, a reduction in material quantities needed. Table 1 provides a comparison of structural steel, reinforcing steel and deck concrete requirements for three standard bridge designs used by the Oklahoma Department of Transportation. The quantities are for a simple span, 55 ft. 0 in. long, 26 ft. 0 in. wide roadway bridge. Non-composite, composite and prestressed optimized designs are compared. The advantages of prestressing are obvious.

This paper reviews the available methods for prestressed-steel concrete slab bridge construction, discusses advantages and disadvantages of the method of construction, and presents some test results concerning shrinkage, creep and fatigue of a prestressed-steel concrete slab bridge unit.

METHODS OF PRESTRESSING

At least three methods exist to prestress steel beams.[1] One method is to use end-anchored high-strength wires or bars. A second method is to stress components of hybrid beams. And the third method is to cast a concrete slab, in composite fashion, to a deflected beam.

Figure 1 illustrates the method of prestressing by end-anchored wires or bars. Beams with tensioned rods, Fig. 1(a), have been used with many older bridges. It consists of a trussed beam with a turnbuckle, or a steel or wrought iron

Table 1.
55'-0" Bridge, 26'-0" Roadway
Comparison of Quantities

	Steel Stringers (lbs)	Reinforcing Steel (lbs)	Deck Concrete (cy)
I-Beams Non-Composite	51,920 (A36)	6535	35.7
I-Beams Composite	29,700 (A36) 25,520 (A588)	9310	44.4
Pre-Stressed Composite	18,150 (A588)	6412	32.4

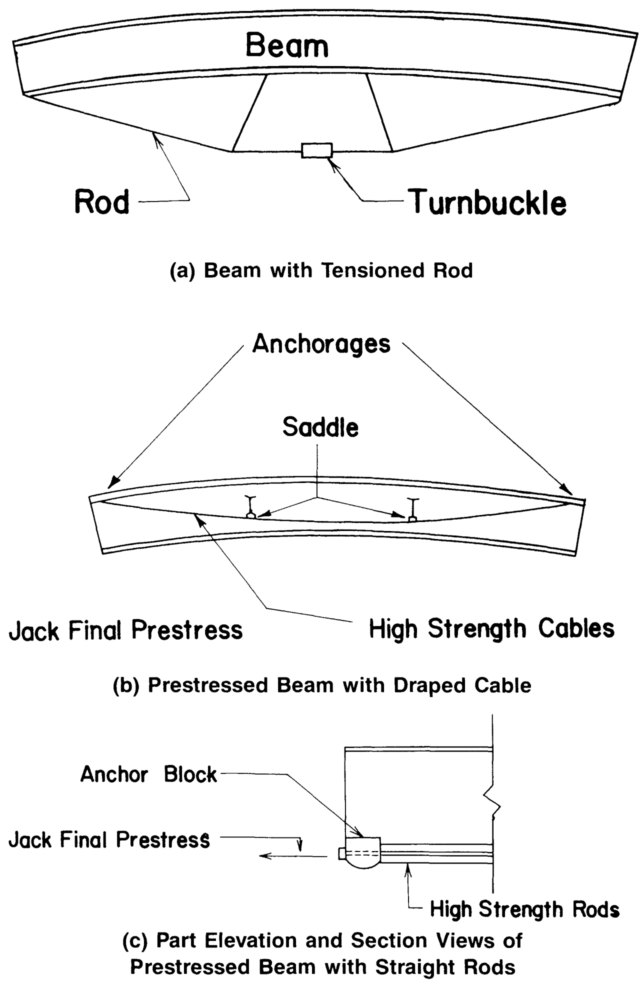


Fig. 1. Method of prestressing by end-anchored with straight rods.

Thomas A. Densford is structural engineer, Scanlon and Associates, Santa Fe, NM.

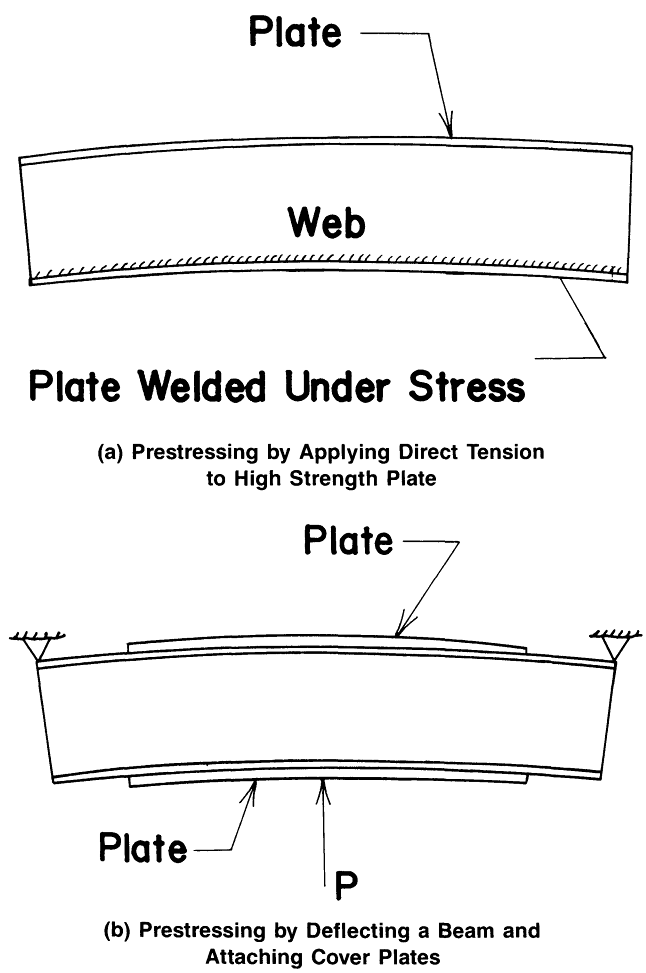
Thomas L. Hendrick is structural engineer, Wallace Engineering, Structural Consultants, Tulsa, OK.

Thomas M. Murray is Montague-Betts Professor of Structural Steel Design, Virginia Polytechnic Institute and State University, Blacksburg, VA, formerly Professor-in-Charge, Fears Structural Engineering Laboratory, University of Oklahoma, Norman, OK.

rod. The rod is fastened to each end of the beam and exerts a negative moment primarily by means of inclined struts located near the third points. A second version is shown in Fig. 1(b). This arrangement is considerably more convenient in that the cables are contained within the outside dimensions of the beam. A sectional and part-elevation view of a wide flange beam which has been prestressed by two straight high-strength rods reacting against end bearing blocks is shown in Fig. 1(c). This is basically the method used for the Bonners Ferry and Muddy Boggy River bridges.

The second method of prestressing steel members is stressing components of hybrid beams. Two possible means of fabricating such beams are shown in Fig. 2. The first is similar in principle to that used in pretensioned prestressed concrete as shown in Fig. 2(a). A direct tensile force is applied to the high strength plate. While the tension is maintained, the plate is welded to an unstressed T-section of structural steel. The force in the plate is released upon completion of the welding and the prestressing is completed. The second means is shown in Fig. 2(b). A structural steel I-beam is deflected in loose contact with one or more high-strength cover plates. While the system is maintained in a deflected position, the cover plates are welded to the flanges of the beam. Subsequent release of the jacking load results in the desired prestress.

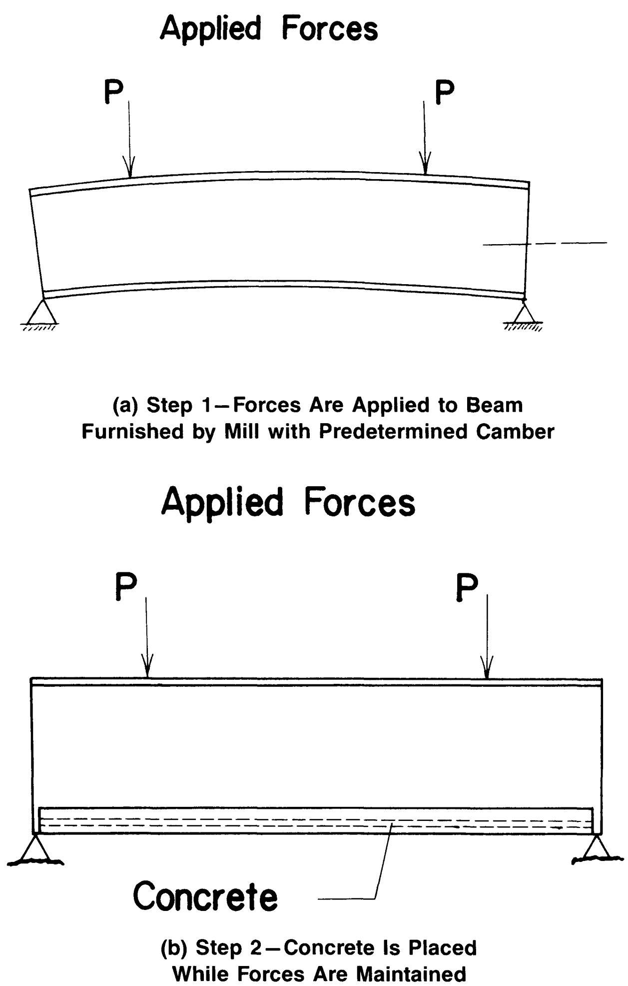
Figure 3 illustrates the third method of prestressing, whereby a steel beam is first deflected and then a concrete slab is cast against the beam. In Fig. 3(a), the forces are applied in a downward direction. In Fig. 3(b), a concrete slab is shown cast in composite fashion with the lower portion of the beam. Following curing, the applied forces are removed and compressive forces are locked into the concrete. If the applied load is supplied by external jacks, the procedure is known as the "Preflex Technique" which has been patented by the Preflex Corporation of America. The INVERSET system (also patented) uses the weight of a heavy steel form and the concrete itself to supply the applied prestressing load.



(a) Prestressing by Applying Direct Tension to High Strength Plate

(b) Prestressing by Deflecting a Beam and Attaching Cover Plates

Fig. 2. Method of prestressing by stressing components of hybrid beams.



(a) Step 1—Forces Are Applied to Beam Furnished by Mill with Predetermined Camber

(b) Step 2—Concrete Is Placed While Forces Are Maintained

Fig. 3. Method of prestressing by using applied loads.

ADVANTAGES AND DISADVANTAGES

The principal advantage of prestressed-steel concrete slab bridges is the reduction in required material. A second advantage exists if the prestressing sequence is such that the concrete is precompressed. One of the major bridge problems in regions with a large number of annual freeze-thaw cycles is concrete deck deterioration. High compressive stresses in the concrete decrease cracks and crack growth and, therefore, reduce deterioration of the deck. Further, if the concrete slab is cast upside-down, "pore holes," which form because of water evaporating from the top of the concrete during curing, are actually facing downward in the completed structure allowing water to drain from the slab.

Disadvantages include possible loss of prestress due to creep and shrinkage of the concrete, and slip at the shear connectors (particularly because of the relatively high and sustained concrete stresses), fatigue, and excessive deflections and vibrations due to the reduced mass and stiffness. To investigate sustained loading and fatigue effects, two full bridge units, constructed using the INVERSET method, were studied in a four year research program at the Fears Structural Engineering Laboratory, University of Oklahoma under the sponsorship of the Oklahoma Department of Transportation. Details of the method used to design the unit and results of the research are briefly discussed in the following sections. More detailed information is found in References 2, 3 and 4.

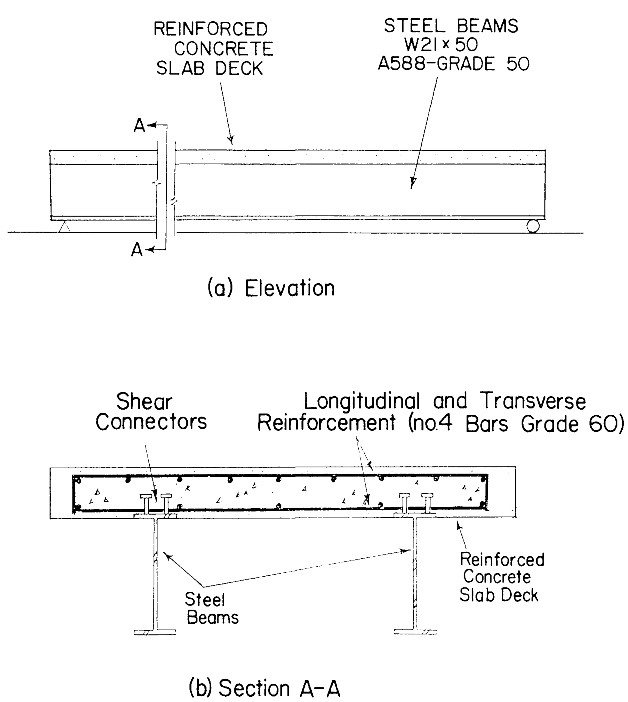


Fig. 4. Typical bridge unit details.

THE INVERSET CONCEPT

The prestressed composite bridge units studied consist of a concrete slab connected by shear connectors to two steel beams as shown in Fig. 4. The units are usually prefabricated and transported to a site. There, 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 primarily for county road bridges, but the possibility of use in state highway bridges exists.

The method of construction of the bridge units is unique and patented. Shear connectors are welded to two steel beams which are then inverted and simply supported above a form containing a mat of concrete reinforcing steel. The concrete forms are then hung from the steel beams as shown in Fig. 5. The bridge deck concrete is then poured into the forms and additional dead load may be applied to the beams to increase the unit deflection to a predetermined level such that the proper prestress level is obtained. When the concrete has cured and the unit is unloaded, forms are stripped, and the unit turned over. The resulting composite beam is similar to a composite beam obtained using shored construction methods, with additional stressing of the steel beam in the direction opposite in-place gravity stresses. The prestressing extends the service load range of the units.

OVERVIEW OF TESTING PROGRAM

A testing program conducted by the authors was divided into the phases shown in Table 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 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 2) and as a consequence, the results of the static flexural test to failure (Phase VIII) are

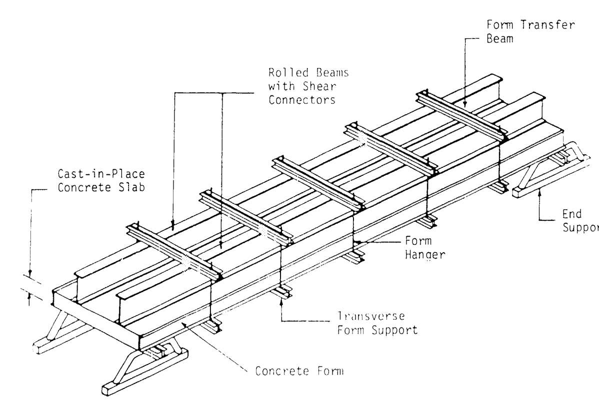


Fig. 5. Method of fabrication of bridge unit.

Phase	Description
	Table 2. Research Phases
	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-20) loading test.
IV.	Two years of observation under sustained loading (totaling three years of sustained loading). The unit was accidently dropped at the end of this phase.
V.	An additional 1,500,000 cycles of repeated (HS-20) loading (totaling 2,000,000 cycles).
VI.	Repeated operating ratings (HS-30) loading of 2,000 cycles. Fatigue cracks were repaired at the end of this phase.
VII.	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.
XIa.	Static flexural test to failure of second unit (conducted by others[6])
	SUPPLEMENTARY TESTS
XII.	Transverse slab strength tests on first bridge unit.
XIII.	Shear connector specimen observation and strength tests.

Table 3. Measured Material Properties

(a) Steel Components

Test Specimen	Component	Yield Stress (ksi)
First Unit	W21×50	56.0
Second Unit	W21×50	58.0
First Unit	#4Gr60	67.2
Second Unit	#4Gr60	79.5

(b) Concrete

Test Specimen	Age at Cylinder Tests (days)	Compression Strength (ksi)	Elastic Modulus (ksi)
First Unit	28	5.30	4394*
	1408	7.40	4365
Second Unit	51	6.45	5335

* *Calculated*

questionable. A second unit was then constructed and used for Phases IX through 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, and on separately constructed shear connector specimens to study possible sustained loading effects for two types of shear connectors. Only results from the primary tests are discussed herein.

Phases I through XI were considered to be primary test phases. 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[4] 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 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 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.

Subsequent to the above program, the second bridge unit was loaded statically to failure by Prasad, Wallace and Bush[16], Phase XIa in Table 2.

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 W21×50 steel beams of A588 Grade 50 steel, connected by 3×3×¼-in. steel angle cross-frame diaphragms, located at the ends and third points of the beams. Pairs of ¾-in. diameter by 4-in. high welded stud shear connectors, spaced along the beam flanges in accordance with the AASHTO Specification[4] were welded to the beams prior to casting the concrete deck. For each unit, a full length, reinforced concrete slab of 6 ft. 9½ in. width was cast against the top flanges of the parallel steel beams. Slab thicknesses were 7½ 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. Figure 4 shows the cross-section of the first test unit and Table 3 gives measured material properties.

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 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 slab 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 References 2 and 3.

OBSERVATIONS

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 Fig. 6).
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 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.
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.
4. The sustained loading induced relatively minor increases in bottom flange stresses; however, these increases reduce the yield strength of the bridge units.

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 were applied, the first unit did not exhibit significant changes in stiffness (as shown in Fig. 7, 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

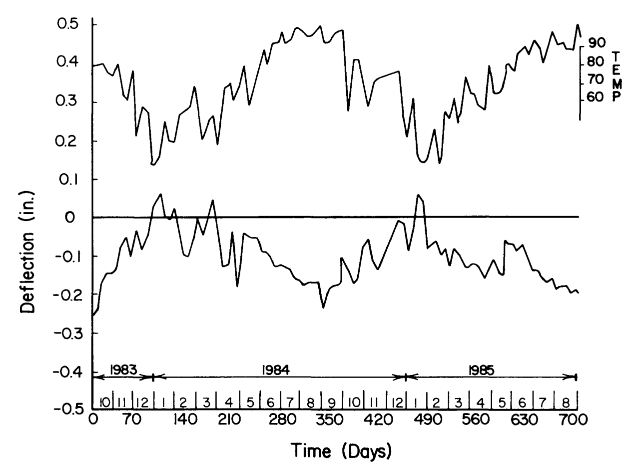


Fig. 6. Change in vertical deflection vs. time.

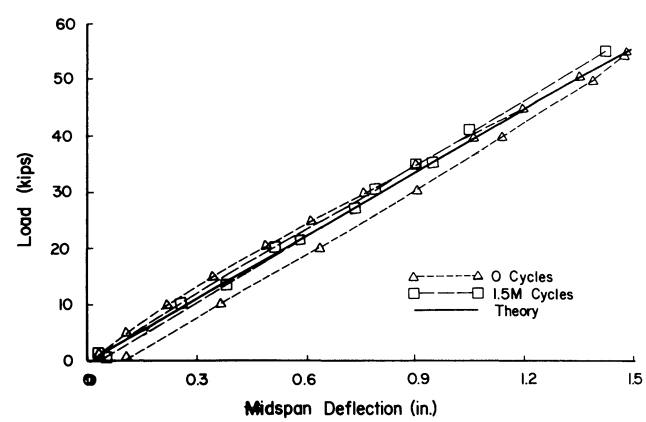


Fig. 7. Load vs. deflection response before and after fatigue loading.

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.

Static Loading Tests

In the static test phases, the first unit was subjected to: (a) one static cycle HS-20 loading after each 50,000 cycles of HS-20 fatigue loading, (b) one HS-30 static overload cycle after the first 500,000 HS-20 fatigue loading cycles, and then (c) statically loaded to failure after the completion of all cyclic and sustained loading tests. The final static test was conducted after the unit had been dropped between Phases IV and V and after fatigue cracks in the tension flanges, which developed at the end of Phase VI, had been repaired. The test was stopped when all of the repaired fatigue cracks ruptured. Results of these tests were compared to predictions based on measured material properties and classical elastic flexure theory, including reduced elastic modulus effects due to creep and shrinkage. The failure load was compared to AASHTO ultimate load predictions. The following observations were then made (References 2, 3 and 4 provide more details):

1. Unit stiffness and stresses are predictable by classical elastic flexure theory if experimentally obtained material properties are used (see Fig. 8).
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 the 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 was 5.4 ksi for the first unit.
4. The first unit reached 94% of its predicted yield moment, which was computed considering the theoretical loss in prestress noted above. The unit reached 84% of its ultimate moment before fracture occurred at a welded flange repair (see Fig. 8).

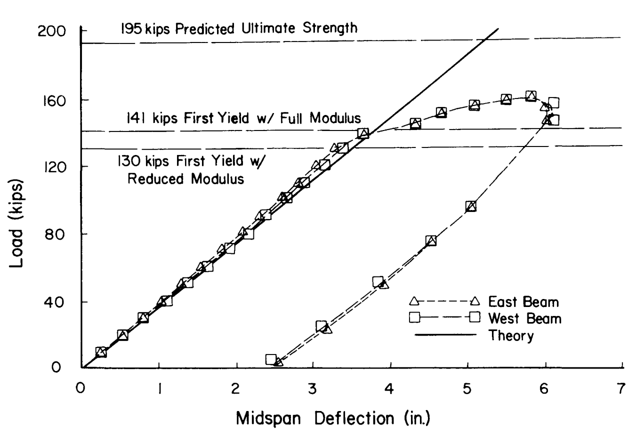


Fig. 8. Load vs. deflection, static test to failure of first unit.

The second unit was subjected to: (a) one static cycle HS-20 loading after each 50,000 cycles of HS-20 fatigue loading, (b) a static test to first yield, and (c) a test conducted by Prasad et al.[5] to flexural failure. Failure was by crushing of the slab at the load application points. Results of these tests were compared to predictions. The following observations were then made:

1. Unit stiffness and stresses are predictable by classic elastic flexure theory to first yield (see Fig. 9).[2,3,4]
2. Due to accumulated error in estimating prestressing load magnitudes which directly affects prestress levels, the bottom flange of the second unit had 2.4 ksi less prestress than specified in the design.[3,4]
3. 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,

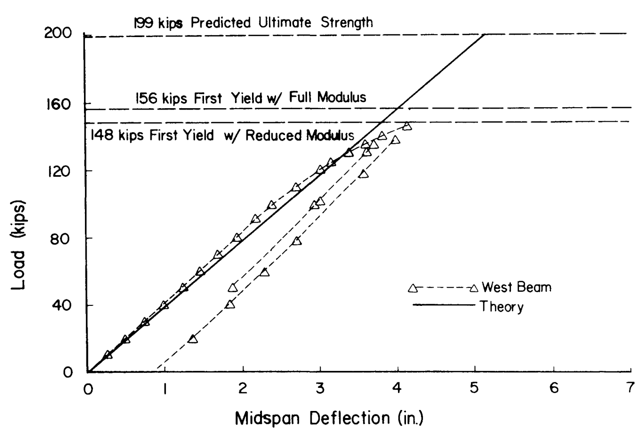


Fig. 9. Load vs. deflection, static test to first yield of second unit.

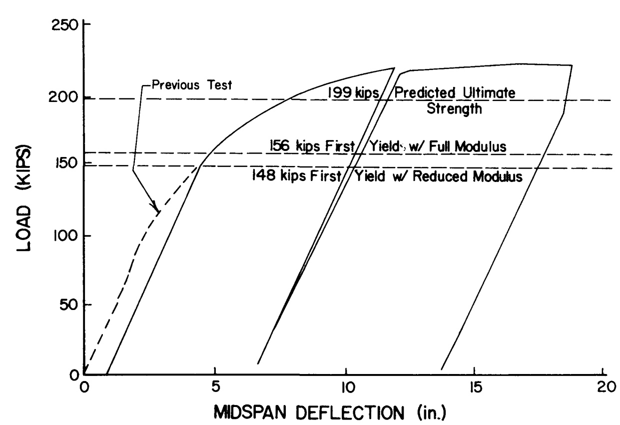


Fig. 10. Load vs. deflection, static test to failure of second unit.

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.[3,4]

4. 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 an effective concrete elastic modulus method.[3,4]
5. From Reference 6, the maximum applied load (224 kips) in the static test to failure was 13% greater than the predicted ultimate load (199 kips). This excess capacity may be due to strain hardening in the bottom flanges of the steel beams.
6. The maximum measured deflection in the final static test was 18.6 in. as shown in Fig. 10. Measured strains indicated that yielding occurred over nearly the full depth of the beam webs.[6]
7. Measured horizontal slips at the slab girder interface were less than 0.6 in. at ultimate load. The maximum slips occurred only in regions where yielding of the girders extended well into the beam webs.[6]
8. The behavior of the bridge units tested can be predicted with commonly used analysis procedures.[2,3,4,6]

CONCLUSIONS AND RECOMMENDATIONS

Short span prestressed steel bridges can provide economical alternatives for rural road and state highway system bridges. From a four year study of two prestressed units, it was found that, even though the prestressing is provided by ordinary mild steel, the units performed satisfactorily when subjected to two and one-half years of sustained loading, 2,000,000 cycles of simulated truck loading, and static overloading to first yield. A subsequent test to flexural failure showed that the ultimate strength of the bridge units can be predicted using standard design procedures.

It is recommended that at design overload, the maximum stress in a prestressed bridge unit not exceed 95% of the beam material yield stress. This limit is affected by prestressing. It is also recommended that a maximum load check be made for the fully composite cross-section. This capacity is unaffected by prestressing and will probably govern most designs.

REFERENCES

1. ASCE-AASHO Subcommittee 3, "Development and Use of Prestressed Steel Flexural Members," *Journal of the Structural Division,* Vol. 94, ST9, September, 1968, pp. 2033–2060.
2. Hendrick, T., C. Clottey and T. Murray, "Experimental Investigation of a Prestressed Bridge Unit, 3rd Progress Report," Report No. FSEL/ODOT 83-02 submitted to the Research Division, Oklahoma Department of Transportation, Fears Structural Engineering Laboratory, University of Oklahoma, December 1983, 147 pages.
3. Densford, T. and T. Murray, "Experimental Study of Two Prestressed Steel Beam-Concrete Slab Bridge Units," Research Final Report, FSEL/ODOT 86-02 submitted to Research Division, Oklahoma Department of Transportation, Fears Engineering Laboratory, University of Oklahoma, August 1986, 260 pages.
4. Densford, T. and T. Murray, "Experimental Study of Two Prestressed Steel Beam-Concrete Bridge Units—Executive Summary," Report No. FSEL/ODOT 87-02, submitted to the Research Division, Oklahoma Department of Transportation, Fears Structural Engineering Laboratory, University of Oklahoma, May 1987, 40 pages.
5. *Standard Specification for Highway Bridges,* 13th Edition, American Association of State Highway and Transportation Officials, 1983.
6. Prasad, R., B. Wallace and J. Bush, "Experimental Determination of the Ultimate Capacity of a Composite Prestressed Bridge Unit," Report No. FSEL/ODOT 89-01 submitted to Research and Development Division, Oklahoma Department of Transportation, Fears Structural Engineering Laboratory, University of Oklahoma, February 1989, 44 pages.