# Final Report

# FATIGUE AND RESIDUAL STRESS INVESTIGATION OF COMPOSITE PRESTRESSED STEEL BEAMS

by

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## ABSTRACT

Two composite, prestressed, steel beams, fabricated by slightly different methods, were fatigue tested to destruction. Stresses and deflections were measured at regular intervals, and the behavior of each beam as failure progressed was recorded. Residual stresses were then evaluated by testing segments of each beam. An attempt was made to assess the effects of the residual stresses on fatigue strength.

## INTRODUCTION

In recent years the failure phenomena known as fatigue has become increasingly important to engineers due to the technological advancements of mankind. The machines and structures which engineers are designing today make increasingly exacting demands on the materials used, particularly in regard to frequency of loading. Therefore it is not surprising to find that large amounts of money and effort are being expended for better understanding of the behavior of materials of construction under repeated loads. The new weldable high strength constructional alloy steels make new concepts in the design of steel structures possible, in that the designer has a choice of materials as well as of geometry.

One interesting and economical method of utilizing these new materials has been developed by engineers of the Iowa State Highway Commission[43]. A prestressing technique was employed to obtain a favorable stress pattern in members composed of rolled beams of A35 steel with "T-1" constructional alloy steel cover plates.

The engineers who developed the method fabricated and analyzed five beams to determine the feasibility of the procedure[5]. Two of these beams simulating those found in the negative moment region of a bridge were later tested under repeated load[46]. The results of the tests of these two beams, especially the state of stress, proved to be enlightening.

The study reported herein was initiated to evaluate further this Iowa State Highway Commission design by fatigue testing two specimens planned for the positive moment region of a bridge. After the specimens had failed under the repeated loading, residual stresses were measured in more than 180 elemental strips which had been removed from the beam. Other means of improving fatigue strength are presented in the Appendix.

### RESEARCH AND THEORIES

Experiments about one hundred years ago first established the failure of materials under repeated loads, and the conclusions reached are valid today, although materials have changed considerably[37]. Since then a great deal of data has been accumulated. But nearly all the data have necessarily been rather specific in nature, and the interpretation of the data has tended to be empirical rather than theoretical.

To aid in the interpretation of the results of the experimental work reported herein, a literature study of the general subject of metal fatigue was undertaken, particularly as related to steel weldments. Much of the basic study of the mechanisms of fatigue failure was found in the field of metallurgy. Extensive applied research in the aeronautical and automotive fields has made a major contribution to the search for better understanding of the fatigue phenomena. Therefore reported research in related fields such as these was reviewed, as as well as that on structural steel beams and connections.

## Theories of Fatigue Failure

Early theories were generally formulated in accordance with theories of elasticity[9]. Failure was assumed to occur as a result of having exceeded some limiting value of stress or deformation. Among these theories were:

1. The theory of maximum stress or the principal stress theory (Rankine)
2. The theory of maximum strain (Saint Venant)
3. The maximum shear-stress theory
4. The distortion-energy theory.

None of these theories has proved to be satisfactory or conclusive in dealing with fatigue phenomena. Each of these early theories begins with the assumption that a given material is homogeneous and isotropic, which is now known to be untrue, particularly for rolled or extruded products. In addition, microscopic stress irregularities have been detected in these in-homogeneities or variations of grain orientation. These stress irregularities are thought to affect the fatigue behavior of metals. Even Hooke's Law may not be applicable for repeated loadings, since a certain amount of hysteresis is involved; and inelastic or plastic deformation are often observed in the fatigue process.

Most recent efforts have therefore been concentrated on attempts to explain fatigue pheonomena on the basis of the mechanism involved. Several theories have been proposed emphasizing the localized nature of fatigue crack formation, attributing the cause generally to a minute imperfection, plane of weakness, or a particular crystal orientation. Because the problem is complex, a number of simplifying assumptions must be made; and therefore none of the theories advanced so far has fully explained all facets of fatigue behavior. Nevertheless, such theories do provide insights into the basic behavior of metals and suggest avenues for further research.

The strain-hardening theory, based on observations of the processes of slip band formation and strain hardening, has contributed a great deal to the understanding of the mechanisms of fatigue failure, particularly for ductile metals[21]. According to this theory a metal contains a number of plastic in-homogeneities within elastic surroundings. These in-homogeneities cause unequal micro stress distributions. In ductile metals yielding occurs at locations of peak stresses in the form of slip bands before fracture occurs. Under cyclic loading localized strain hardening takes place at these slip bands, and continued plastic yielding is accompanied by an increase in stress. Eventually this stress rises to a value above fracture strength, and a micro crack is formed. The process is then repeated at the ends of this micro crack where a new stress concentration is found until it cracks, and so progressively on to failure. The major limitation to this theory comes from the fact that cold-worked metals have been shown to strain-soften under cyclic stressing rather than strain-harden.

The fatigue process has been characterized as occurring in three stages[11]. First, work hardening accompanies a slip and fragmentation of crystals. Sub-microscopic cracks then occur due to the disruption of the crystalline lattice. Finally these cracks join to form visible spreading cracks, eventually causing failure. The first stage is very brief, and the last stage is also relatively short, with most of the process occurring during the second stage. Fatigue cracking may be "a progressive piling up of an avalanche of disruptions" leading to local fragmentation[11, p. 49].

There are really two different mechanisms involved in the fatigue phenomenon[58]. These may be separated at the knee of the S/N curve of steel. Fatigue failures represented by the steep portion of the curve are described as a delayed static fracture mechanism, and the fatigue failures associated with the flat portion of the curve are attributed to a general structural deterioration. The former is characterized as having a coarse slip similar to that produced by a static test. But the latter results in a fine slip and exhibits very little strain hardening. The deterioration in the second mechanism was illustrated by micrographs showing the transformation of slip zones which appeared early in the fatigue process as sharp fissures. (No more than 1/10 of the expected specimen life is required for this transformation. In fact, the time required seems to determine the life of the specimen). Finally a crack is formed traversing a grain to connect two or more fissures.

The following observations on the development of the fatigue mechanisms were based on failures associated with the flat portion of the S/N curve[59, 60]:

1. Fine slip movements concentrate into slip zones which are areas of mechanical weakness and potential fissures.
2. To-and-fro movements in the slip zones tend to distort surfaces, sometimes building up sharp notches.
3. Notches become fissures early in the fatigue process.
4. Fissures then join to form a microcrack.

One might conclude that the theories so far evolved, though useful in certain applications, are only approximations based on simplifying assumptions. Even so they may become very complex in analysis of a particular case. Each particular fatigue problem is more readily understood at present as an individual case; and, when treated as such in the laboratory, satisfactory predictions concerning its engineering behavior can be evolved.

## Factors Affecting Fatigue Strength

Several factors affect the fatigue behavior of a structural member. According to Reemsnyder[45, p. 22], some of these are as follows:

- A. Load spectrum
  1. Range of stress
  2. State of stress
  3. Repetition of stress
     - a) Regular or random
     - b) Frequency
     - c) Rest periods
  4. Understressing or overstressing
- B. Nature and condition
  1. Prior stress history
     - a) Presence or absence of residual stresses
     - b) Work hardening
  2. Size and shape of spectrum
     - a) Presence of notches
     - b) Size effects (models)
  3. Metallurgical structure
     - a) Microstructure, grain size, and chemical composition
     - b) Mechanical properties
  4. Welding
     - a) Mechanical
     - b) Metallurgical
- C. Environment
  1. Temperature
  2. Atmosphere

A comprehensive examination of the influence of the known variables on the results of fatigue tests has been presented[55, pp. 94-132].

## Effects of Residual Stress

In static load tests it is generally agreed that in most materials residual stresses are of little significance. An exception occurs in columns where residual tensile stresses in the extreme fibers seriously affect the stability of the member. Reductions in strength of up to 35% have been noted in tests on A7 steel[6].

Under repeated load the pattern is not so clear. Most researchers agree that compressive residual stresses tend to increase fatigue strength, and the tensile residual stresses reduce it[45, p. 17]. In mild steel there is a marked tendency for the residuals to fade during cycling, particularly if local stresses approach the yield strength. A recent study has reported evidence of this tendency, even under low stress amplitudes[42]. This study showed that residual stresses continued to fade at 10 million cycles under a stress amplitude of only 8000 psi. Generally the amount of fading increased with increase in stress amplitude and number of cycles. For mild steel "the upper limit of the fatigue range of stress (with zero mean stress) is approximately equal to the yield point . . . so that . . . yielding may occur and reduce the effective applied stress"[7, p. 81]. For this reason residual stresses should not reduce the fatigue strength more than 5 to 10 percent. The same conclusion was arrived at based on studies of stress relieved specimens[49].

There is considerable evidence, however, that residual stresses have far more effect on the fatigue strength of hard or high strength steels. Residual stresses in mild steels under alternating stress may relax before significant damage can occur, but in quenched and tempered steels fatigue damage may occur before residual stresses have relaxed to safe values[31].

One series of investigations resulted in the conclusion that residual stresses are basically similar to improved static stresses in their effect on fatigue behavior of metals[48]. This research also found that residual stresses in a steel which has been hardened by heat treatment will relax much less than those in a steel which has been softened or annealed. This was particularly applicable if the applied stresses were low and the fatigue life was long.

In an investigation of notched specimens it was found that static properties such as tensile strength and elongation give no reliable indication of permissible stresses under cyclic loading[13]. When tensile residual stress was present around the root of a sharp notch, specimens of mild steel were appreciably stronger than those of high strength steel[47].

A possible explanation of this phenomenon may be reached by extending to its logical conclusion the argument set forth regarding high strength steel[7]. For example, from the modified Goodman diagram representing the fatigue strength of "T-1" steel the upper limits of the fatigue range of stress (with zero mean stress) are only about one-third of the yield point stress. Therefore "T-1" steel in this fatigue range of stress never approaches the yield point of the material, and consequently there is insufficient yielding of areas of locally high tensile residual stress and little opportunity for their relaxation.

## Residual Stresses Due to Welding

In the process of welding a great deal of heat is generated. Most of this heat is localized in the fusion zone of the weld and gradually diminishes through the heat-affected zone and the rest of the material. The fusion zone is that area of the weld section which is fused with the weld metal and undergoes a degree of inter-granular mixing of weld metal and parent metal. The heat-affected zone is that area just around the fusion zone which undergoes metallurgical change during the weld process[39].

Because the temperature gradient between the fusion zone and the body of the parent shape is steep, the weld area upon cooling must experience a certain amount of plastic flow. The extent of the area involved depends on the geometry of the shapes, the type, heat, and speed of welding, and on the rate of cooling. If the weld area is the last to cool, and if there has been sufficient temperature difference and restraint across the shapes, the residual welding stress will be at the yield point of the weld metal or in the parent metal of the heat-affected zone. This may or may not be greater than the yield point of the unwelded parent metal[54].

In A7 steel plates, the residual stresses in or near the weld metal may be about 50% above the yield strength of the parent metal[44]. This residual stress is not only in the direction of travel of the weld, but may in fact be triaxial, possibly at yield point, depending on the geometry of the section and to a lesser degree on the other factors mentioned. This suggests a basic similarity of stress condition between butt welds and cover plate-to-flange fillet welds.

## Welding of "T-1" Steel

A great deal of study has recently been devoted to the problem of the production of good welds in "T-1" steels. Detailed studies of the heat affected zone (HAZ) indicate that the good notch toughness found in "T-1" steel is due to the formation of low-carbon martensites created in the quenching and tempering process[39]. If the welding is done with no preheat and low energy input, this quality is retained in the steel. However, under welding conditions involving high restraint, moisture on the steel, too-high moisture content in the electrodes, or low weld metal ductility, preheating may be required. But preheating or high energy inputs decrease the cooling rate which results in the low carbon martensite being replaced by high-carbon martensite or bainite and a corresponding decrease in notch toughness. There is a marked tendency on the part of "T-1" steel weldments to crack on cooling. A series of tests employing the Lehigh and Tekken restraint specimens[32] indicates that most cracking initiated at the root of the weld about three minutes after welding at temperatures below 90°C. Many of the cracks did not appear on the surface. Beneficial effects were reported from either preheating or post-heating. Another investigation found that rapid cooling of the welds by water quenching also eliminated cracking in cruciform tests using either 1/2" or 1" plates[8]. These are very severe tests, so it appears that with reasonable care cracking can be eliminated from most applications involving "T-1" steel. The manufacturer points out that with proper electrodes, correct welding heat, and recommended procedure, "T-1" steel can usually be easily and reliably welded[52].

## Fatigue Strength of Welded Members

The effects of the residual welding stresses on the fatigue strength of the structure are similar to those of the residual stresses discussed before except that they are in general more severe and are more subject to discontinuities. Studies of the fatigue characteristics of welded joints in structural steel have shown that the effects of stress raisers, such as notches and discontinuities, may be greater than the effects of residual welding stresses[50]. The results of several fatigue tests of butt welds show that the formation of metallurgical and geometrical stress raisers and the intrinsic change in homogeneity of material due to welding have been interpreted to cause the reduction in fatigue life found in welded joints[23]. A method for determining a fatigue factor to be used in reducing allowable design stresses for various weld configurations has been developed[4].

At the University of Illinois investigations of the fatigue behavior of welded beams and girders and associated details have yielded much information[17, 26, 33, 51]. These tests generally point up the danger of indiscriminate welding, particularly on or near the tension flange. In a recent summary of these fatigue studies the conclusion is that for welded girder highway bridges loaded from 1/4 to 1/2 tension to tension, all members tested but those with partial length cover plates, and possibly those with splices, appear to have adequate fatigue capacity at basic design stresses for 2,000,000 cycles of loading[38]. If AWS Formula 1 is used for the allowable design stresses at the ends of partial length cover plates, they too should be safe.

An earlier investigation checked the fatigue strength of rolled beams with cover plates attached by continuous and intermittent fillet welds so that the ends of the cover plate were not critical[56]. It was determined that for 2,000,000 load cycles the fatigue strength of the beams with cover plates attached by continuous fillet welds was only 73% of that for plain rolled beams, and with cover plates attached by intermittent fillet welds, only 53%. The corresponding applied stresses were 31,200 psi for plain rolled beams, 22,800 psi for beams with cover plates attached by continuous fillet welds, and 16,500 psi for the beams with cover plates attached by intermittent fillet welds. On the basis of static load tests the rolled beams carried 71.2% of ultimate for 2,000,000 applications of load; the beams with cover plates attached by continuous welds carried 38.5%, and by intermittent welds, 33.5%. No investigation seems to have been made regarding residual stresses in these tests, however.

Fatigue tests conducted on "T-1" steel have demonstrated that butt-welded plates have much lower fatigue strength than as-received plates[20]. In fact much of the advantage of using high strength steel seems to be lost in this process, at least at butt-welded splices. Results of constant stress bending tests of butt-welded steel plates of 50 ksi and 90 ksi yield strengths indicated that the fatigue strengths of the lower strength steel for 100,000 cycles was at or near yield, but that the fatigue strength of the higher strength steel was considerably below yield, and in fact had an advantage of only 4 ksi over the lower strength steel[16]. In these tests it was noted also that weld bead shape seemed to be the most important variable.

Welding in most cases has been found to reduce the fatigue strength of the unwelded parent metal. This reduction is most notable and consistent when the number of cycles is relatively large. The effects are also more marked in higher strength steels.

## PREPARATION OF SPECIMENS

The basic components of the specimens were a rolled shape, a cover plate, and a concrete slab. In a previous research project a cover plate had been so welded to the bottom flange of each rolled shape that the resulting beam had been prestressed. In this project a concrete slab was cast on the top flange of these beams to form a composite beam (figure 1).

The first beam had been fabricated at a plant in Des Moines, Iowa, and was designated as the "Des Moines beam". The second beam was fabricated by welders of the same Des Moines firm in the Iowa State Highway Commission facilities in Ames, Iowa. This beam is the "Ames beam".

### MATERIALS

The steel beams were prestressed to utilize more efficiently two different types of steel. High strength steel was used for the 6 x 3/8 in. cover plates. The steel, a quenched and tempered weldable low-carbon alloy, was developed by United States Steel Corporation and patented as "T-1" constructional alloy steel[53]. The rolled shapes used were ordered to meet the minimum requirements of ASTM-A36-60T[2, pp. 258-260]. The shapes were standard 18-inch wide-flange sections, each 26 feet long and weighing 50 pounds per foot. The rolled section of the beam identified as the Des Moines beam was suppled by the Bethlehem Steel Company, and the rolled section of the Ames beam was furnished by the Inland Steel Company.

Concrete for the slab was obtained from a local ready-mix establishment. A high strength mix was used to develop the necessary strength and to lower the shipping weight. The mix design was the standard Class X-4 of the Iowa State Highway Commission tandard Specifications[29, pp. 324-326]. The coarse aggregate was Ferguson crushed rock with 1-in. maximum size.

TABLE I. PROPERTIES OF MATERIALS

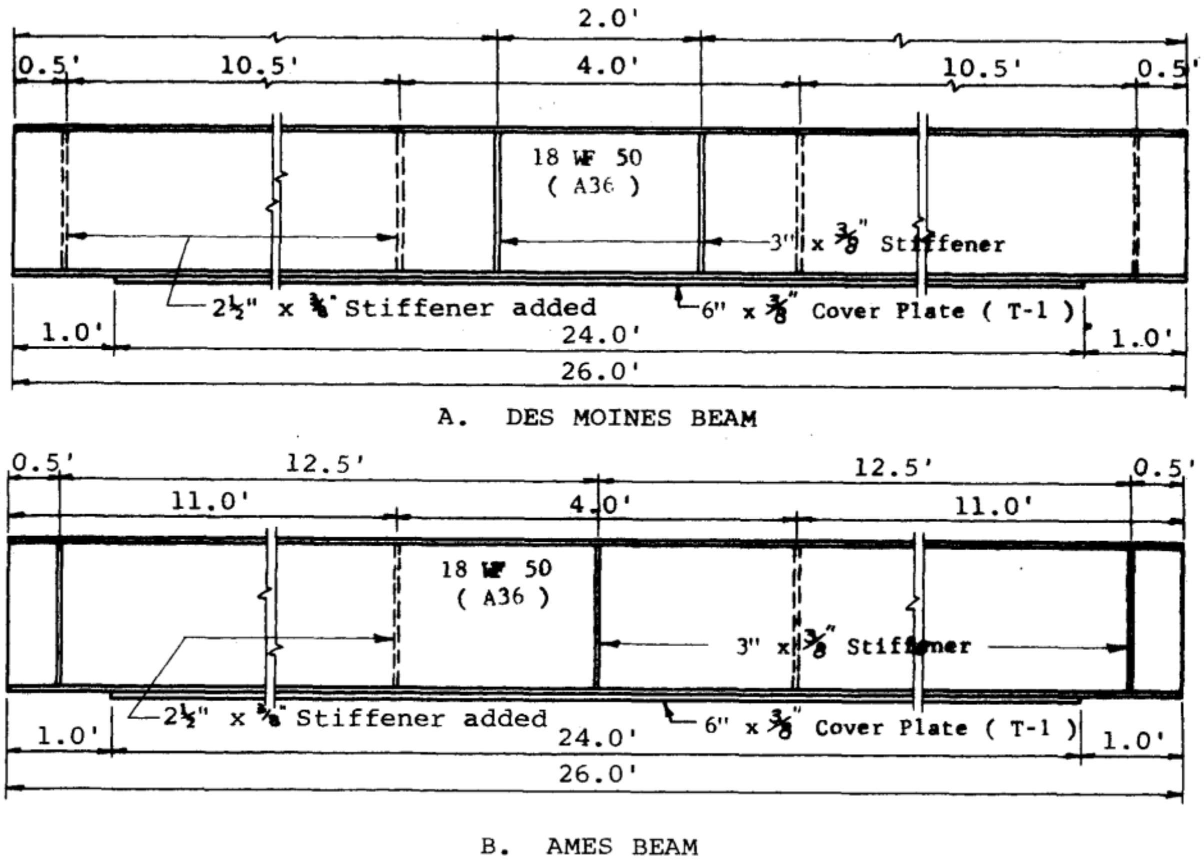
| Property | Steel[30] A36 | Steel[30] "T-1" | Concrete 7-day | Concrete 28-day |
|---|---|---|---|---|
| Modulus of elasticity times $10^6$ (psi) | 29.8 | 28.2 | 3.47 | 3.94 |
| Yield point (ksi) | 38.1 | | | |
| Yield strength for 2% offset (ksi) | | 118.1 | | |
| Tensile strength (ksi) | 64.8 | 132.5 | | |
| Compressive strength (psi) | | | 5,540 | 7,050 |
| Elongation in 8" (percent) | 29.8 | | | |
| Elongation in 2" (percent) | | 17.3 | | |

Properties of the steels were determined by coupon tests at the Ames Laboratory of the Iowa State Highway Commission[30]. The results reported for the concrete were based on tests on standard 6 x 12-inch cylinders.

### DESIGN AND FABRICATION

Specimens used in this investigation were designed as composite beams, to simulate the practice of utilizing composite action between the roadway slab and the steel beams in the positive moment region of an I-beam bridge. The steel portion of the specimens were designed and fabricated as part of an earlier research project sponsored by the Iowa Highway Research Board in cooperation with several steel producers and fabricators.

Each of the two specimens was prestressed, but slightly different methods had been used. The Des Moines beam was prestressed by welding the unstressed cover plate to the beam while the beam was held in a deflected position by a jack load. After simply supporting the rolled shape on 25-foot centers the "T-1" cover plates were positioned on the top flange. A jack load of 30.0 kips was




A. DES MOINES BEAM

B. AMES BEAM

**FIGURE 1. DETAILS OF STEEL BEAMS AS RECEIVED**

Note: Dashed lines indicate location of stiffeners added as part of project reported herein.

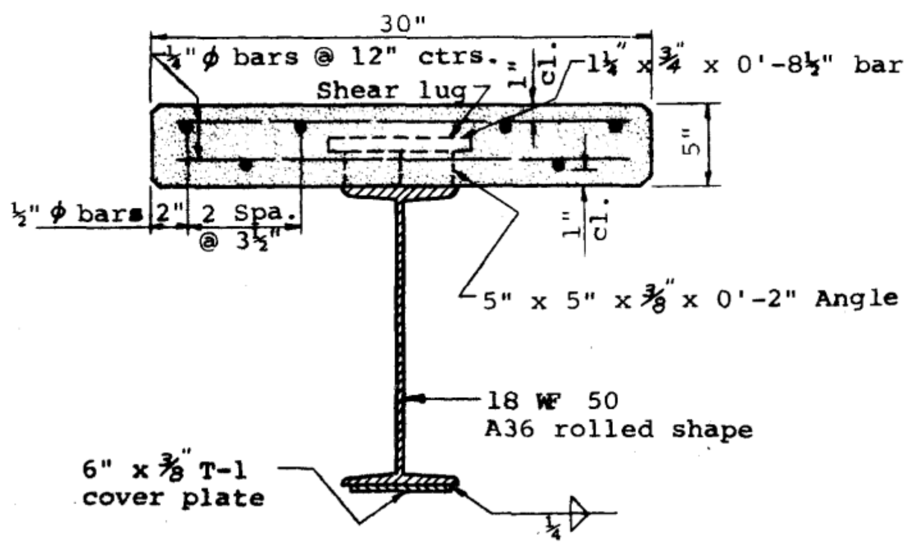



**FIGURE 2. TYPICAL CROSS-SECTION OF SPECIMENS**

applied at midspan. The cover plate was tightly clamped and lightly tack-welded. Then, with the jack load still applied, continuous fillet welds were laid on each side at the same time for the full length of the cover plate. The welds were allowed to cool, and then the jack load was released.

No attempt was made in the fabrication to hold the jack load constant. The initial 30.0-kip jack load increased to a maximum of 35.5 kips during the welding operation and fell to 16.5 kips after the welds cooled to room temperature.

The Ames beam was prestressed similarly except that the jack load was increased to 39.0 kips and the "T-1" plate was tack-welded with 4-inch welds at 18-inch centers starting from midspan and proceeding alternately both ways. Then the jack load was released, and full-length continuous fillet welds were laid on both sides of the plate beginning at one end.

Continuous cover plate-to-flange fillet welds on each beam were placed by a semi-automatic shielded metal-arc process.

TABLE II. WELDING DATA FOR BEAMS[30]

| | |
|---|---|
| Wire | L-60 (3/32) |
| Flux | 760 |
| Voltage | 34-38 |
| Amperage | 525-560 |
| Speed (in./min.) | 25+ |
| Fillet weld size (in.) | 1/4 |
| Welding time (min.) | 17+ |

To determine the stress effects of the fabrication procedures, both beams were instrumented to obtain deflection and strain readings at regular time intervals. Both Whittemore mechanical and SR-4 electrical strain gages were used.

The final specimen design was proportioned so that the desired stress range could be applied to the steel without producing high stresses in the concrete. A certain amount of creep would occur in the concrete, and the amount would increase in direct proportion to the size of the applied stresses. A certain amount of recovery was also expected in the concrete during rest periods[40]. Theoretically inelastic behavior such as creep changes the location of the neutral axis, and therefore the stress in the steel might vary undesirably, although the applied load was kept constant.

A 5-inch slab, 30 inches wide, was selected (figure 2). The slab was reinforced with six 1/2-inch diameter deformed bars placed longitudinally and 1/4-inch rods on 12-inch centers placed transversely.

Composite action between the concrete and steel was assured by the use of shear lugs such as are commonly used on I-beam bridges by the Iowa State Highway Commission. Fifteen shear lugs were welded at about 18-inch centers to the top flange of each rolled beam before the concrete was placed. The effect of shear lug welding on the state of stress in the bottom flange was investigated by the use of a Whittemore strain gage.

Measurements taken on the bottom flange indicated that a compressive stress of 2.3 ksi to the bottom flange and cover plate at midspan was added as a result of shear lug welding stresses. (This was somewhat offset by a computed tensile stress of 1.6 ksi due to the dead load of the slab and beam.)

## FATIGUE TESTS

A study of the flexural fatigue behavior of the specimens was one of the objectives of this study. Both beams were therefore tested to failure under a repeated load.

### DETERMINATION OF STRESS RANGE

Fatigue life is influenced primarily by three load factors: the maximum unit stress, the stress range, and the number of load or stress applications. Previous tests had indicated that a stress range of 15 ksi and a maximum theoretical stress of 50 ksi in the "T-1" cover plate might be expected to cause failure at about the desired 2,000,000 cycles[46]. Since only two specimens had been tested previously, further information for this same stress range seemed desirable. The loads which would produce this stress were determined by conventional methods of composite design.

To examine more thoroughly the probable behavior of the specimens, theoretical stress-load data were plotted for both steels and related to the respective 2,000,000 cycle envelopes for these materials in the as-received condition[20]. The envelopes were plotted in the form of modified Goodman diagrams (figure 3). The selected stresses were about 50% of the critical range reported for these steels in the as-received condition for 2,000,000 cycles of applied stress.

The stresses in a weld area are actually triaxial in nature. Since fatigue failure is a highly localized phenomenon, the direction of the weld with respect to the direction of applied stress may not be a factor. If so, the fatigue strength of a cover plate-to-flange fillet weld and of a butt weld should be compared. The 2,000,000 cycle envelopes for butt-welded joints have been plotted (figure 4), from data for as-received plates[3, p. 60]. The design stress-load data were also plotted. The applied stresses are nearly the same as those which might be expected to cause failure of a butt-welded joint at 2,000,000 cycles.

Another consideration in the process of load selection involved the fatigue strength of the concrete. To insure that failure would not occur in the concrete the design stress range in extreme fiber was checked against a concrete failure envelope for 2,000,000 cycles[4]. The diagram shown in figure 5, for 5,000 psi concrete indicated the proposed design would be satisfactory.

The equipment used to apply the repeated load was an Amsler variable speed pulsator operating through two hydraulic jacks. Each jack had a maximum load capacity rating of 50,000 lb. and a maximum cyclic stroke of 1/2 inch. The selection of a design section and load range was made to meet these limitations.

A significant relationship was noted between the stress range as finally determined and the design stresses on a prototype bridge now in service on U. S. Highway No. 6 over Silver Creek in Pottawattamie County, Iowa[5]. The minimum applied load (P = 14 kips) produces applied stresses nominally equivalent to the dead load stresses on the prototype; and the dead load, plus live load, plus impact stresses are approximated by the maximum applied load (P = 35 kips).

### TESTING PROCEDURE

Each specimen was fatigue tested as a simply supported beam, utilizing the facilities

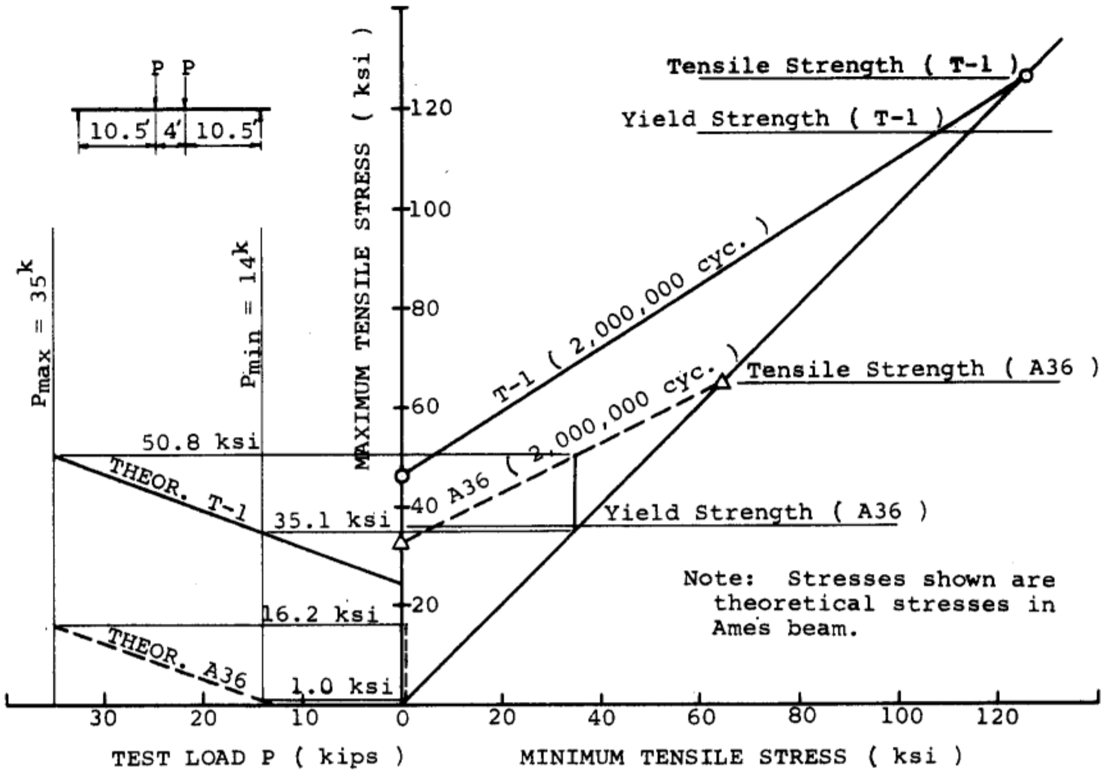



**FIGURE 3. DESIGN STRESS-LOAD AND FATIGUE STRENGTH DIAGRAMS FOR STEEL**

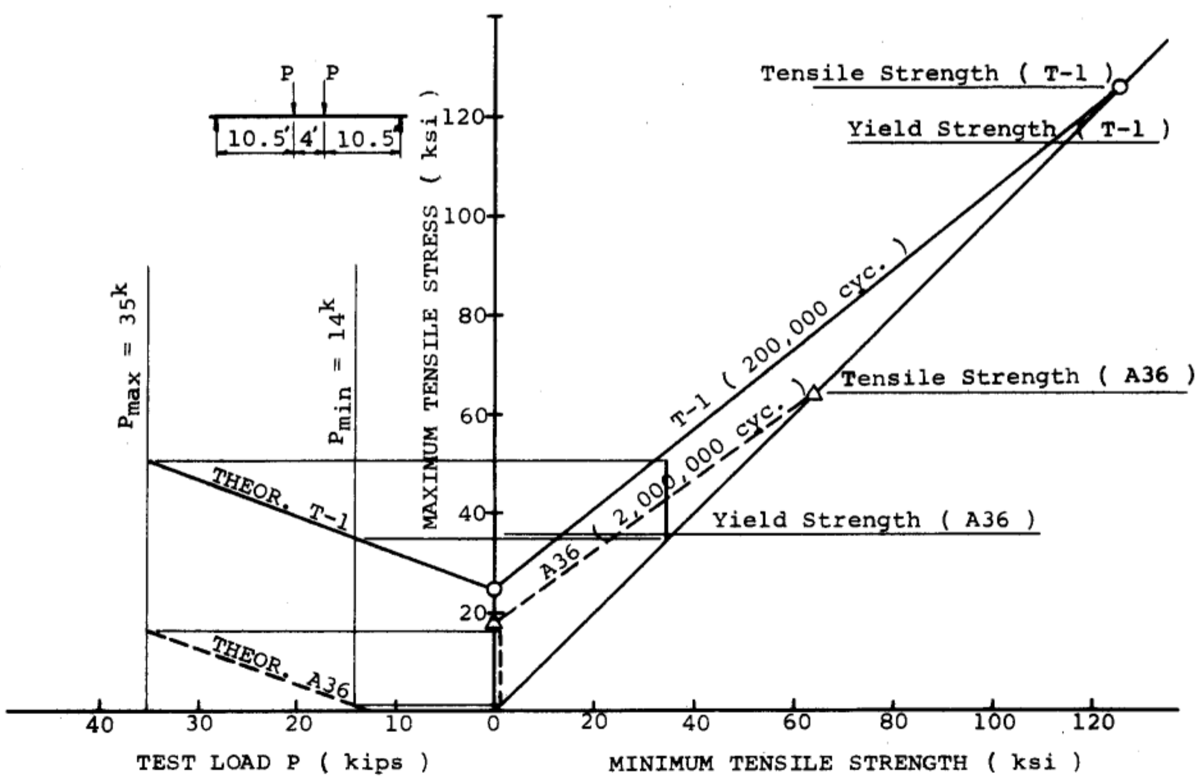



**FIGURE 4. RELATIONSHIP OF DESIGN STRESS-LOAD DIAGRAM TO FATIGUE STRENGTH DIAGRAMS FOR TRANSVERSE BUTT-WELDED JOINTS**

of the Association of American Railroads in Chicago, Illinois (figures 6 to 10). A pulsating sinusoidal load was applied by the jacks shown in position near midspan. At intervals these jacks were also used to apply a static load so that strain and deflection readings could be obtained during the course of the tests.

Each beam was instrumented to obtain strain measurements and deflections under static load at regular intervals during the fatigue loading period. SR-4 electrical strain gages were attached to the top of the concrete slab and the bottom of the "T-1" steel cover plate (figure 11). These readings were supplemented by strain measurements using a Berry mechanical strain gage primarily to detect any possible fading of the SR-4 gage readings during the repeated loading of the specimens. Deflection readings were taken utilizing an Ames dial reading to the nearest 0.001 inch and a taut wire and scale gage reading to the nearest 0.01 inch.

TABLE III. SUMMARY OF THEORETICAL STRESSES

| Item | Des Moines Beam | Ames Beam | Prototype[a] |
|---|---|---|---|
| Minimum tensile stress in "T-1" steel (ksi) | 29.0 | 34.8 | 32.2[b] |
| Maximum tensile stress in "T-1" steel (ksi) | 43.5 | 49.9 | 50.6[c] |
| Minimum tensile stress in A36 steel (ksi) | 2.9 | 0.7 | 0.3[b] |
| Maximum tensile stress in A36 steel (ksi) | 17.1 | 15.4 | 18.7[c] |
| Fatigue crack location | "T-1" | "T-1" | |
| Cycles to first crack | 646,000 | 1,392,200 | |
| Cycles to collapse | 836,500 | 1,500,400 | |

[a]Source: P. F. Barnard[5].

[b]Dead load.

[c]Dead load + live load + impact.

An inspection of the bottom flange area, particularly around the welds, was made daily. A small flashlight and mirror were used in this examination. It was found that this inspection could best be made while the specimen was under cyclic loading so that any crack could be observed opening and closing.

The general daily operating procedure was as follows:

1. Each morning, test a static load and record strain and deflection readings at zero load, minimum cyclic load, and maximum cyclic load.
2. Start pulsator and adjust to desired minimum and maximum cyclic loads.
3. Inspect for cracks while under cyclic load.
4. After about 7 hours of operation stop pulsator and repeat static load test, taking readings as in Step 1.
5. Re-start pulsator as in Step 2.
6. Stop pulsator about midnight.

This procedure took around 14 to 16 hours of cyclic loading per day. The specimen under test was then rested for 8 to 10 hours each day. Rate of load repetition was held constant at 110 cycles per minute during the testing periods.

Under cyclic loading both specimens were found to be very stable. The Amsler equipment operated without any difficulty, and there was no tendency on the part of the beams to work their way out from under the load. Lateral stability was provided by light steel straps which were snugly attached to the testing frame and to 1/4-inch steel loops cast into the edge of the slab at the supports and at the 1/3 points of the span.

## RESULTS

Neither specimen carried the design cyclic load for the projected 2,000,000 cycles. Both failures appeared at stress raisers in

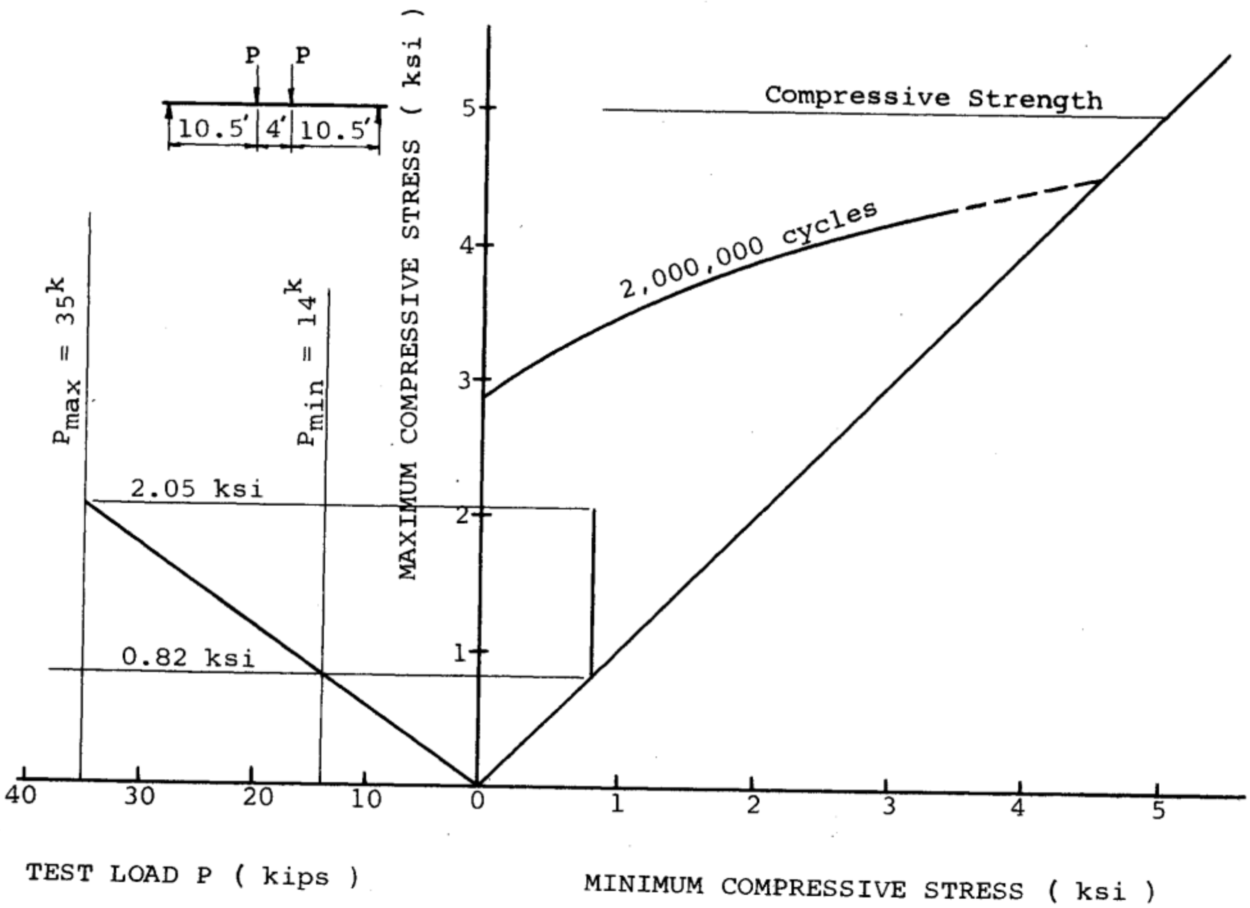



**FIGURE 5. DESIGN STRESS-LOAD AND FATIGUE STRENGTH DIAGRAM FOR CONCRETE**

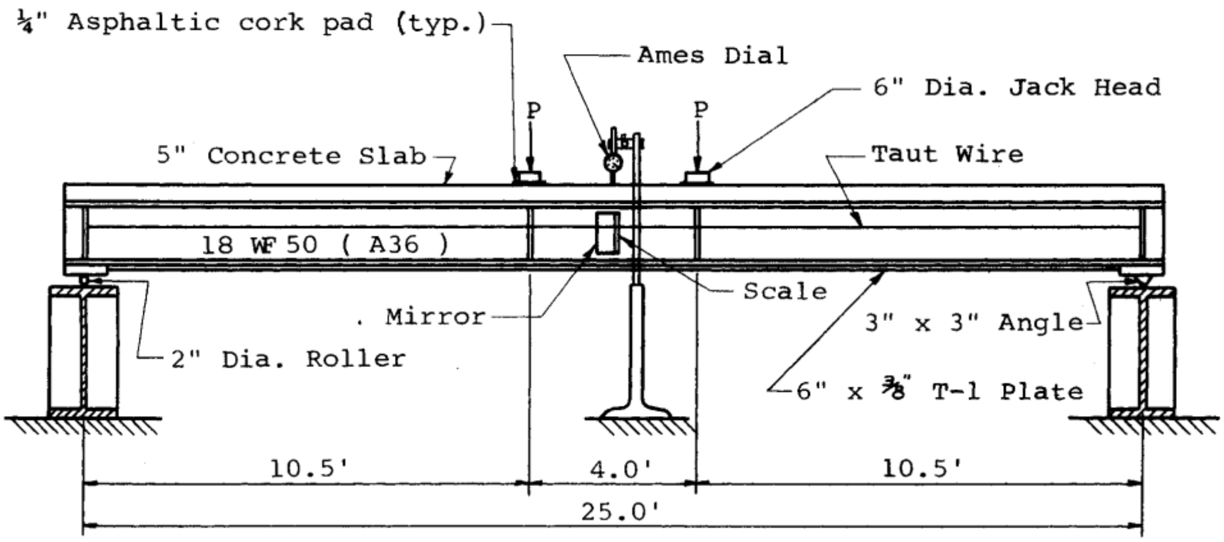
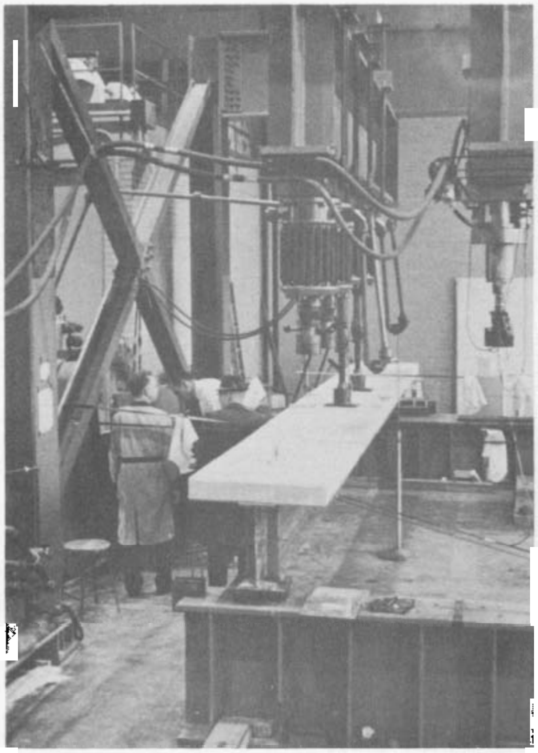
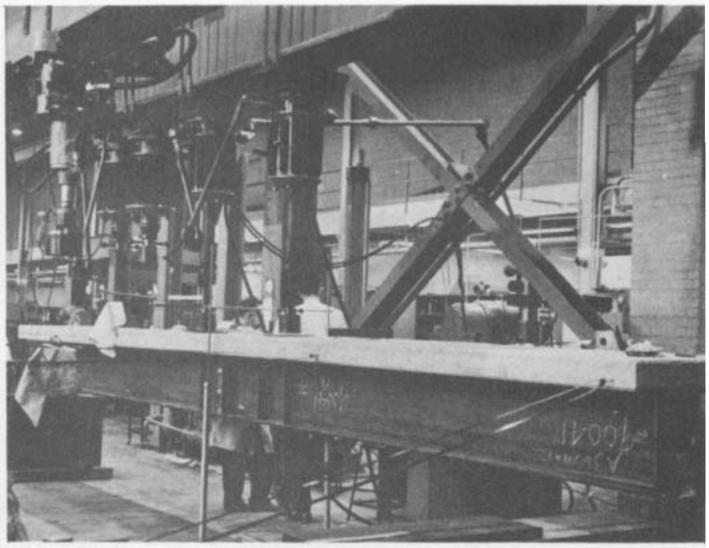
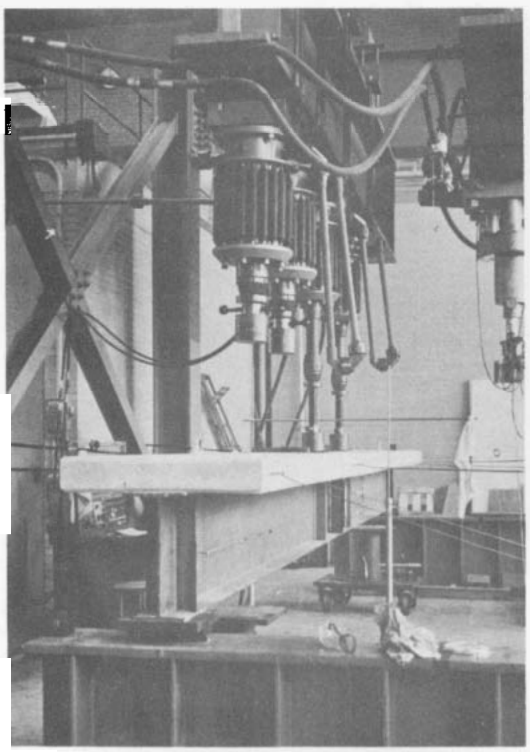


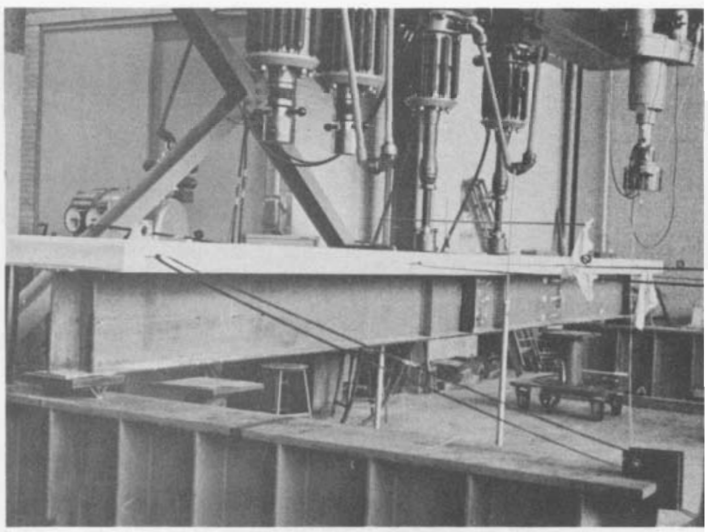

**FIGURE 6. SET-UP FOR CYCLIC LOAD TESTS**



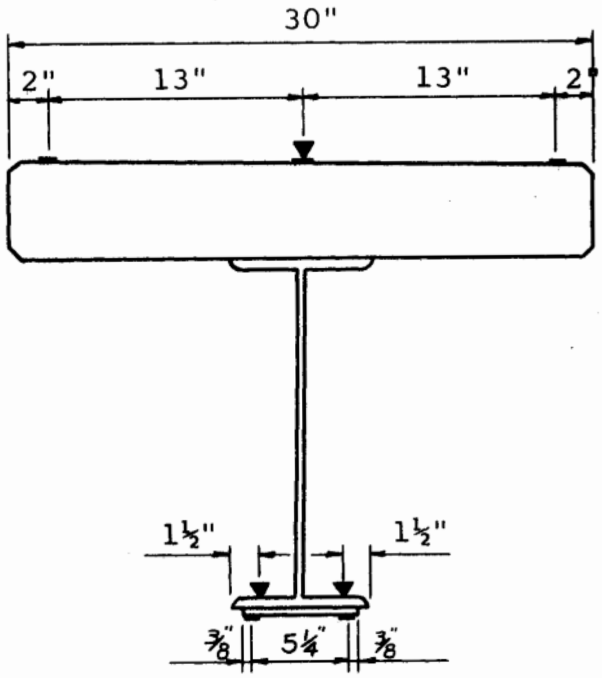


Figures 7 and 8. The Des Moines beam in position for the fatigue tests.





Figures 9 and 10. The Ames beam in position for the fatigue tests.




CROSS-SECTION OF SPECIMENS

LEGEND:

SR-4 ELECTRICAL RESISTANCE STRAIN GAGE

GAGE HOLES FOR WHITTEMORE OR BERRY MECHANICAL STRAIN GAGE

**FIGURE 11. DETAILS OF STRAIN INSTRUMENTATION AT MIDSPAN FOR CYCLIC LOAD TESTS**

the form of weld flaws (table III).

A crack in the Des Moines beam was first noticed at 646,000 cycles. This crack at that time extended from the edge about 3 inches or halfway across the bottom "T-1" cover plate. When the crack was discovered, the section seemed to have stabilized somewhat, because in the next 80,000 cycles the crack extended only an additional 1/2 inch. The flange of the A36 beam was not yet visibly cracked. At about 730,000 cycles a crack was noticed just above the crack in the cover plate at the edge of the rolled beam flange. This crack and the one in the cover plate then progressed at about the same rate, so that when the cover plate had completely ruptured the crack in the rolled shape could be seen to be about 3 inches in from the edge of the flange. The rupture then extended rapidly in a tearing process with much "necking." Finally at 836,500 cycles, 190,500 cyles after a crack had first been noticed, the beam failed to carry the design load, so the test was stopped.

Although no unusual flaw was apparent at the point of failure before testing, a highly discolored area across the weld and into the "T-1" cover plate face was found on the crack surface (figures 12 and 13). The well-defined shape of the discolored area suggests that this crack may have started during the fabrication process, probably when the weld cooled. Where the crack appeared, almost no fusion was found between the weld metal and the flange of the rolled beam. This may explain the late development of the crack in the rolled beam.

The fatigue strength of the Ames beam was much better. A crack was first noted on this specimen in the weld area after 1,392,200 cycles. At that time the crack extended about 1 inch in from the edge of the cover plate and up into the bottom flange of the rolled shape (figure 15). At about 100,000 cycles later, the crack had progressed approximately 2 1/2 inches across the bottom cover plate and about the same amount across the rolled beam flange. The rate of the crack enlargement then accelerated rapidly however, and the beam failed to take the design load less than 10,000 cycles later. The final portion of the failure was quite ductile, and the crack could be seen to advance with each load application as in the Des Moines beam.

Failure was at a weld flaw (figures 14, 15). The stress raiser was apparently not a crack in the weld, but rather it was at a point where an arc had been struck on the edge of the "T-1" cover plate forming a notch. The crack was not at the root of this rounded notch but at the sharp shoulder (figures 16, 17).

In neither the Des Moines nor the Ames beam did the flexural behavior of the beams give any indication of impending collapse. This is a usual characteristic in fatigue failures of this type. Neither static load deflection nor strain readings showed any significant change though failure was imminent. For example, just 10,000 cycles prior to collapse of the Ames beam, the camber had decreased only 0.015 inch or about 2.5 percent of the total deflection due to the applied jack loads. Strain readings under static load failed to show any appreciable change.

Although the static load deflections and strain measured throughout the tests remained nearly constant, the zero readings indicated that some plastic flow or creep was taking place, and an increasing amount of "set" or loss of camber was shown in the beam as the cyclic loading progressed (figure 18). The difference in the behavior of the two specimens was the result of a difference in their stress

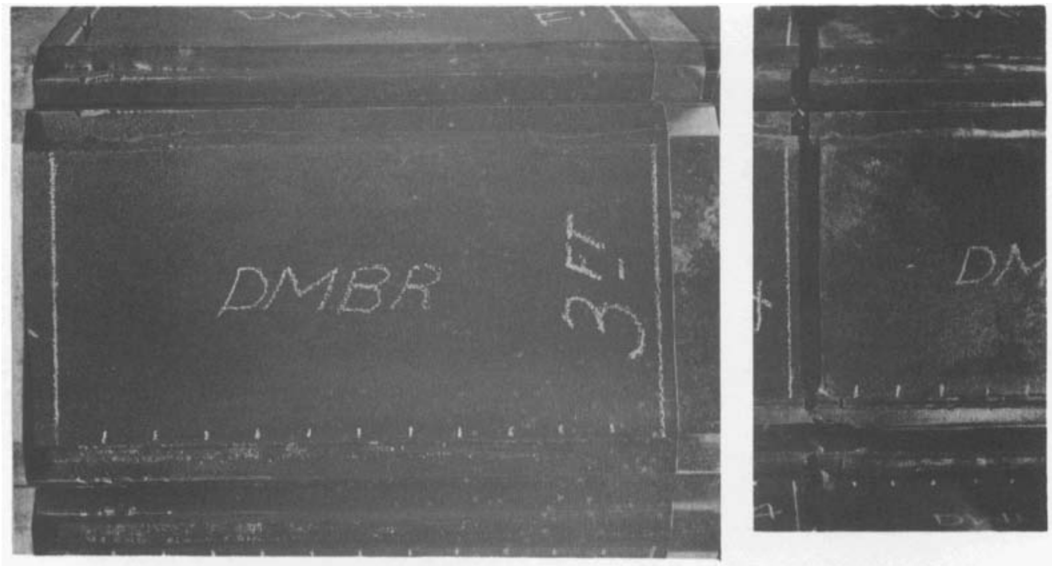



Figure 12. At left, the welds in the Des Moines **beam before testing.** Mirror images of the welds are at the top and **bottom of the beam.** Arrow at top left shows where fatigue crack developed. **Fatigue** crack after test is shown at right.

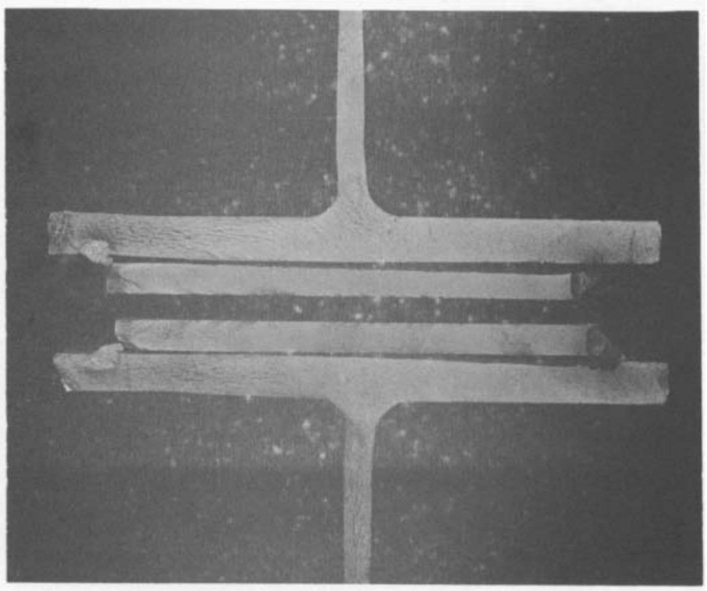


Figure 13. Fatigue fracture surface of Des Moines beam.


Figure 14. The welds in the Ames beam before testing. Mirror images are at the top and bottom.

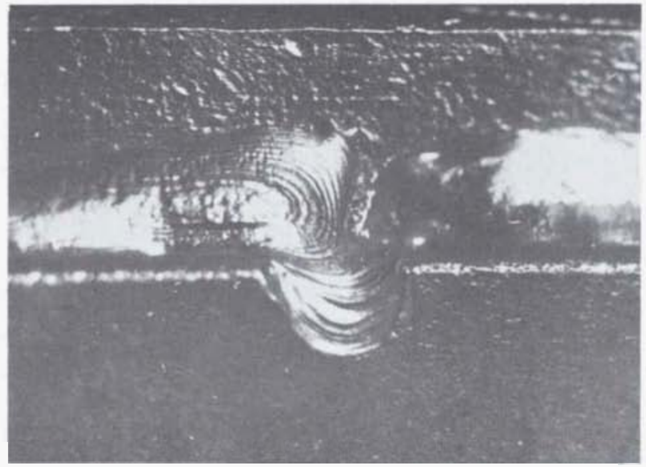


Figure 15. The fatigue crack when first noticed.


Figure 16. The fatigue crack at end of test.

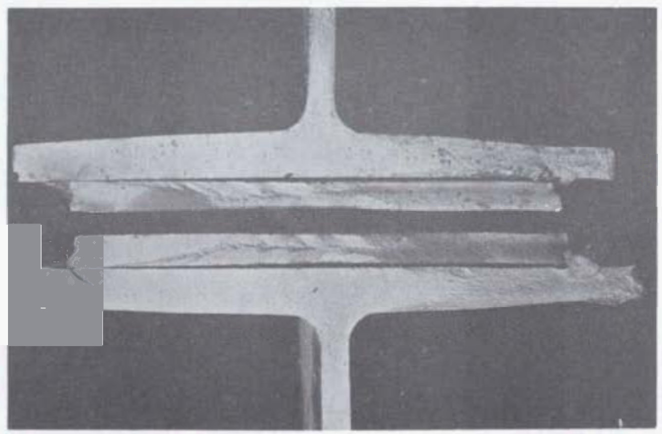


Figure 17. Fatigue fracture surface of Ames beam.

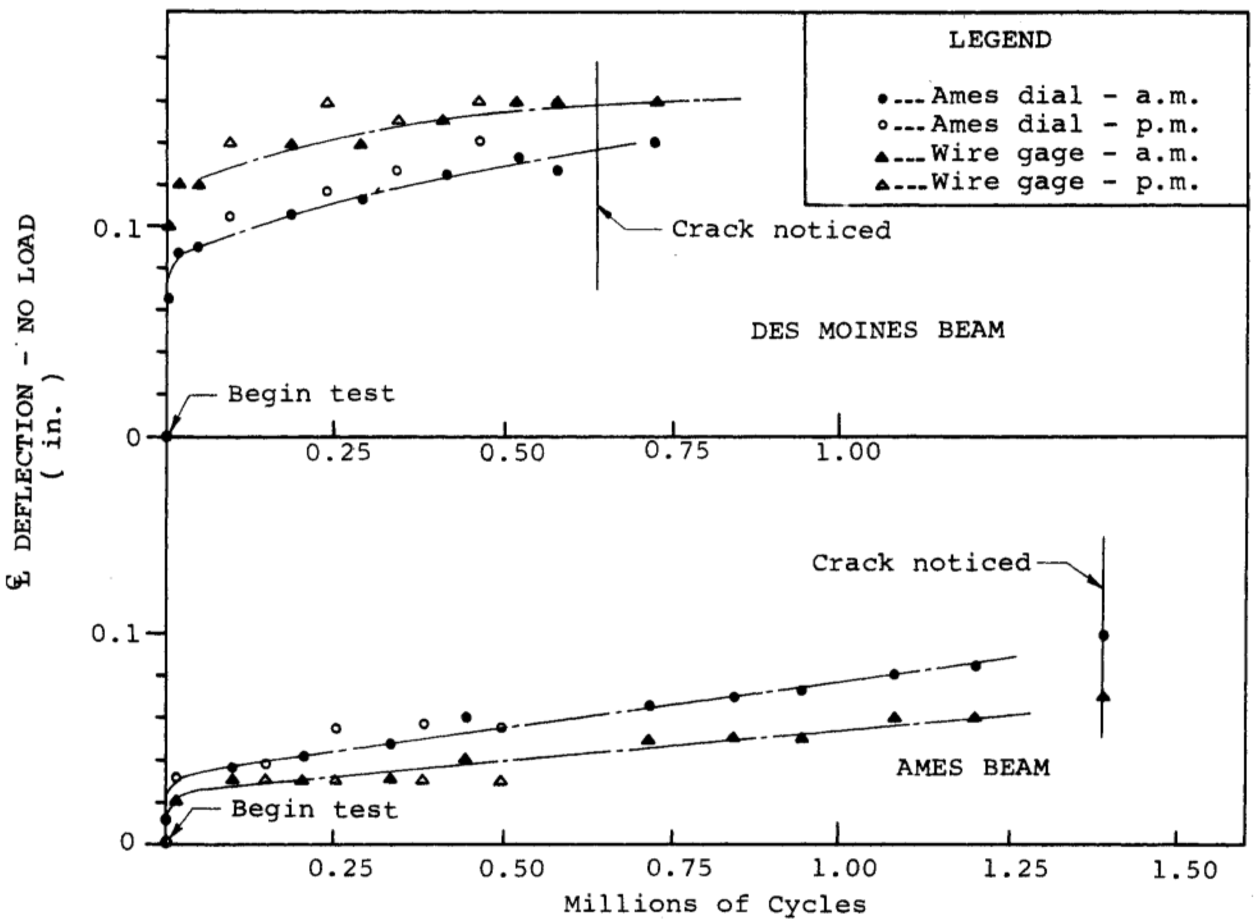



FIGURE 18. CHANGE IN CAMBER DURING CYCLIC LOADING

history. The steel portion of the Ames beam had been statically load-tested shortly after fabrication at the Iowa Highway Commission Laboratory in Ames. This loading had produced a maximum applied stress of about 33 ksi in the cover plate at midspan which resulted in an immediate loss of camber of slightly more than 0.2 inches. Therefore, the steel section of this specimen had in effect been pre-strained, but the steel portion of the Des Moines beam had not been loaded previously. Inelastic deformation, or "set", took place in the steel of the Des Moines beam during the first loading. This "set" continued to increase as the number of cycles increased, but at a decreasing rate. In the Ames beam there was almost no change in the steel zero load strain readings. Nearly all the "set" was in the concrete (figure 19).

Nevertheless, the properties of the concrete slabs remained quite constant throughout the tests. A small amount of creep was experienced, but the application of a constant load was found to produce a constant strain as each test progressed.

A certain amount of recover was noticeable in the concrete after a rest period. Deflection readings under zero jack load revealed that usually afternoon readings taken after a period of cyclic loading indicated more "set" than similar readings taken the following morning after a period of rest (figure 18).

## RESIDUAL STRESS DETERMINATION

In previous tests the state of stress developed in the fabrication of prestressed steel beams similar to those tested in this study was of much interest[46]. The residual welding stresses found were such that they might certainly be expected to contribute to an early fatigue failure. A similar investigation of the state of stress in these specimens was conducted.

### PROCEDURE

After failure of the specimens in the fatigue study, the method of sectioning was used to evaluate the remaining residual stresses[35]. Segments of the beams were removed and sawed into strips. Strain measurements were made by use of a Whittemore mechanical strain gage. The stress distribution was evaluated only in the bottom flange of the rolled shape and cover plate.

Six segments of the cover-plated flange were selected on the unbroken half of each beam (figure 20). These segments were numbered consecutively from 1 through 6 beginning at the segment nearest the midspan. The internal state of stress was evaluated throughout segments 1, 4 and 6. The remaining segments were investigated by strain measurement only along the welded edge of the "T-1" cover plate to determine the degree of stress continuity.

The residual stresses at these locations were evaluated by sawing the segments into strips. Before being sawed, each strip was given a designation which identified it as to the beam it came from, the segment of the beam, and the location in the segment. For example, the designation, AP4-24, indicates the strip came from the Ames beam (A), cover plate (P), segment 4. The first number to the right of the hyphen, 2, indicates the strip came from side 2 of the beam, and the last

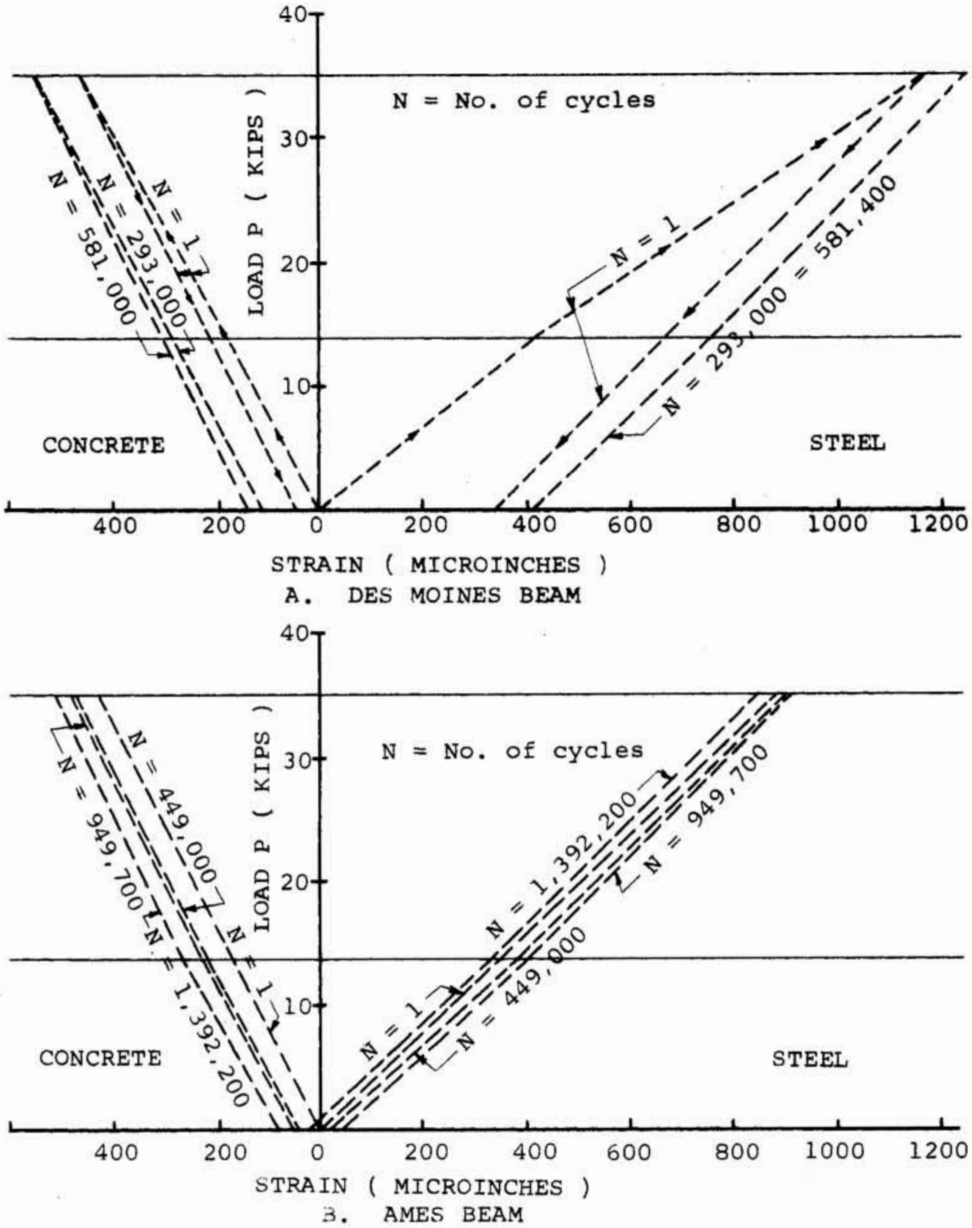



FIGURE 19. STRAIN-LOAD BEHAVIOR

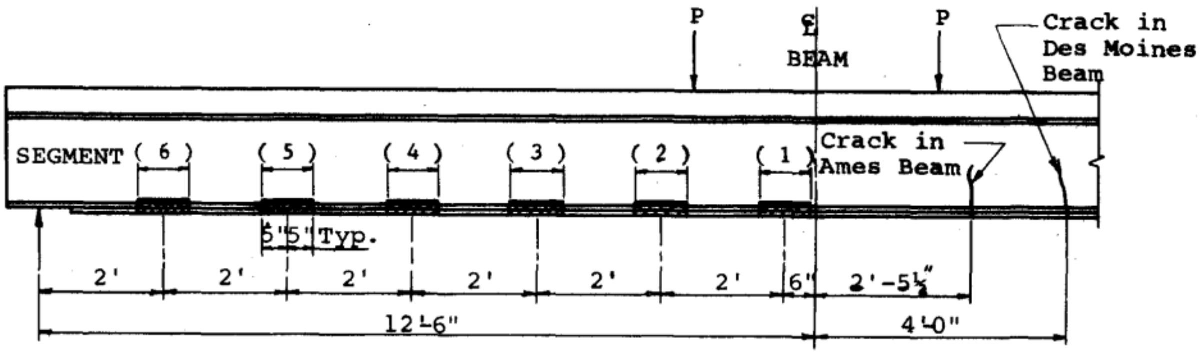



FIGURE 20. SEGMENT LOCATIONS AS SELECTED ON CRACKED BEAMS

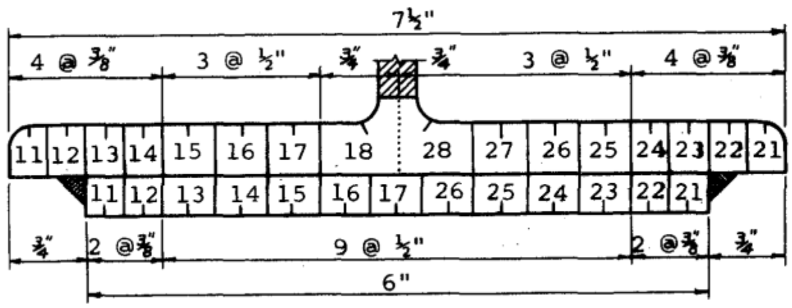



FIGURE 21. STRIP LOCATION WITHIN SEGMENTS

number, 4, shows it was the fourth strip in from the edge (figure 21).

Gage holes were drilled on each end of each strip to accommodate a Whittemore strain gage with a 10-inch gage length and a least count of 0.0001 inch. Strains could be recorded to the nearest 10 micro inches per inch. The initial readings were taken while the beam was still intact and before any sawing or cutting operations were started. Final readings were made after the strips had been sawed out. The change in the length of the strips represented the average longitudinal strain in each strip. The corresponding average stress was determined by multiplying this strain by an assumed modulus of elasticity of 29,000,000 psi.

As has been noted, shear lugs had been welded on the compression flange, and a concrete slab had been cast to form a composite beam. Due to the discontinuity of residual stresses effected by this welding, no attempt was made to determine stresses in the top flange or in the web. These stresses were judged to be irrelevant as far as the results of these tests were concerned. Therefore the internal stress distribution was evaluated only in the bottom cover-plated flange of these specimens.

## RESULTS

The residual stresses measured by this procedure were developed in the specimens during fabrication. Included were the effects of rolling and cooling, prestressing, and welding. Subsequent loading of the specimens caused some plastic flow and therefore a certain amount of redistribution of residual stresses. The values determined, therefore, were the end results of the entire stress history of the specimens.

The final state of stress was evaluated in the bottom flange and cover plate of each specimen (figures 22 to 25). The intended internal stress contribution of the prestressing operation, and the theoretical prestress values were also determined.

Although the method employed yielded quantitative results in the interpretation of the values, the residual stresses determined for each strip were considered simply an average longitudinal stress along a 10-inch gage length. Within a particular strip, the extreme values, which are particularly significant in a fatigue test, were not determined. Adjacent to the weld the stress gradient was usually found to be very steep. What might be the actual shape of this stress gradient has been superposed on the results obtained from segment 1 of the Des Moines beam cover plate (figure 27). The suggested curve indicates the maximum tensile stress at the welded edge of the plate was somewhat higher than the value measured. In addition, much variation of average residual stress values was found in successive strips of the same continuous weld (figure 26). In the weld area, much larger stress value deviations might be found at geometrical or metallurgical discontinuities in any selected strip.

A general tendency of the residual stresses to diminish toward the center of the span was observed (figures 22 to 26). The maximum cyclic stresses have been shown superposed on the residual stresses found for each strip. Generally, where the largest superposed maximum cyclic stress occurred (near midspan), the total stress (residual stress plus maximum cyclic stress) was the smallest. Therefore, the residual stress appears to have relaxed most where the applied stress was the largest[42].

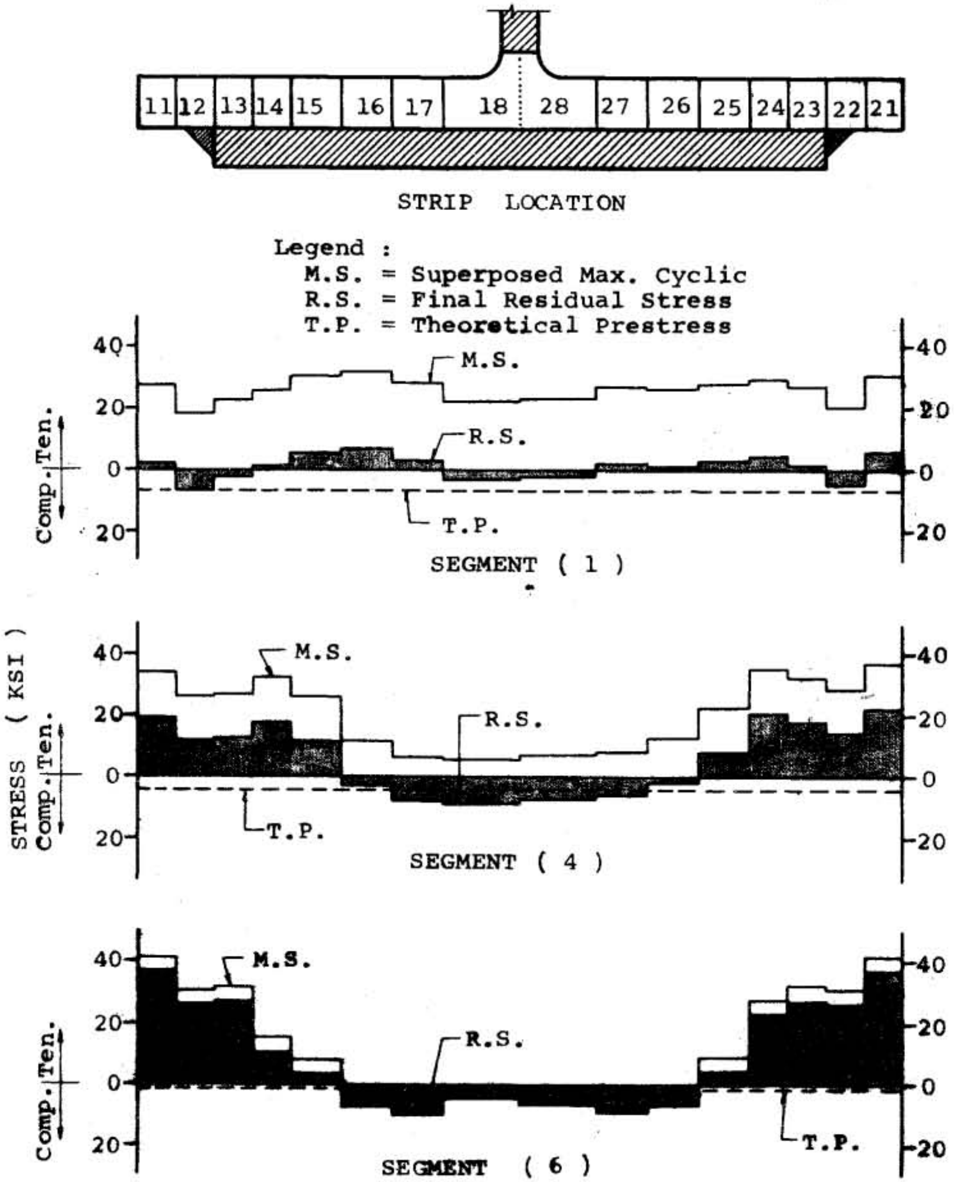



FIGURE 22. STATE OF STRESS IN BOTTOM FLANGE OF DES MOINES BEAM

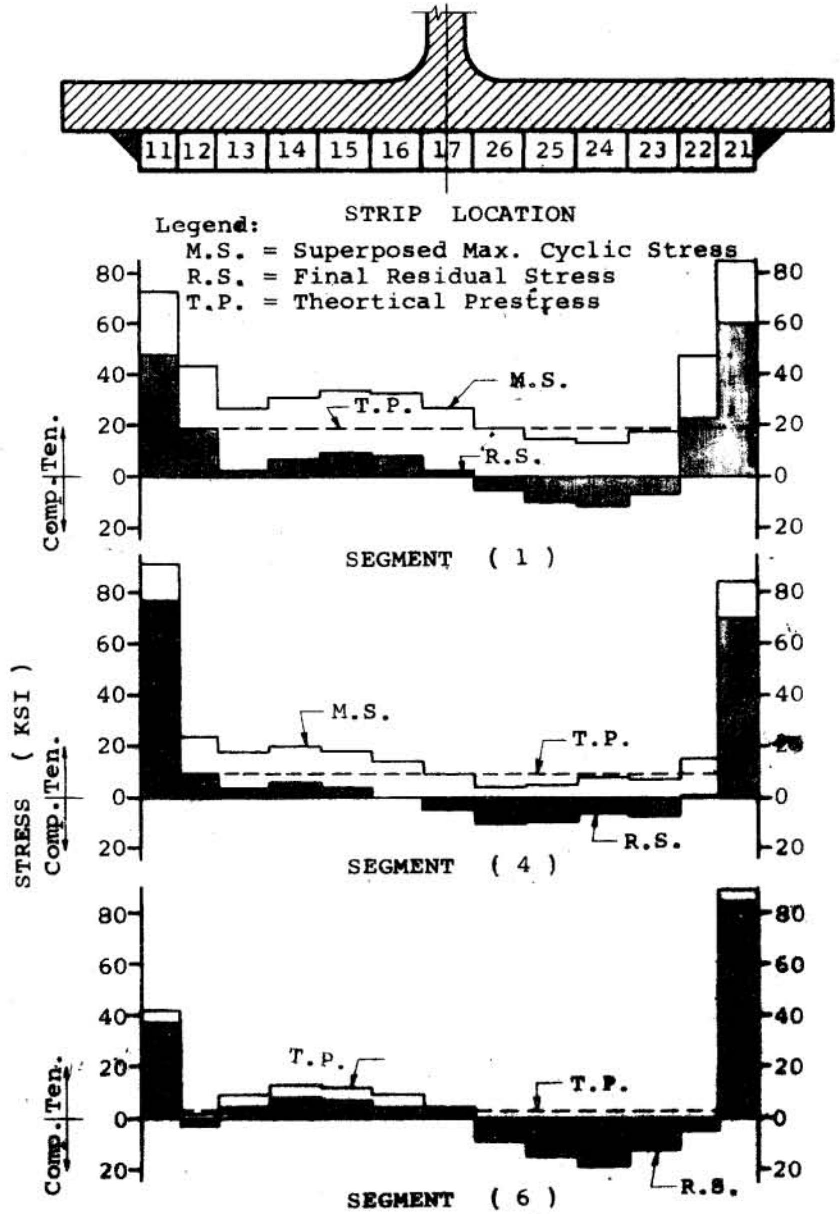



FIGURE 23. STATE OF STRESS IN COVER PLATE OF DES MOINES BEAM

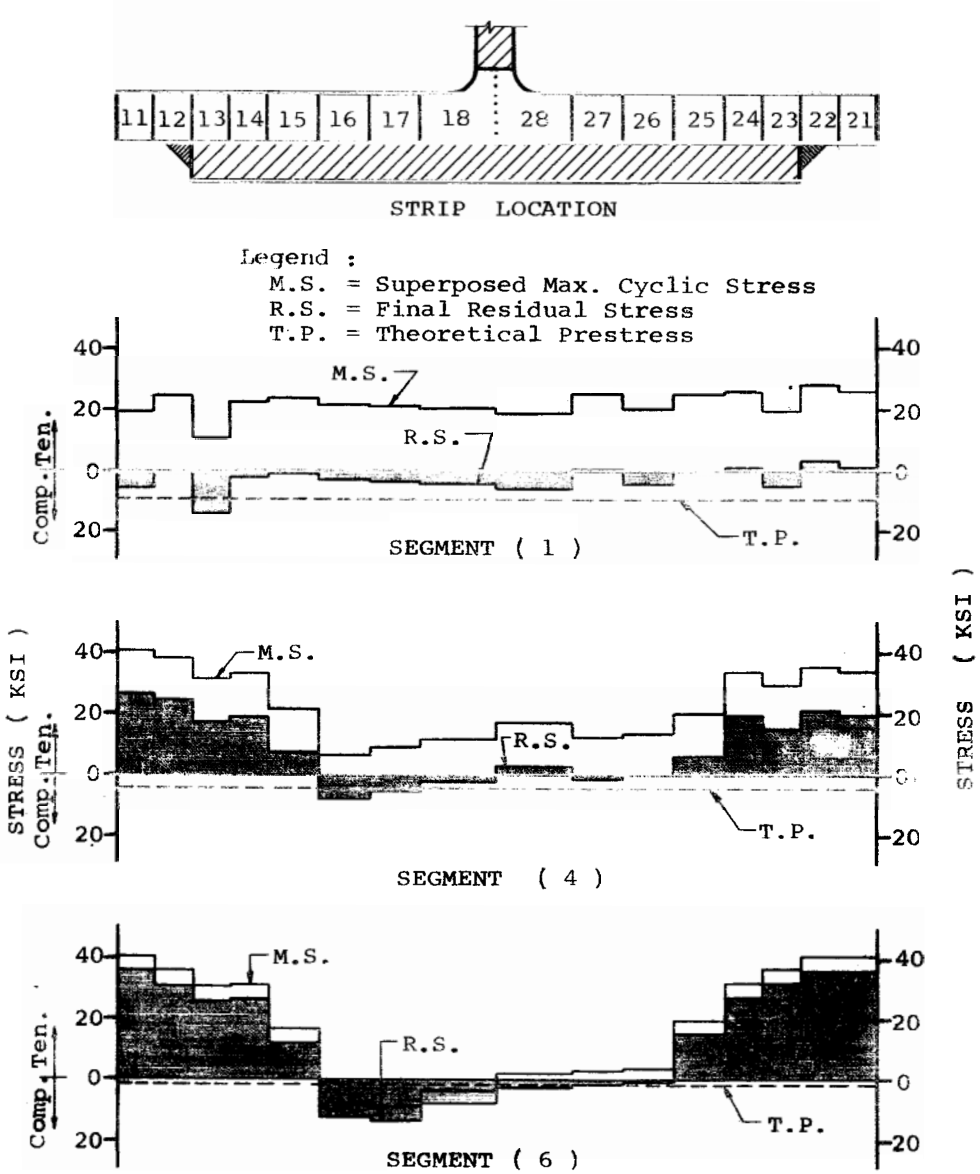



FIGURE 24. STATE OF STRESS IN BOTTOM FLANGE OF AMES BEAM

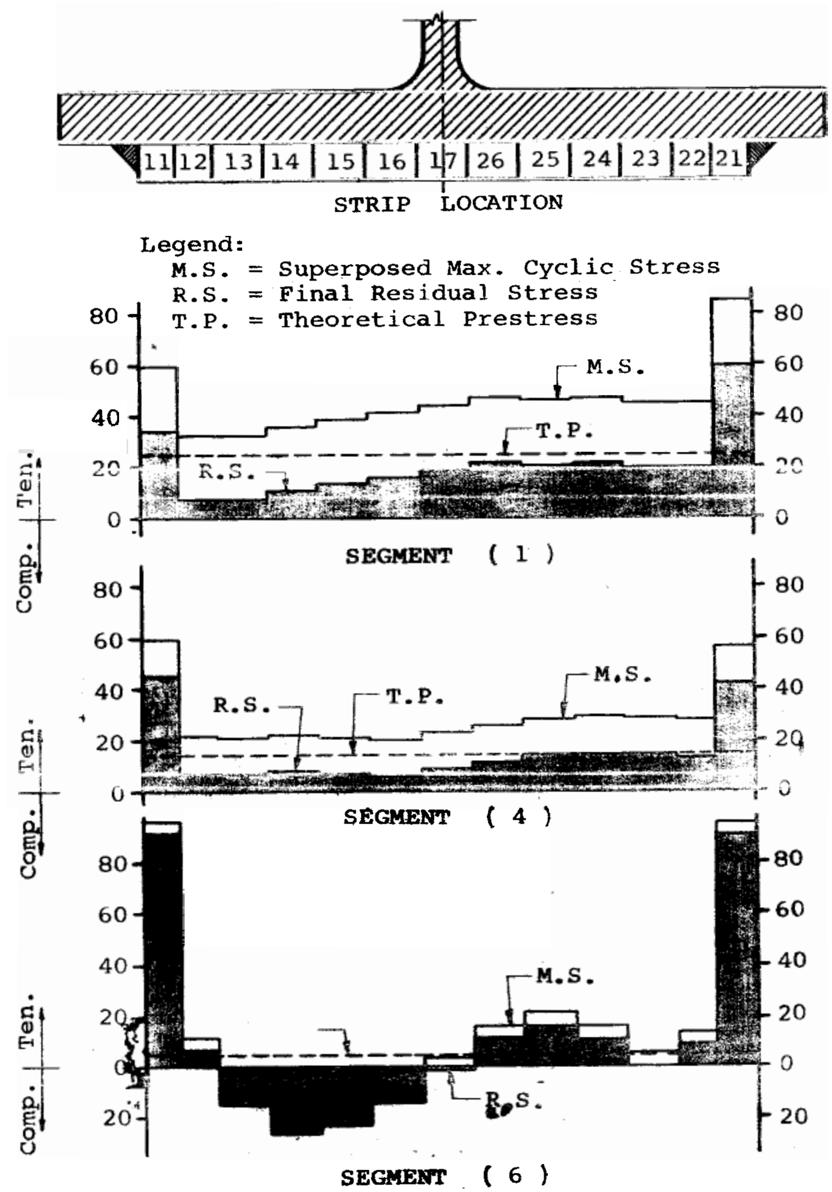



FIGURE 25. STATE OF STRESS IN COVER PLATE OF AMES BEAM

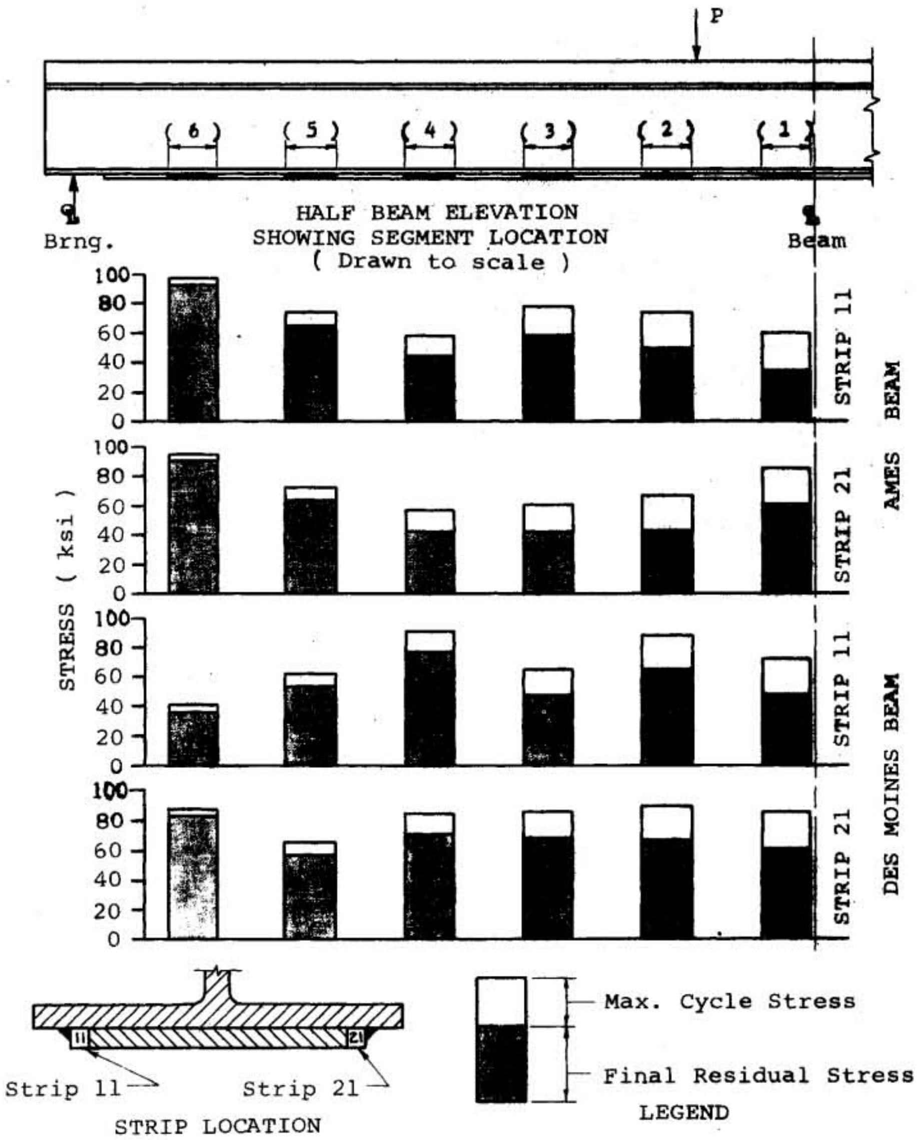



FIGURE 26. STATE OF STRESS ALONG WELDED EDGE OF COVER PLATE

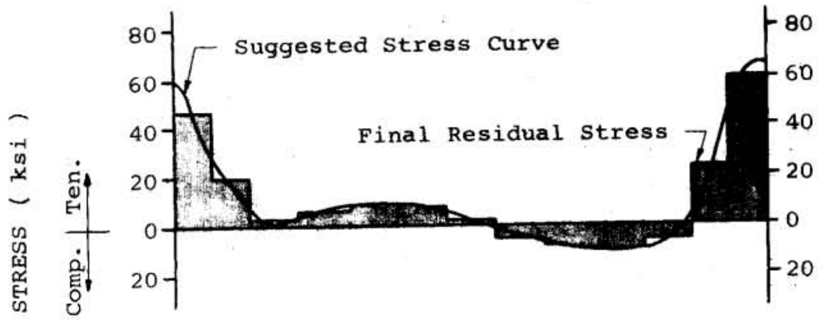



FIGURE 27. SUGGESTED SHAPE OF STRESS CURVE FOR SEGMENT 1 OF DES MOINES BEAM COVER PLATE

## DISCUSSION OF RESULTS

A comparison was made of the performance of these specimens with what might be expected from this same rolled shape and concrete slab with the welded cover plate omitted. On the basis of theoretically determined stresses (minimum flexural stress = 15.3 ksi, maximum flexural stress = 35.9 ksi) and comparison with the fatigue data for A36 steel, this reduced section would probably never have failed under the same loading. This is true though each cycle of load would have applied stresses up to the yield point of the steel. While the static strength would be reduced and the deflection increased, the fatigue strength would be increased.

A theory has been proposed that the loss of strength found in a section with a welded cover plate, as compared with the loss in a rolled beam, is due to the effects of the residual welding stresses[46]. According to this concept, if the tensile residual stress is added to the applied stresses, the true operating stresses are obtained. If the resulting range of stress is plotted on a modified Goodman diagram for as-received plates, failure of the welded member will be indicated after about the same number of stress applications as found for as-received plates.

Diagrams have been prepared to check this proposal using the results obtained in these tests. For the Ames beam this procedure has been performed for the strip designated AP-21 (figure 19). The theoretical load-stress diagram assumes an initial prestress of 24.7 ksi with no load. The minimum and maximum cyclic loads produced flexural stresses in the selected strip of 10.1 ksi and 25.2 ksi respectively, or theoretical cyclic stresses ranging from 34.8 ksi to 49.9 ksi. If the applied flexural stresses of 10.1 ksi and 25.2 ksi, however, are added to the measured average residual stress in strip AP1-21 of 60.0 ksi, the cyclic range of stress becomes 70.1 ksi minimum to 85.2 ksi maximum.

The same procedure was followed in determining the results obtained from the Des Moines beam (figure 28). Strips DP1-21, DP2-21, and DP4-11 were selected for this presentation.

The diagram of the results for neither beam indicated failure for any of the selected strips at less than 2,000,000 cycles of load application. Nor did failure actually occur in any one of these strips. However, residual microstresses in excess of the measured average stresses were indicated. Therefore, the lack of correlation found in the diagrams between the proposed theory and the results of this study was attributed to the inability of the investigator to determine the residual microstresses. The condition of the welds suggested a severity of stress discontinuities not normally found in similar weld constructions. Had it been possible to determine accurately the residual stress, good correlation might have been found.

This theory suggests that whether the material is smooth, welded, or notched, the fatigue strength of the material depends entirely on the state of stress at the point of failure. When the tensile residual stresses can relax, or can be made to relax, the fatigue strength should be improved. (See Appendix.)

Fatigue failures are generally the result of a localized microstress condition. The tests carried out in this program determined the fatigue strength of the specimens only at

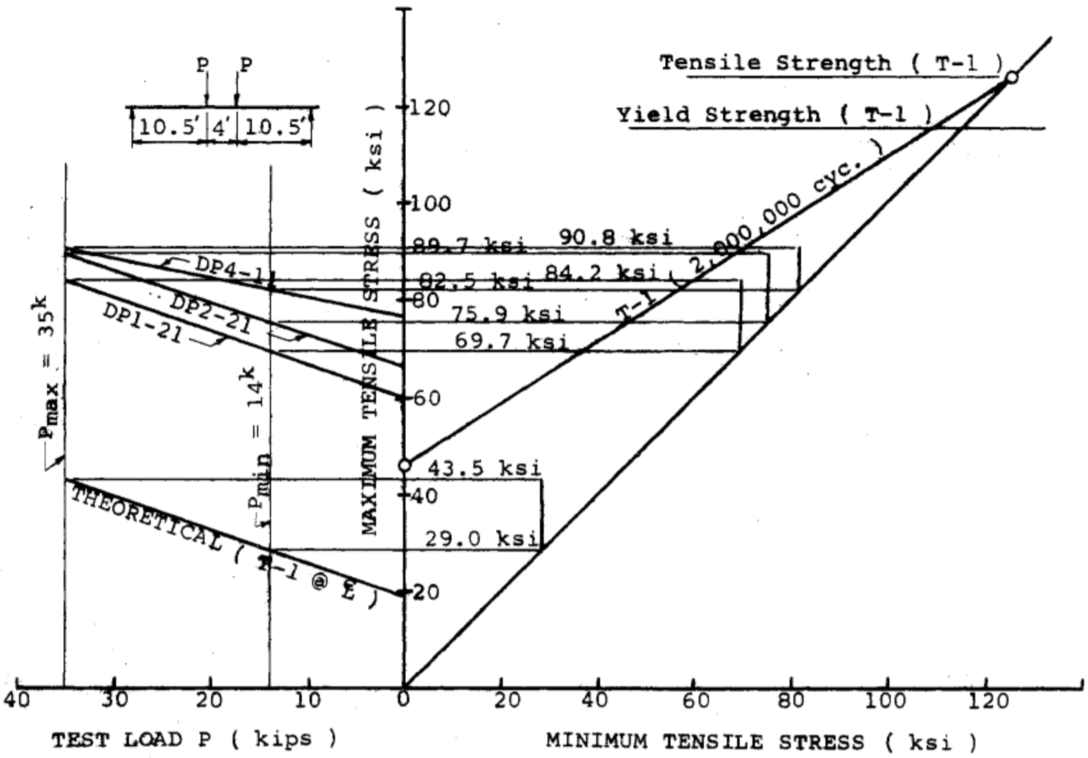



FIGURE 28. STRESS-LOAD AND FATIGUE STRENGTH DIAGRAM FOR T-1 COVER PLATE OF DES MOINES BEAM

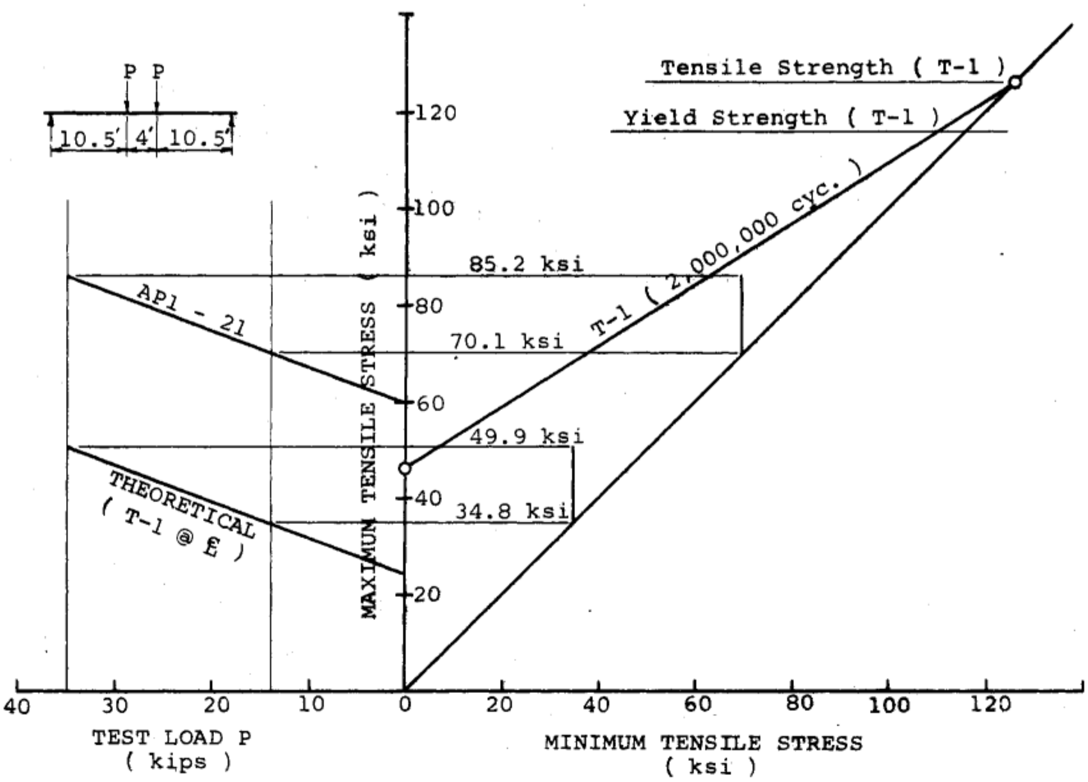



FIGURE 29. STRESS-LOAD AND FATIGUE STRENGTH DIAGRAM FOR T-1 COVER PLATE OF AMES BEAM

the precise location where the crack appeared. In each fatigue test, therefore, the entire beam was not evaluated, but only the weakest point in the beam at a particular loading.

This is of significance when one considers the condition of the welds and the residual stress concentrations associated with weld flaws such as those found on the two specimens. The average residual stress values reported were extremely high and could readily be expected to contribute to premature failure. But the notch effects in the weld area suggested strongly that the residual microstresses were far in excess of the average values measured. In the high strength steel some readjustment of the initial residual stresses probably took place as plastic flow or slip, but the readjustment was not enough to relax the stresses to safe values. Since the applied stresses were added to the high tensile residual welding stresses, failure in the resulting stress range is not surprising, particularly at a stress concentration.

This "weakest plane" where failure occurred does determine the strength of the entire specimen. Therefore in a set of given conditions, the larger the specimen, the more likely becomes the possibility of a critical flaw or group of flaws forming a plane of weakness.

In welded members the possibility of the occurrence of such flaws and defects is inherent. It must be conceded that the stress raisers in the welds were exceptionally severe in these specimens. On neither specimen did the welding conform to the minimum requirements of the American Welding Society[3].

Perhaps members similar to those tested would have peformed adequately in service. The static ultimate strength of the specimens, though not determined, would probably have measured up to the expectations of the designer. And even 600,000 cycles of stress repetition at full design load may be quite satisfactory. The relationship between the frequency of application of a load in a constant-load fatigue test and actual service loads is not clearly established.

The increase in strains and deflections, and the general decrease in residual stress, are indications that some redistribution of internal stress took place in these tests. It is likely that most of this redistribution came as the result of plastic flow in the weld area. The extent of this fading of the residual welding stresses was not determined. The picture in this respect is clouded by hysteresis, work hardening, and other effects.

As previously pointed out, the specimens were prestressed to obtain a more favorable stress pattern, thus allegedly utilizing more fully the properties of the high strength alloy steel. The influence of this prestressed condition on the behavior of the specimens under repeated load is not indicated clearly. The residual stress evaluations indicated that most of the internal stresses "locked in" by the prestressing operation were undesirably concentrated in the weld area. The same general pattern would be expected in an un-prestressed beam, although the residual tensile stresses might be higher in the weld area of the A36 steel at midspan. The relative fatigue strength might then depend on whether relaxation of these welding stresses can occur in the un-prestressed beam.

## CONCLUSIONS

Several conclusions may be drawn from the results of these tests. While the number of specimens was much too limited to justify a quantitative analysis, taken in relation to data and observations previously reported, the following observations are valid:

1. The high residual stresses in the weld area contribute substantially to eventual fatigue failure at points of high stress concentration. Notch effects of many types are commonly found in welded construction, and fatigue damage may occur at such stress raisers before residual stresses can relax to safe levels.
2. The applied stress values superposed on the measured final residual stress values determine a stress range. This range plotted on appropriate modified Goodman diagrams for as-received plates gives a useful explanation of the effects of residual welding stresses on fatigue strength.
3. In the design of welded built-up members the usefulness of modified Goodman diagrams for as-received plates such as presented herein is limited by the indeterminate nature of the residual welding stresses. A more direct approach based on modified Goodman diagrams for butt-welded plates may be practical, but further study is required.
4. When subjected to repeated loads a certain amount of relaxation of the high tensile welding stresses takes place in beams of this type, both in the A36 and "T-1" steels.
5. Impending failure may not be detected by deflection or strain measurements, even after a sizable fatigue crack has formed.
6. The fatigue results of these tests were not adversely affected by the inelastic behavior of the concrete slab on the compression flange.
7. Further economy in the field of welded member design depends on the development of a sure and economical method of reducing the loss of fatigue strength associated with the welding process (see Appendix).

No firm conclusions regarding the benefits associated with the prestressing operation used in the preparation of these specimens could be drawn. Theoretically the process should result in a more economical use of the materials, but this seems to be offset somewhat by the effects of welding on the high strength steel.

Fatigue strength has not been generally accepted as a design criterion, but there can be no doubt as to the effect of repeated loading on the service life of the structure. The AWS Specifications[3] provide guidance with respect to what constitutes good welded design for fatigue; but the interpretation and application of these requirements are left to the engineer. A basic knowledge of fatigue behavior will therefore serve as an aid in ensuring that structures are designed for efficiency and endurance.

Future studies may show that significant improvements in welded design may be realized by one or more of the methods outlined in the Appendix or perhaps some other procedure may yet be developed. Because of costs of this type of construction, an intensive research program is warranted. The contribution to society of the structural engineer depends not only on the quantity but also on the efficiency and quality of the structures he designs.

ACKNOWLEDGEMENTS

The investigation reported herein was carried out at the Iowa Engineering Experiment Station at Iowa State University. The Highway Research Board of the Iowa State Highway Commission was the sponsor.

The facilities of the Research Center of the Association of American Railroads were made available for the fatigue testing portion of this study. Of particular value were the interest and cooperation of E. J. Ruble, Executive Research Engineer, and Freeman P. Drew, Research Engineer, Structures. The staff of the Structures Laboratory performed the setting-up and fatigue testing operations expeditiously and competently.

The preparation of the specimens, including the forming and casting of the slabs, all necessary welding and loading for shipment, as well as the removal of segments and strips for the residual stress determinations, was accomplished by personnel of the Engineering Shop of the Iowa Engineering Experiment Station. The shear lugs were furnished by the Pittsburg-Des Moines Steel Co., Des Moines, Iowa.

The advice and assistance of Frank A. Easton, who has had much experience in bridge construction, were helpful.

Several individuals on the staff of the Iowa State Highway Commission were of assistance. The interest and cooperation of S. E. Roberts, Head of the Research Department and C. A. Pestotnik, Bridge Engineer, were appreciated, as were the contributions of Y. H. Gee, John Roland, and Jim Johnson. Especially helpful was the advice of P. F. Barnard and W. D. Reneker.

# APPENDIX

## WELDING AND FATIGUE STRENGTH

Welded cover plates or girders are widely used and constitute a very practical mode of construction. However, the welding process by its very nature produces residual stresses and notch effects which lower fatigue strength appreciably. The efficiency of welded design would be much improved if the original fatigue strength of the parent of base metal could be restored in an economical and dependable manner. Therefore, a study was made to evaluate the various methods which have been proposed to increase the fatigue strength of welded members and to suggest avenues for further research.

### WHY RESEARCH IS NEEDED

Structural design in steel leaves much to be desired so far as fatigue strength is concerned. A significant loss of fatigue strength invariably results when welding procedures are employed in the tensile region of a structural member. This loss of strength is not reflected in the specifications so far as welded cover plates are concerned[1, 3]. Two members, therefore, designed to the same specifications may have the same static strength; but the fatigue strength or service life of the two members may be much different.

In the last decade a number of high strength constructional alloy steels have been developed. The fatigue properties of the newer steels reveal that they are influenced to a greater degree by welding or notch effects. While the static strength is increased appreciably, the fatigue strength may be increased only slightly, if at all[16].

This does not mean that current practice is unsafe. The point is that the most efficient use is not being made of the materials. The specifications are intended to insure that a minimum degree of safety is maintained in every case. It may be that at least some modification of the specifications may be in order to correct this deficiency.

However, knowledge of fatigue phenomena has not developed sufficiently to justify substituting fatigue properties for static properties as a design criteria. Allowable stresses for such a modification would probably have to be based on interpretation of a modified Goodman diagram, not only for each steel but also for each detail of fabrication. (To some extent this has been done for A7 steel[3, pp. 53-65]). The development of a comprehensive set of Goodman diagrams would be an enormous task, and their use might prove very complicated and perhaps even confusing. "It must be agreed that present analytical methods for fatigue design are, by themselves, inadequate to provide an efficient structure that is safe from fatigue failure[25, p. 121].

### RESEARCH POSSIBILITIES

Since large residual welding stresses commonly occur and the fatigue strength of members when welded generally decreases, several investigators have tried to find how to reduce these residual welding stresses or their accompanying notch effects to increase fatigue life. Several methods have been used with varying degrees of success. These methods might be grouped into two general categories: redistribution, or relief of residual welding stresses, and reduction of notch effects due to welding.

It is usually difficult to estimate the economy of a particular method. Each process costs an additional amount, which must be balanced against increase in fatigue strength. Since the specifications currently provide for the same allowable stress for rolled beams as for welded beams, the same section is usually required whether or not the fatigue strength is improved. Therefore the designer must determine the economy of a method in terms of increased structure life and more efficient use of materials.

Redistribution of residual welding stresses

In general the procedures in this category rely on the occurrence of a certain amount of plastic flow in the weld area. This yielding results in a relaxation or redistribution of the residual stresses due to welding. In many cases an increase in fatigue strength is then experienced.

Low temperature stress relief. Low temperature stress relieving methods were developed during World War II in the shipbuilding industry[22]. Stresses of 40 to 50 ksi were reduced to about zero in butt-welded plates of mild steel. This was accomplished by heating strips on each side of the weld to about 350°F. The expansion of the heated zones then forced plastic flow in the unheated weld area.

Recently two Russian investigators have demonstrated that this general method can be utilized to increase the fatigue strength of welded built-up members of mild steel[15]. Following a heat treatment, longitudinal welds, originally under high tensile residual stress, were found to be under compressive residual stress. Tensile residual stresses were set up in the unnotched portions of the section away from the welds, thus improving overall fatigue strength about 20%.

The chief value of this procedure would seem to lie in the relief of high stress concentrations such as are found at the ends of welds, where increases in fatigue strength of 100 - 600% are reported[15]. For this reason the method may be particularly useful in repair work.

Additional study will be required to determine how satisfactory this method will be as a standard procedure for built-up members. A jig could be designed to hold the torches. The beam might then either be moved through the jig, probably on rollers, or the jig could be moved along the beam similar to an automatic welding machine.

No studies of this method are known to have been conducted on high strength steels. Presumably a similar increase in fatigue strength could be expected.

High-temperature stress relieving. Attempts to increase fatigue strength by reducing residual welding stresses by high temperature stress relieving have been very successful[49, pp. 99-100]. This method depends on the fact that the yield point of the steel at high temperature (about 1200°F) is lowered and plastic flow takes place in areas of high residual stress. It has become a rather standard practice to relieve stresses in welded pressure vessels in this manner[53, pp. 18-21]. This process is expensive, and care must be exercised to keep the cooling rate slow enough that new residuals are not developed.

The tendency of shapes to warp during this process may create fabrication difficulties. Generally this method is not applicable to welded structural beams.

Prestraining. In experiments on wide plates of mild steel, investigators have found it possible to reduce the residual welding stresses by a tensile preloading or prestraining[36, 57]. It was reported that release of a tensile load in the direction of a weld would reduce the residual welding stresses in an amount approximately equal to the applied nominal tensile stress. The yielding which takes place in the weld area is evidenced by a permanent "set" in the member. On mild steel a slight increase in fatigue strength was noted after this treatment.

The "Bauschinger effect" has been shown to increase fatigue strength provided the stress direction is not reversed[12]. This requirement makes the prestraining method particularly applicable to bridge beams, since stress reversal occurs only in areas of reduced moment.

Further prestraining should prove most useful for high strength steels, but the steel must be capable of strain-aging in order that this beneficial effect be realized.

On the other hand, in applying this method to welded steels, particularly high strength welded steels, it is assumed that the required plastic flow can actually take place. The available ductility of the metal may be exhausted by this process; and embrittlement may result, especially in a weld area. This is, in effect, the cause of initial weld cracking[41, pp. 338-339].

Therefore further laboratory investigation of this method is required for high strength steels. While the procedure is relatively simple and inexpensive, the effects on fatigue life are not apparent, and the author knows of no investigations in this regard.

Miscellaneous procedures. Coaxing, peening, and cold working have been used, primarily in the machine design field, to improve fatigue strength[34, 48]. These methods are used to set up favorable compressive residual stresses on the surface of the member. The stresses act to reduce the tension stress of an applied load at the point where fatigue cracks might be formed. However, it is difficult to determine how much improvement in fatigue strength is due to the residual stresses so developed and how much to the hardening of the material. These procedures are not generally applicable to bridge members, although a type of peening which may be useful has recently been tested for ship construction[10]. Fatigue strength is said to be restored to 70 to 80% of that of the base material following a drop after welding to 50 to 60%.

Reduction of Notch Effects

The methods used to reduce notch effects are directed to controlling the formation of stress concentrations or to reducing their effects on fatigue strength. The procedures make no attempt to alter the internal stresses of a member once they are formed.

Improved geometry. Several tests have shown that the quality, shape, and position of the weld bead have an effect on fatigue strength[23, 26, 50, 9]. Generally these studies have shown that fatigue strength can be improved by:

1. removal of weld reinforcement
2. use of proper electrode
3. depositing the weld in a flat position
4. inclining the weld 45° to the direction of tension.

In tests at Iowa State University the effects of the ratio of longitudinal weld bead area to plate area on fatigue strength of mild steel will be investigated. The premise is that a reduction in the size of the weld may also reduce the resulting notch effects or stress concentrations.

Current practice usually requires only that welds in welded bridge members be deposited in accordance with accepted procedures. Quality control of the end product continues to be a problem in spite of advancements in the field of non-destructive testing. Several new and sophisticated techniques have been introduced, but the problem is a difficult one for the bridge engineer because of the shape and location of the welds and the size of the members involved. The problem is of such importance that constant review and study of the various techniques is warranted.

Use of surface coatings. Another method of improving fatigue strength may be found through the use of surface coatings. A German investigator has found that the application of an epoxy coating to the weld and weld-affected areas of butt-welded plates will increase their fatigue strength about 75%[19]. This improvement is attributed to the smoothing effect of the epoxy coating and to the reduction in the notch effects found at a weld. Another explanation might be that stress corrosion is often associated with fatigue failure. If the latter, an epoxy coating would shield the material and substantially slow the process of stress corrosion.

Supporting evidence may be found in reports published of studies at the National Bureau of Standards[18, 28]. These tests showed that an improvement in fatigue strength may be realized by coating the specimens with an oleophobic film, a film which is unwetted by all but the lowest-boiling hydrocarbons. These same films are also hydrophobic and are effective in preventing corrosion. They form a tightly packed monomolecular film on metal surfaces. The improvement in fatigue strength was greatest when the coating compound contained a carbon chain of at least twelve.

A practical coating must have a greater fatigue strength than the metal itself, preferably over a long period of time. It must remain impervious to atmospheric conditions and must maintain these properties over a fairly wide range of temperatures.

Present theory of the mechanism of fatigue failure indicates that fatigue is primarily a surface phenomenon and that fissures appear early in the fatigue process[21]. It is likely that a film which forms an effective barrier to oxygen and water molecules will reduce or eliminate corrosion in the stress concentration formed around these fissures.

Tests reported on an unalloyed steel indicated that this preliminary explanation of the improvement noted may not prove satisfactory[27, pp. 400-401]. Specimens were fatigue tested in purified and dried gases, oxygen, nitrogen, hydrogen, and argon, as well as in laboratory air. Very little difference in fatigue strength was noted.

Therefore investigations of the use of surface coatings may result not only in the development of a very practical and significant method of improving fatigue strength, but may also lead to further understanding of the basic mechanism of fatigue failure. Although much additional study will be required, preliminary investigations are very encouraging.

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# PART 2

Iowa Highway Research Board

Project HR - 74

# LIVE LOAD DEFLECTIONS IN A PRESTRESSED STEEL BEAM BRIDGE

Prepared by

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December, 1963

# ABSTRACT

This report describes the measurement of dynamic (live load) deflections in a 240' x 30' three span continuous prestressed steel bridge, skewed 30°. The design assumptions and prestressing procedure are described briefly, and the instrumentation and loading are discussed. The actual deflections are presented in tabular form, and the deflections due to the design live load are calculated. The maximum deflections are presented as a ratio of the span length, and the further use of prestressed steel beams is recommended.

LIVE LOAD DEFLECTIONS IN A
PRESTRESSED STEEL BEAM BRIDGE

By Neil Welden

The subject of this investigation is a 240' x 30' three span continuous prestressed steel beam bridge, skewed 30° clockwise, over Silver Creek on US Highway No. 6 in Pottawattamie County, Iowa. The spans are 73'3 - 93'6 - 73'3. The slab is supported by four lines of beams, spaced 9'8 center to center, spliced 22'0 each side of each interior support, near the point of dead load contraflexure. The exterior beams are 30" wide flange 108#, and the interior beams are 33" wide flange 130#. All beams are A.S.T.M. A-36 steel, and all have high strength steel [(1)] cover plates. The design live load is AASHO H20-S16-44. Live load moments causing tension in the bottom flange and compression in the top flange of the beams are resisted by composite action of the beam and slab. The structure will hereafter be referred to as the Silver Creek bridge.

The prestressing principle used in this structure was conceived by Mr. P. F. Barnard, Structural Engineer in the Bridge Design Division of the Iowa State Highway Commission,

(1) USS T1, a quenched and tempered weldable alloy steel, which has a yield strength of 100,000 psi and a working stress of 55,000 psi.

in July 1960. Briefly, it consists of loading a beam by jacking to produce stresses, below the elastic limit, opposite in sense to those produced by structural loads. With the beam so stressed, a T1 cover plate is welded to the compressed flange of those beams which will resist live load moments by composite action with the slab, and T1 cover plates are welded to both flanges of those beams in which composite action is not considered effective. The load is then released, producing stresses in the beam and cover plate which are the same as those produced in the cover plated beam by a load equal in amount, and opposite in direction, to the jacking load.

Since all stresses are below the elastic limit, the resultant stresses in the beam and cover plate are the algebraic sum of the jacking stress and the release stress. The resultant stresses in the beam are opposite in sense, and in the cover plate identical in sense, to the stresses caused by structural loads. In the stucture, the structural loads first reduce the resultant stresses in the beam to zero, and then increase them to the working stress in the opposite sense. The resultant stress in the cover plates is increased by the structural loads, but the high allowable working stress of the T1 plates enables them to tolerate this increase. If a proper choice of beam section, cover plate size, and prestress loading is made, the maximum stresses under structural loads will approximate the allowable working stresses in the beams and cover plates.

The Iowa State Highway Commission was favorably impressed by the prestressed steel concept, and authorized the Bridge Design Division to use prestressed steel in the deisgn of the Silver Creek Bridge. The United States Bureau of Public Roads considered the concept experimental, and declined the use of Federal funds; the project was therefore made non-participating.

In order to establish fabrication procedures, to determine load deflection characteristics, and to check actual stresses with the analysis, we decided to fabricate prototype beams, and Mr. Barnard designed two, one with a single cover plate, and one with two cover plates, one on each flange. The United States Steel Corp., Bethlehem Steel Co. and Inland Steel Co. furnished the material, and the beams were fabricated in the Des Moines, Iowa shops of the Pittsburgh-Des Moines Steel Co., with a group of fabricators (2) sharing the cost.

The first pair of beams were prestressed by jacking, blocked in that position, and the cover plates completely welded to the beams. The results were disappointing, since the distortion due to welding while the beam was held by the blocks reduced the prestress considerably. Mr. Barnard suggested that this effect could be minimized by prestressing the beams and tack welding the cover plates to them, using only sufficient weld metal to

(2) American Bridge Division of the US Steel Corp., Clinton Bridge Division of the Allied Structural Steel Co., Des Moines Steel Co., Missouri Valley Steel Co., and Pittsburgh-Des Moines Steel Co.

assure interaction between the beams and cover plates. The prestress load would then be released, and the weld completed with the beams free to deflect. A second pair of beams was fabricated in this manner, and strain gage measurements showed that the stresses agreed closely with the analysis.

In order to compare the effect of welding on prestressed and unprestressed beams, material was obtained for another single plated beam, and this beam was fabricated in the shops of the Des Moines Steel Co., Des Moines, Iowa using procedures identical with those used on the single plate prestressed beam, except that the beam was not prestressed.

These three beams - two fabricated by the prestress - tack weld - release - weld procedure and one fabricated without prestress - were taken to the laboratory of the Materials Department of the Iowa State Highway Commission, where they were loaded to determine the load-deflection characteristics. All beams, on first being loaded, showed a small permanent set, but subsequent loads within the same range showed excellent proportionality. No significant difference was found in the performance of the single plate prestressed and unprestressed beams.

Mr. Barnard presented a report to the National Engineering Conference, sponsored by the American Institute of Steel Construction, in Minneapolis, Minn., on May 11, and 12, 1961, dealing with the prestressed steel concept and the fabrication and testing of the prototype beams. The report was published in the "Proceedings" of that conference.

After the prototype beams had been tested at the Iowa State Highway Commission, they were turned over to the Civil Engineering Department of Iowa State University, Ames, Iowa, where they were subjected to fatigue tests. The results of these tests were published by the Iowa Engineering Experiment Station in a report entitled "Flexural Fatigue Strength of Prestressed Steel Beams" by W. Daniel Reneker and Carl E. Eckberg, Jr., Project 435 S, dated May 1, 1962.

Preliminary designs for the Silver Creek bridge had been made by Mr. R. D. Upsahl and Mr. Domingo Silverio of the Bridge Design Division of the Iowa State Highway Commission. When fabrication and testing of the prototype beams had shown that fabrication was feasible if proper procedures were used, and that the load-deflection characteristics of these beams conformed to the theoretical analysis, the design was completed, and the structure was let on February 14, 1961. W. H. Herberger, Indianola, Iowa, was the successful bidder, and the steel was fabricated by the Pittsburgh-Des Moines Steel Co., in Des Moines, Iowa.

Fabrication was inspected by Mr. Robert Brandser, Mr. Gordon Gwinn, and Mr. Harold Chandler, of the Des Moines laboratory of the Materials Department, Iowa State Highway Commission. Detailed instructions for prestressing, including jacking loads, anticipated deflections before and after release, and welding details, were included in the design drawings. The beam deflections were checked by a surveyors level telescope attached to one end of the beam sighting a target at the other end and a scale at the center line. When the calculated deflection was reached, T1 cover plates were tack welded in position. The load was then released, and the weld completed. No unexpected difficulties were encountered in fabrication.

Construction was under the supervision of Mr. R. H. Given, District Engineer, with Mr. David Skaff, Resident Construction Engineer, in charge of the project. No special construction procedures were required, except that reasonable precautions were taken to avoid dropping or jarring the beams.

The bridge was completed and accepted on May 10, 1962. It has been in service since that time and is operating satisfactorily. To an observer on the bridge, deflections and vibrations during the passage of a loaded truck are perceptible, but not objectionable.

The AASHO "Standard Specifications for Highway Bridges" limits the computed deflections due to live load and impact to 1/800 of the span, where the span is defined as the distance center to center of bearings. For this bridge, the computed deflection due to live load and impact at the center line of the center span is 1.36". The span is 93'6, which gives a ratio of 1/825. This meets the AASHO specification, but the calculation depends on assumptions of the modulus of elasticity of the concrete, and of the extent of composite action between steel and concrete.

Furthermore, the AASHO specification reguires that rolled beams used as girders have a depth greater than 1/25 the span, where the span is defined as the distance between points of dead load contraflexure in continuous beams. In this bridge, the exterior beam is slightly more than 30" deep, including the cover plate. The distance from the abutment bearings to the point of dead load contraflexure in the end span is slightly greater than 50', and the distance between points of dead load contraflexure in the center span is slightly less than 50'. This gives a depth to span ratio of about 1/20, which meets the AASHO criterion, but indicates a relatively flexible beam.

Since the computed deflection and the depth - length ratio approach the AASHO specification limit, we considered it advisable to measure the deflections of the bridge under a known live load which approximated, or could be related to, the H20-S16-44 truck used in design. The present project, designated HR-74, was therefore proposed to the Iowa Highway Research Board, recommended by the Board, and authorized by the Iowa State Highway Commission.

The investigation was directed by Mr. Lyman Moothart and Mr. Harold Sharpnack of the Materials Department of the Iowa State Highway Commission. After consulting with Mr. P. F. Barnard and the author, Structural Engineers for the Bridge Design division of the Iowa State Highway Commission, they decided:

1. Deflections should be measured on one exterior and one interior beam at 29 feet from the center of each abutment bearing and at the center of the center span.

2. A test should be made to check the lateral symmetry of the bridge.

3. The truck should be a five axle vehicle, loaded to approximate, as closely as possible, the AASHO H20-S16-44 truck used in design.

4. Truck speeds should vary from 3 mph (assumed to approximate a static load) to 50 mph (legal speed limit for trucks in Iowa), each run at a constant speed.

5. Runs should be made with the center line of truck at 4.5', 7.5' and 10.5' from each gutter.

6. Deflections should be recorded in a permanent form, preferably to an exaggerated scale.

In accordance with these criteria, test runs were planned as follows:

Test 1 - Runs 1 to 32, inclusive (item 1)

1. Center line of truck at 4.5' from north gutter
   2 runs at 3 mph
   2 runs at 20 mph
   4

2. Center line of truck at 7.5' from north gutter
   2 runs at 3 mph
   2 runs at 35 mph
   8

3. Center line of truck at 10.5' from north gutter
   2 runs at 3 mph
   2 runs at 20 mph
   2 runs at 40 mph 16
   2 runs at 50 mph

4. Center line of truck at 10.5' from south gutter
   2 runs at 3 mph
   2 runs at 20 mph
   2 runs at 40 mph
   2 runs at 50 mph (3)

32

5. Center line of truck at 7.5' from south gutter
   2 runs at 3 mph
   2 runs at 35 mph

6. Center line of truck at 4.5' from south gutter
   2 runs at 3 mph
   2 runs at 20 mph

Test 2 - runs 33 to 56 inclusive (item 2)

1. Center line of truck at 4.5' from north gutter
   2 runs at 3 mph
   2 runs at 40 mph

2. Center line of truck at 7.5' from north gutter
   2 runs at 3 mph
   2 runs at 40 mph

3. Center line of truck at 10.5' from north gutter
   2 runs at 3 mph
   2 runs at 40 mph

4. Center line of truck at 10.5' from south gutter
   2 runs at 3 mph
   2 runs at 40 mph

5. Center line of truck at 7.5' from south gutter
   2 runs at 3 mph
   2 runs at 40 mph

6. Center line of truck at 4.5' from south gutter
   2 runs at 3 mph
   2 runs at 40 mph

The method of measuring and recording deflections was similar to that used by the United States Bureau of Public

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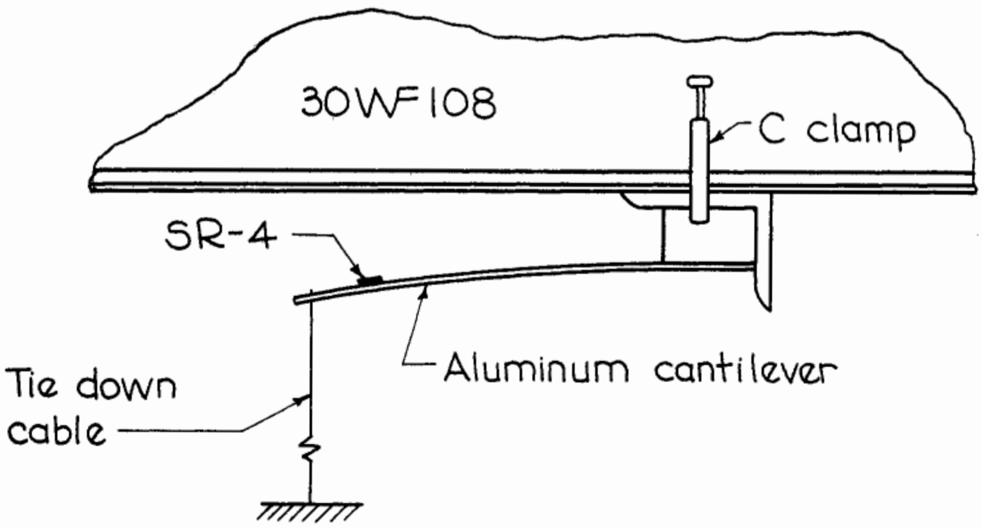
(3) Rough pavement west of the bridge made it impossible to obtain this speed safely. Actual speed is shown in the tabulation of deflections.

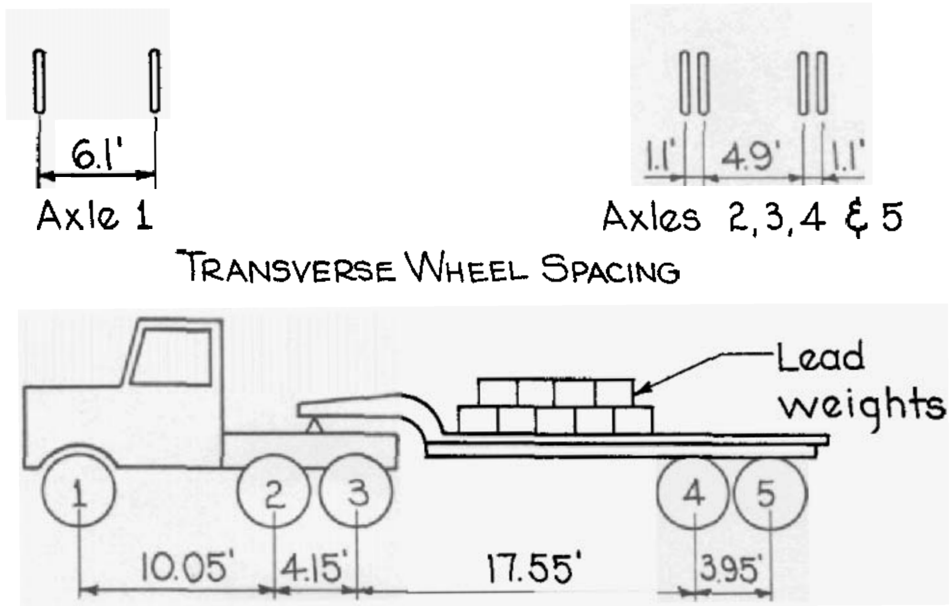
Roads for measuring bridge deflections and vibrations. Instruments used were SR-4 strain gages, with temperature compensation, three Honeywell 130-2C carrier amplifiers, and a 906 C Visicorder oscillograph, capable of recording six traces simultaneously.

The procedure was as follows:

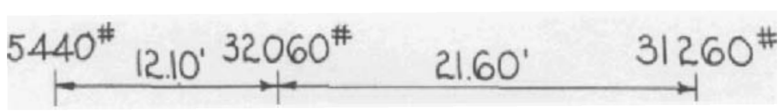
An SR-4 strain gage was attached near the free end of an aluminum alloy cantilever beam, which, in turn, was rigidly attached at the other end to one of the bridge beams so that the cantilever was parallel to the length of the beam, approximately on its center line. The free end of the cantilever was tied down by a light cable to a weight on the ground below the bridge. The cable was tightened until the cantilever had an initial deflection of about two inches. (See figure 1)



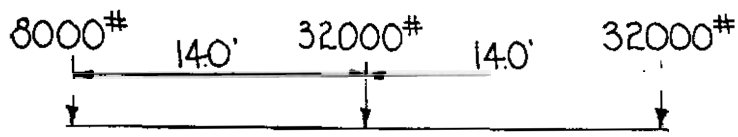

DEFLECTION GAGE
Figure 1




TEST TRUCK
(a)




LOADING TEST TRUCK
(b)




LOADING DESIGN TRUCK
(c)

Figure 2

The SR-4 strain gage and its associated temperature compensating gage were connected to the oscillograph through the amplifiers by a four strand shielded cable.

The equipment was calibrated by a dial gage placed vertically at the end of the cantilever beam. A turnbuckle in the tie down cable was tightened until the dial gage showed ¼" deflection. The amplifier gain was then set so that the oscillograph showed 1" deflection. The gages were calibrated at the start and end of each days run.

As the bridge beam deflects under load, the cantilever deflection changes by an equal amount in the opposite direction, and the strain recorded by the SR-4 gage changes in proportion to the deflection. This change of strain is recorded on the oscillograph tape as four times the deflection, and this multiplication allows the deflection to be scaled to ± 0.01".

The truck used for loading the bridge was rented from Mr. Delbert Grosse of Council Bluffs, Iowa, who drove the truck during the tests. Figure 2(a) shows a sketch of the truck, figure 2(b) shows the loading diagram of the test truck used for calculating maximum moments, and figure 2(c) shows the loading diagram of the H20-S16-44 truck used for design. The test truck was loaded with lead weights, as shown, to produce the loading shown in figure 2(b).

Calculation of the maximum moments produced in the end span and in the center span by the test truck and the design truck showed that the design truck produced about 15% more moment (14.9% in the end span and 15.1% in the center span). Deflections produced by the test truck were therefore increased 15% to approximate the deflections produced by the design truck.

Shortly before the field tests began, Mr. Derwin Merrill, a summer employee of the Materials Department, joined Mr. Moothart and Mr. Sharpnack. Mr. Merrill was in charge of the oscillograph during the tests, and afterward measured the deflections and calculated and recorded the data. Mr. Merrill is now a member of the faculty of the Engineering Mechanics division of Iowa State University, Ames, Iowa.

In order to guide the truck in the test runs, lines of tape were placed longitudinally on the bridge floor to locate the center line of the truck at 4.5', 7.5' and 10.5' from each gutter (see figure 3) and a boom was attached to the front of the truck with a tell tale hanging down located so that it was visible to the driver. By keeping this tell tale above the appropriate line of tape, the driver was able to follow the proper path.

To check the actual location of the truck, four lines of tape were placed on the bridge floor normal to the center line of roadway, as shown in figure 3. Immediately before each run, shaving lather was extruded from a pressure can on each of these transverse tapes. The truck left clear tracks in the shaving lather, and measurement from the gutter line to these tracks located the trucks path accurately.

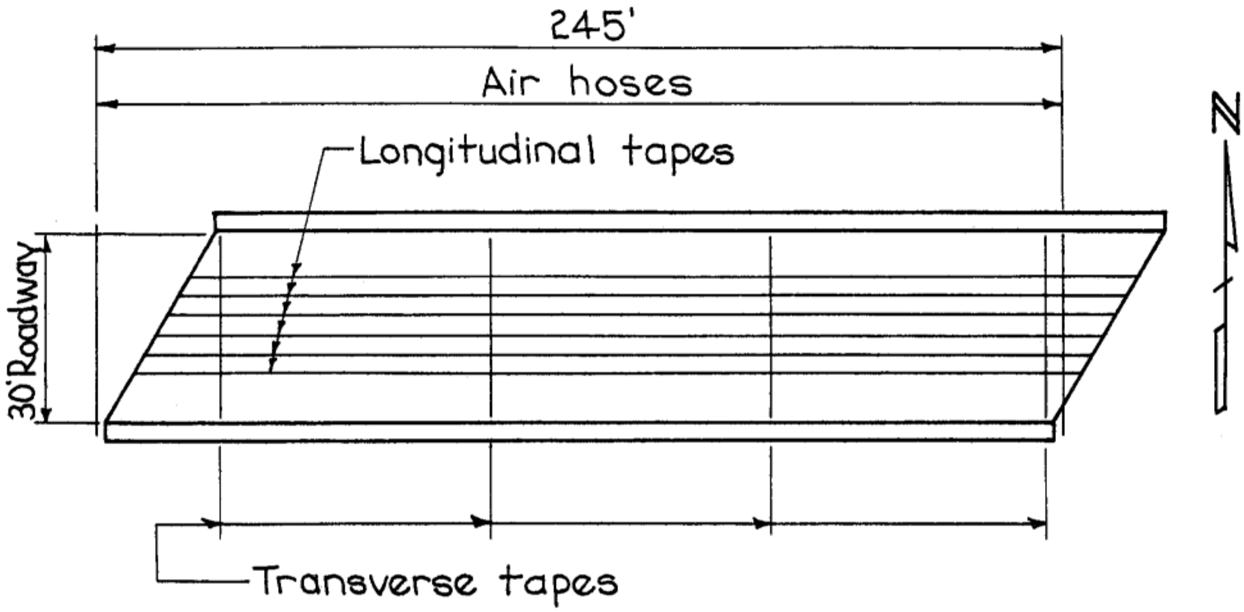



Figure 3

To determine the speed of the truck, air hoses, actuating pressure switches, were placed near the ends of the bridge, 245' apart, as shown in figure 3. When the truck passed over these tapes, the pressure switches produced blips on the oscillograph tapes. The scaled distance, in inches, between these blips, divided by the known speed of the tape in inches per second gives the time in seconds of the truck's passage. 245', divided by the time, gives the truck's speed in feet per second, which is translated into miles per hour.

For test 1, SR-4 gages were placed on the two north beams, 29' from the center line of abutment bearings in the end spans, and midway between pier bearings in the center span. Figure 4 is a structural layout of the beams, showing the location of these gages.

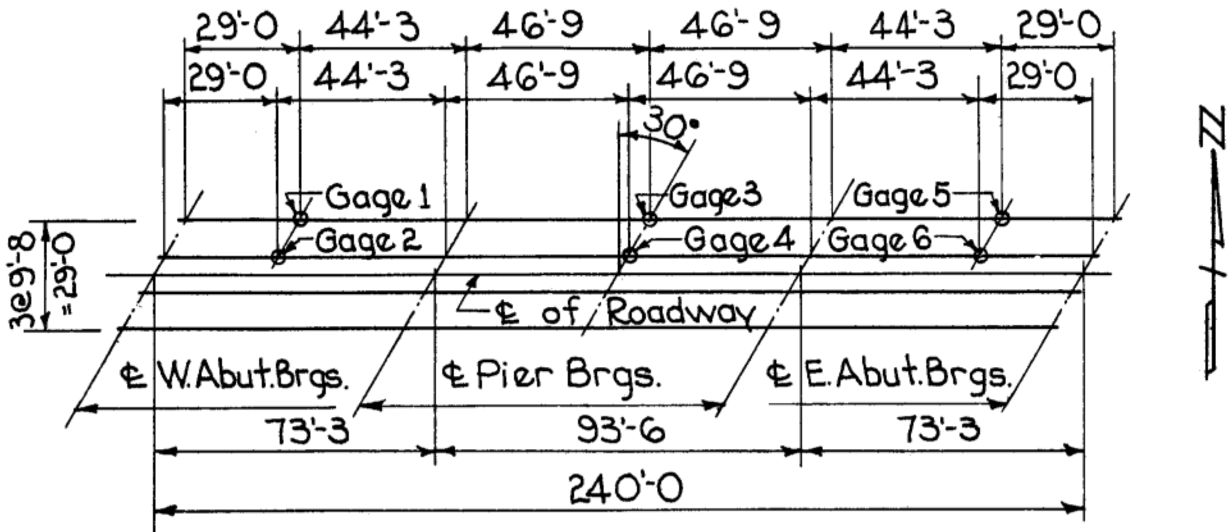



Figure 4

Test 2. was made to check the lateral symmetry of the bridge. For this test, gages 5 and 6 were placed on the south interior and exterior beams, respectively, midway between pier bearings in the center span. Figure 5 shows a structural layout of the beams in the center span, locating these gages.

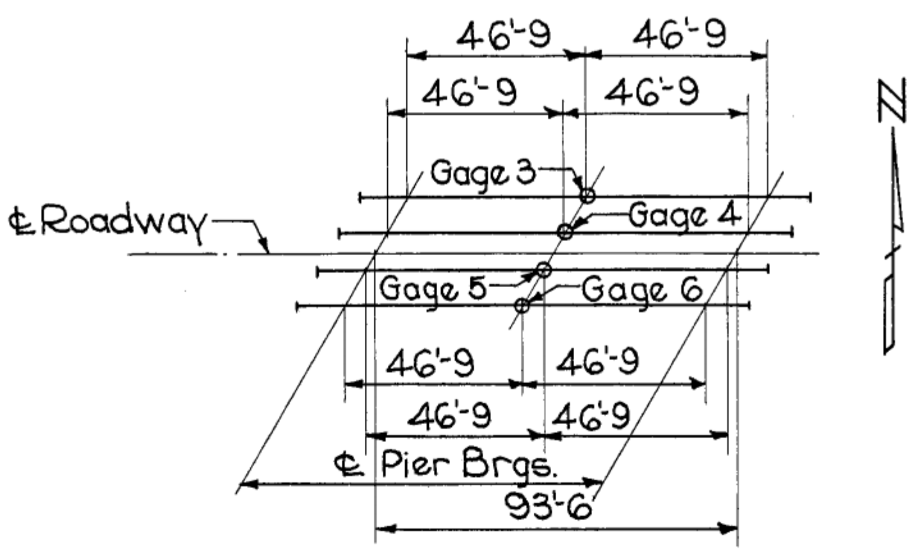



Figure 5

The tests were run on the 18th, 19th, and 20th of June, 1963. A total of 49 runs were made for Test 1, and 25 for Test 2. 17 runs were eliminated from Test 1, and 1 from Test 2, because of misplacement of truck, wrong speed, or mechanical failure. The deflections, recorded on oscillograph tape, were permanized by chemical treatment, and are retained in the files of the Iowa State Highway Commission as a permanent record.

Tables 1 and 2 show the results of Tests 1 and 2 respectively. For each run, the maximum deflection for each gage, including vibration and impact, are recorded to the nearest even hundredth of an inch. Table 2 indicates reasonably symmetrical behavior.

TABLE 1

TEST 1 - RUNS 1-32 INCLUSIVE

| Run No. | Location of Truck & Direction | Speed mph | Deflection in Inches at Gage No. 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|---|
| 1 | 4.5' from | 3.03 | 0.35 | 0.27 | 0.68 | 0.47 | 0.42 | 0.33 |
| 3 | North gutter | 2.78 | 0.38 | 0.28 | 0.68 | 0.47 | 0.38 | 0.31 |
| 5 | East to West | 22.4 | 0.41 | 0.27 | 0.67 | 0.46 | 0.44 | 0.33 |
| 7 | | 20.4 | 0.40 | 0.28 | 0.67 | 0.46 | 0.45 | 0.32 |
| 9 | 7.5' from | 2.60 | 0.32 | 0.28 | 0.53 | 0.43 | 0.32 | 0.31 |
| 11 | North gutter | 3.06 | 0.30 | 0.28 | 0.57 | 0.47 | 0.35 | 0.32 |
| 13 | East to West | 35.0 | 0.39 | 0.35 | 0.58 | 0.48 | 0.35 | 0.32 |
| 15 | | 36.7 | 0.39 | 0.36 | 0.59 | 0.49 | 0.33 | 0.34 |
| 17 | 10.5' from | 2.79 | 0.23 | 0.29 | 0.42 | 0.42 | 0.26 | 0.30 |
| 19 | North gutter | 5.71 | 0.25 | 0.30 | 0.46 | 0.47 | 0.26 | 0.32 |
| 21 | East to West | 20.1 | 0.23 | 0.30 | 0.44 | 0.46 | 0.26 | 0.32 |
| 23 | | 20.6 | 0.23 | 0.26 | 0.43 | 0.44 | 0.26 | 0.31 |
| 25 | | 41.6 | 0.29 | 0.31 | 0.48 | 0.47 | 0.29 | 0.37 |
| 27 | | 41.5 | 0.25 | 0.32 | 0.49 | 0.47 | 0.26 | 0.31 |
| 29 | | 53.9 | 0.24 | 0.29 | 0.49 | 0.51 | 0.28 | 0.33 |
| 31 | | 54.7 | 0.25 | 0.30 | 0.50 | 0.50 | 0.26 | 0.32 |
| 32 | 10.5' from | 43.4 | 0.09 | 0.20 | 0.20 | 0.36 | 0.09 | 0.21 |
| 30 | South gutter | 43.7 | 0.09 | 0.20 | 0.21 | 0.35 | 0.12 | 0.21 |
| 28 | West to East | 39.9 | 0.08 | 0.20 | 0.24 | 0.37 | 0.11 | 0.23 |
| 26 | | 41.0 | 0.09 | 0.19 | 0.22 | 0.35 | 0.10 | 0.22 |
| 24 | | 21.7 | 0.09 | 0.20 | 0.19 | 0.32 | 0.09 | 0.21 |
| 22 | | 20.2 | 0.08 | 0.20 | 0.18 | 0.31 | 0.09 | 0.21 |
| 20 | | 3.90 | 0.08 | 0.19 | 0.17 | 0.32 | 0.09 | 0.20 |
| 18 | | 2.65 | 0.09 | 0.20 | 0.17 | 0.30 | 0.08 | 0.18 |
| 16 | 7.5' from | 35.5 | 0.07 | 0.18 | 0.18 | 0.32 | 0.10 | 0.20 |
| 14 | South gutter | 35.9 | 0.07 | 0.16 | 0.17 | 0.31 | 0.10 | 0.19 |
| 12 | West to East | 2.70 | 0.04 | 0.16 | 0.08 | 0.25 | 0.06 | 0.14 |
| 10 | | 2.61 | 0.05 | 0.16 | 0.08 | 0.24 | 0.06 | 0.14 |
| 8 | 4.5' from | 20.8 | 0.02 | 0.12 | 0.04 | 0.22 | 0.02 | 0.11 |
| 6 | South gutter | 19.9 | 0.02 | 0.12 | 0.05 | 0.22 | 0.03 | 0.12 |
| 4 | West to East | 2.89 | 0.02 | 0.12 | 0.03 | 0.20 | 0.02 | 0.11 |
| 2 | | 2.81 | 0.01 | 0.12 | 0.02 | 0.18 | 0.02 | 0.12 |

TABLE 2

TEST 2 - RUNS 33-56 INCLUSIVE

| Run No. | Location of Truck & Direction | Speed mph | Deflection in Inches at Gage No. 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| 49 | 4.5' from | 2.96 | 0.66 | 0.43 | 0.19 | 0.01 |
| 51 | North gutter | 3.21 | 0.67 | 0.44 | 0.19 | 0.02 |
| 53 | East to West | 38.4 | 0.72 | 0.49 | 0.24 | 0.10 |
| 55 | | 40.7 | 0.71 | 0.48 | 0.23 | 0.07 |
| 41 | 7.5' from | 2.70 | 0.61 | 0.44 | 0.25 | 0.12 |
| 43 | North gutter | 2.72 | 0.55 | 0.45 | 0.23 | 0.06 |
| 45 | East to West | 39.6 | 0.57 | 0.47 | 0.25 | 0.09 |
| 47 | | 38.0 | 0.57 | 0.46 | 0.29 | 0.09 |
| 33 | 10.5' from | 2.79 | 0.44 | 0.43 | 0.32 | 0.18 |
| 35 | North gutter | 2.68 | 0.44 | 0.43 | 0.32 | 0.20 |
| 37 | East to West | 35.7 | 0.49 | 0.49 | 0.38 | 0.24 |
| 39 | | 35.2 | 0.50 | 0.50 | 0.37 | 0.24 |
| 40 | 10.5' from | 35.3 | 0.21 | 0.35 | 0.47 | 0.45 |
| 38 | South gutter | 35.0 | 0.21 | 0.34 | 0.49 | 0.45 |
| 36 | West to East | 2.87 | 0.17 | 0.31 | 0.44 | 0.42 |
| 34 | | 2.75 | 0.18 | 0.30 | 0.42 | 0.44 |
| 48 | 7.5' from | 38.9 | 0.11 | 0.25 | 0.47 | 0.59 |
| 46 | South gutter | 39.6 | 0.13 | 0.27 | 0.48 | 0.59 |
| 44 | West to East | 2.90 | 0.09 | 0.23 | 0.42 | 0.52 |
| 42 | | 2.66 | 0.09 | 0.24 | 0.44 | 0.45 |
| 56 | 4.5' from | 40.6 | 0.08 | 0.23 | 0.48 | 0.68 |
| 54 | South gutter | 40.6 | 0.09 | 0.24 | 0.48 | 0.67 |
| 52 | West to East | 3.27 | 0.03 | 0.19 | 0.43 | 0.64 |
| 50 | | 2.96 | 0.03 | 0.20 | 0.44 | 0.62 |

The following procedure was used to calculate the maximum deflection under the design load with two lanes loaded:

1. For each gage, the absolute maximum deflection for any run of test 1, or test 2, if applicable, was chosen.

2. To this was added the maximum deflection, at that gage, caused by any run on the other half of the bridge.

3. The sum of these deflections was multiplied by 1.15 to approximate the deflection caused by the design load.

4. This deflection was expressed as a fraction of the span, for comparison with the AASHO allowable deflection.

The calculations follow:

Gage 1

| | | |
|---|---|---|
| Run 5 | 0.41" | 4.5' from north gutter |
| Run 32 | 0.09" | 10.5' from south gutter |
| | 0.50" | |

For design load H20-S16-44

(0.50")(1.15) = 0.58" $\qquad \dfrac{0.58}{(73.25)(12)} = \dfrac{1}{1520}$ approx.

Gage 2

| | | |
|---|---|---|
| Run 15 | 0.36" | 7.5' from north gutter |
| Run 32 | 0.20" | 10.5' from south gutter |
| | 0.56" | |

For design load H20-S16-44

(0.56")(1.15) = 0.64" $\qquad \dfrac{0.64}{(73.25)(12)} = \dfrac{1}{1370}$ approx.

Gage 3

| | | |
|---|---|---|
| Run 53 | 0.72" | 4.5' from north gutter |
| Run 28 | 0.24" | 10.5' from south gutter |
| | 0.96" | |

For design load H20-S16-44

(0.96")(1.15) = 1.10" $\qquad \dfrac{1.10}{(93.5)(12)} = \dfrac{1}{1020}$ approx.

Gage 4

| | | |
|---|---|---|
| Run 29 | 0.51" | 10.5' from north gutter |
| Run 28 | 0.37" | 10.5' from south gutter |
| | 0.88" | |

For design load H20-S16-44

$(0.88")(1.15) = 1.01"$ $\qquad \dfrac{1.01}{(93.5)(12)} = \dfrac{1}{1110}$ approx.

Gage 5 (Test 1)

| | | |
|---|---|---|
| Run 7 | 0.45" | 4.5' from north gutter |
| Run 30 | 0.12" | 10.5' from south gutter |
| | 0.57" | |

For design load H20-S16-44

$(0.57")(1.15) = 0.66"$ $\qquad \dfrac{0.66}{(73.25)(12)} = \dfrac{1}{1330}$ approx.

Gage 6 (Test 1)

| | | |
|---|---|---|
| Run 25 | 0.37" | 10.5' from north gutter |
| Run 28 | 0.23" | 10.5' from south gutter |
| | 0.60" | |

For design load H20-S16-44

$(0.60")(1.15) = 0.69"$ $\qquad \dfrac{0.69}{(73.25)(12)} = \dfrac{1}{1270}$ approx.

All deflections are well within the allowable limits of the AASHO specifications. The deflections show some inconsistencies, as follows:

1. Deflections in the east end span (gages 5 and 6) are greater than those in the west end span (gages 1 and 2).

2. In the end spans, the deflections of the interior beam is greater than that of the exterior beam; in the center span the deflection of the exterior beam is greater.

These inconsistencies could probably be explained, but we believe the data are insufficient to permit a definite conclusion.

We believe, however, that the deflections obtained approximate quite closely the maximum deflections under the design load, and that prestressed steel beam bridges perform satisfactorily in service.

Although the Silver Creek bridge showed a saving of $2500 over a welded girder bridge of the same dimensions and skew angle, we believe that the economic advantage of prestressed steel construction would be greater for a shorter span, where the competing design would be fabricated from rolled beams. It is possible that a continuous prestressed steel bridge would be competitive in price with prestressed concrete for grade separations on or over an Interstate Highway. We recommend, therefore, that other prestressed steel bridges be designed and built in order to test the economical span, and to establish their cost under normal conditions.

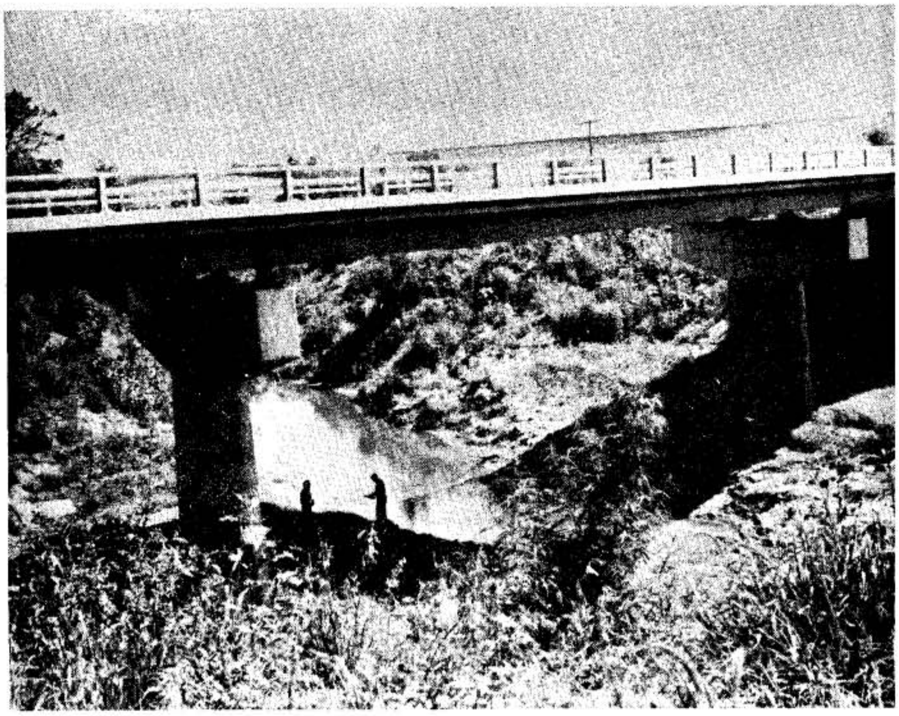


Photo 1: General view of the Silver Creek bridge

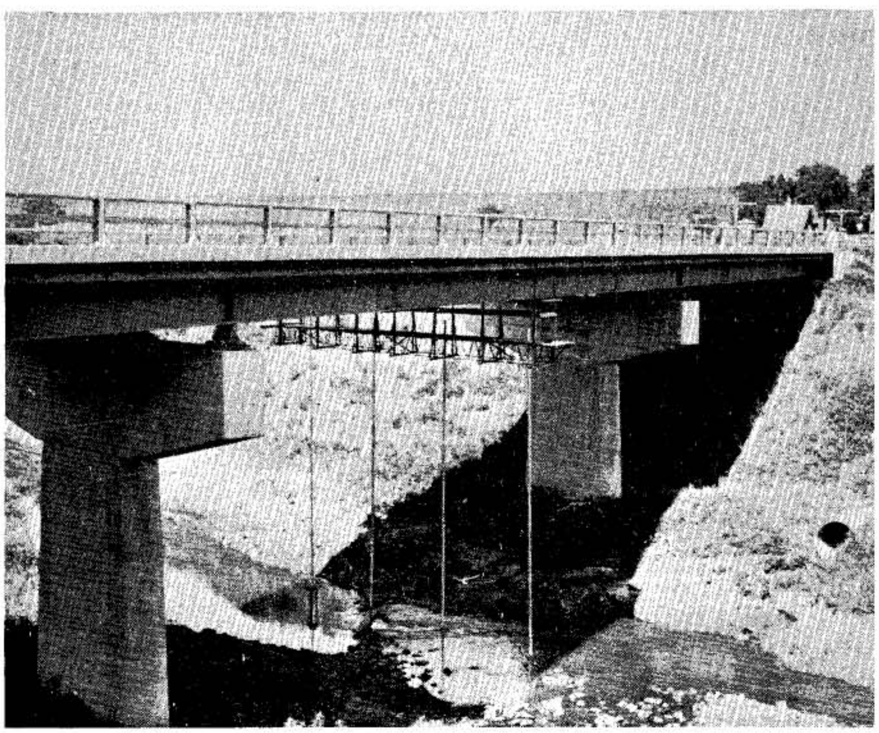


Photo 2: Bridge, showing falsework erected for installing deflection gages.

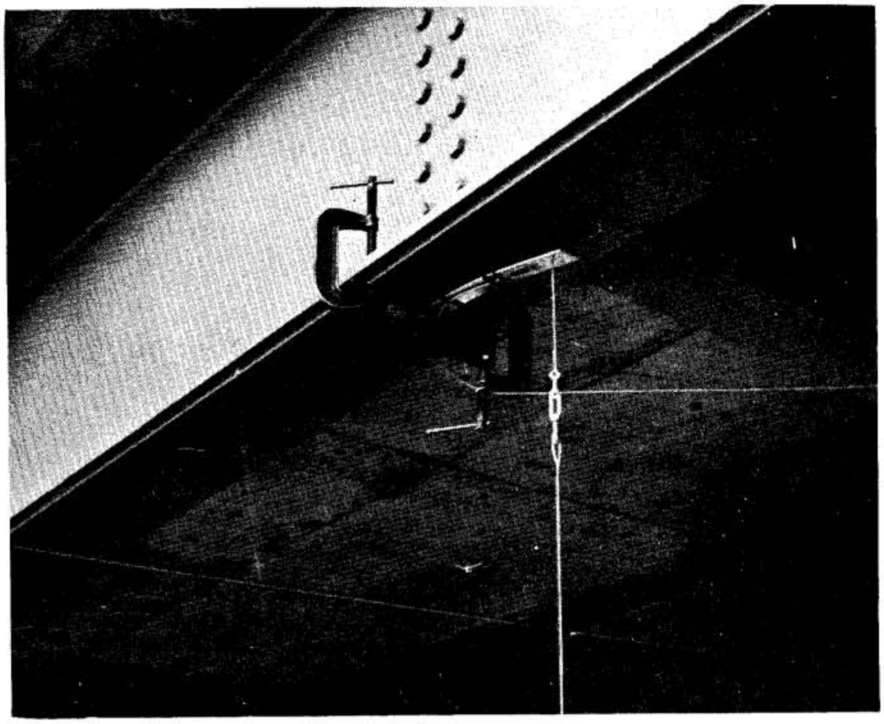


Photo 3: Cantilever type deflection gage attached to bridge beam.

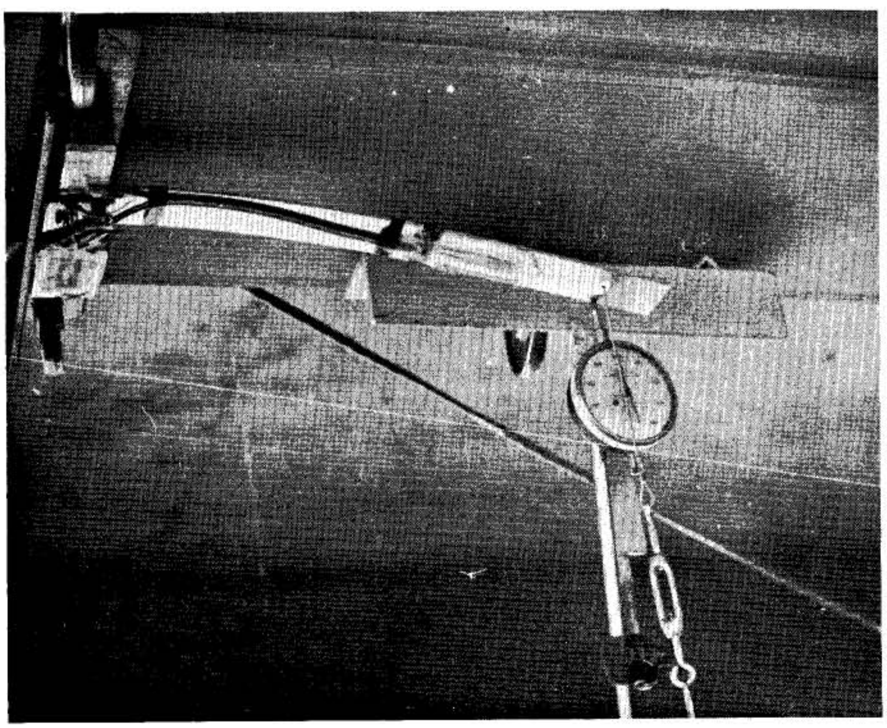


Photo 4: Mechanical calibration of deflection gage.

Photo 5: Test truck, showing tell tale guide for driver.

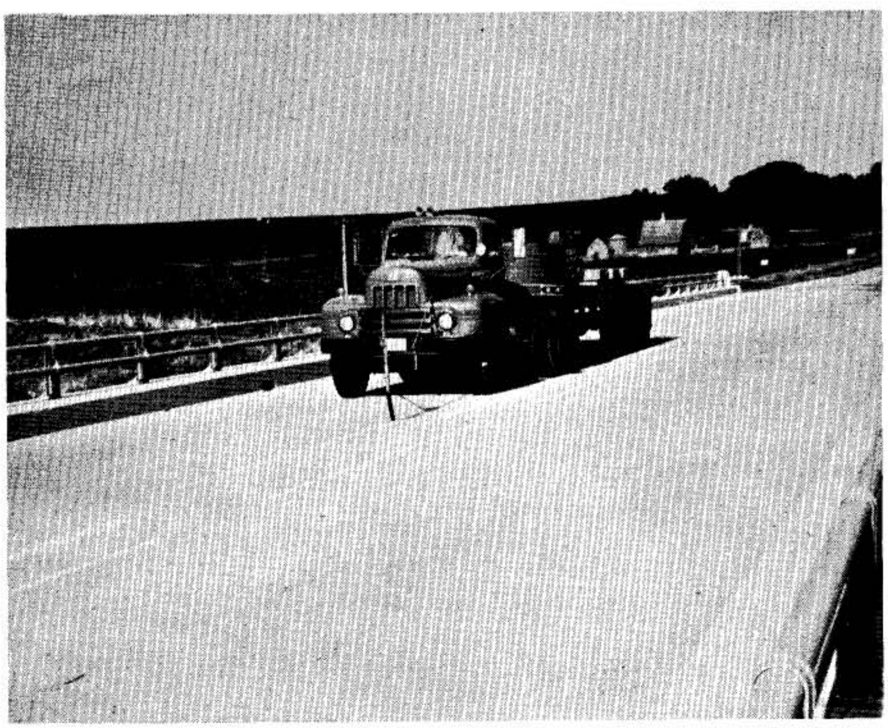


Photo 6: Truck on bridge during test run.