Final Report

HR-73

THE FLEXURAL FATIGUE STRENGTH
OF PRESTRESSED STEEL I-BEAMS

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ABSTRACT

The purpose of this investigation was to study the flexural fatigue strength of two prestressed steel I-beams which had previously been fabricated in connection with a jointly sponsored project under the auspices of the Iowa State Highway Commission.

The beams were prestressed by deflecting them under the action of a concentrated load at the center of a simple span, then welding unstressed high strength steel plates to the top and bottom flanges to retain a predetermined amount of prestress. The beams were rolled sections of A36 steel and the plates were USS "T-1" steel.

Each of the two test specimens were subjected to an identical repeated loading until a fatigue failure occurred. The loading was designed to produce stresses equivalent to those which would have occurred in a simulated bridge and amounted to 84 percent of a standard H-15 live load including impact. One of the beams sustained 2,469,100 repetitions of load to failure and the other sustained 2,756,100 cycles.

Following the fatigue tests, an experimental study was made to determine the state of stress that had been retained in the prestressed steel beams. This information, upon which the calculated stresses of the test could be superimposed, provided a method of correlating the fatigue strength of the beams with the fatigue information available on the two steels involved.

INTRODUCTION

Engineers have been proposing various prestressing techniques to utilize more fully the new high strength steels. The Iowa State Highway Commission has recently developed a method of prestressing rolled wide-flange beams, thus possibly increasing economy for short span continuous highway bridges[15]. Unstressed high strength steel cover plates are welded to the beam while it is held in a deflected position. The cover plates thus "lock in" a predetermined amount of prestress.

A project, HR-73, sponsored jointly by steel producers, steel fabricators, the Iowa Highway Research Board, and the Iowa State Highway Commission, was established to study the problems of fabrication, to substantiate the theoretical analysis and to determine the load-deflection characteristics of several prestressed steel beams.

Five beams were fabricated. Three of the beams were static test loaded in the elastic stress range under the direction of the Research Engineer for the Iowa State Highway Commission. A report[3] of this work was presented at the annual Structural Engineering Conference of the American Institute of Steel Construction at Minneapolis, Minnesota, on May 12, 1961.

When the above testing was completed, the Department of Civil Engineering at Iowa State University was given the beams for investigation of their fatigue strength.

Only the two prestressed steel beams which simulated the negative moment sections (cover plates on top and bottom flanges) were used for the study. Since little research has been done on the fatigue of such members, this investigation was conducted with the following objectives:

(1) To predict a repeated loading for the prestressed steel beams that would be expected to cause fatigue failure near two million cycles.

(2) To investigate the flexural fatigue strength of the beams in the state of stress determined experimentally to be present.

REVIEW OF LITERATURE

The effect of repeated stress on engineering materials has been of concern to engineers for more than a century. Repeated stress tests, were conducted as early as 1829, and the first comprehensive series were conducted in the 1860's. That series of tests resulted in the following conclusions[13, pp. 9-15].

(1) Wrought iron and steel will rupture at a unit stress not only less than the ultimate static strength of the material, but even less than the elastic limit, if the stress is repeated a sufficient number of times.

(2) Within certain limits the range of stress rather than the maximum stress determines the number of cycles before rupture.

(3) For a given minimum or maximum unit stress an increase of range of stress decreases the cycles necessary for rupture.

(4) For a given minimum or maximum unit stress there appears to be a limiting range of stress which may be applied indefinitely without producing rupture.

(5) As the maximum applied unit stress increases, this limiting range of stress decreases.

The first conclusion explains the phenomenon commonly referred to as <u>fatigue failure</u>; the term usually applied to the fourth conclusion is <u>fatigue strength</u>. Recent investigators are conducting studies substantiating and utilizing these same general conclusions[4, pp. 95-96]:

> For the same maximum stress, the number of stress cycles to produce failure increases as the stress ratio (range of stress) is increased. This behavior is typical for most steels and demonstrates the fact that fatigue failure is determined by three factors: (a) the maximum stress in the stress cycle, (b) the stress ratio or range of stress in the stress cycle, and (c) the number of applications of the stress cycle.

During the years that fatigue failures have been observed, many theories have been developed attempting to explain the cause of the failure. Three of these are presented as the theory of crystallization, the slip band theory, and the progressive failure theory[12, pp. 9-10]. The theory of failure by crystallization was based on the idea that under repeated stress, metal changed from a "fibrous" material to one having a crystalline structure, thus becoming brittle and subject to failure along planes between the crystals. This theory has been thoroughly discredited. The slip band theory stated that under severe stress the crystalline grains, of which it has been established all solid metals are made, are split into thin plates which slide on each other, the slip being visible at the surface of the crystals in the form of fine lines across the surfaces. It was assumed that under the action of repeated stress some slip bands develop into cracks. Further it was assumed that final failure of a piece of metal was caused by the spread of one or more of these cracks. The progressive failure theory was presented as being an extension of the slip band theory.

> The theory here presented is based on the idea that the failure of metals under repeated stress is characterized by the spread of minute cracks which may be the result of intra-crystalline slips, but which are minute fractures (as distinguished from slips) which may occur with or without slip at locations where stress is so intensified that the ultimate limit of cohesion is reached between adjacent microscopic parts of the metal.

Probably the two most accepted theories today are the slip band and the progressive failure theories.

Three general types of loading induce fatigue failures: direct axial loading, flexural loading and torsional loading. It has been observed that the endurance limit of a metal established by axial loading is not the same as when established by flexural loading. It was reported[12] that the average value for the ratio of the endurance limit under reversed axial stress to the endurance limit under reversed flexural stress was 0.64[12, pp. 35-37]:

> If there were a definite absolutely fixed tensile endurance limit for a metal, and if that limit could be determined exactly, it would seem reasonable to assume the endurance limit for reversed axial stress to be the same as the endurance limit for reversed flexural stress. This, however, does not seem to be the case, at least for wrought ferrous metals, and the endurance limit for reversed axial stress is found to be lower than for reversed flexural stress. In each case, axial stress and flexural stress, the endurance limit seems to lie within a narrow range of values of unit-stress inside of which the strengthening influences and the destructive influences of repeated stress are very nearly balanced. Below this range the metal does not fail even after an indefinitely large number of cycles of stress; and above this range the metal fails after a comparatively small number of cycles of stress. It does not seem unreasonable to assume that if the metal is subjected to reversed axial stress, owing to the more nearly uniform distribution of stress over the entire cross-section, the probability of fatigue failure developing from any local weakness is greater than for metal subjected to reversed flexural stress with maximum stress confined to the outer fibers of the cross-section.

In one of the most complete tests on flexural fatigue of steel beams the purpose was to determine the relative fatigue strengths of various types of beams[16]. The results of these tests that are of particular interest to this investigation are:

(1) The carrying capacity of various types of beams does not increase directly with an increase in the section modulus of the beam.

(2) The fatigue strength of the compression flanges of beams is about the same as that of the tension flange.

(3) The fatigue strength at two million cycles is materially reduced with intermittent fillet welding on the cover plates as compared to specimens fabricated with continuous welded cover plates.

(4) The carrying capacity of fabricated beams is appreciably lower than the capacity of a rolled beam.

(5) Stiffeners welded in the tension area of the beam can reduce the fatigue strength considerably.

(6) The fatigue strength of a rolled beam is appreciably higher than one with cover plates fillet welded. This is true even for continuous fillet welding.

A study was conducted to obtain quantitative data on the fatigue strength and to evaluate the relative merits of the various end configurations on flexural members[5]. Fabricated beams of A-373-54T steel were used as the flexural members in the tests. Cover plates containing various end configurations were welded to the beams. The specimens were then subjected to repeated loading until fatigue failure occurred. It can be concluded from their investigations that gradual transfer of stress at points of changing cross-section are essential to long fatigue life. Abrupt changes will greatly reduce the fatigue strength of a member.

In a study concerning stiffened beams, it was again pointed out that welding to the tension flange does enhance a flange failure for which a sudden failure could result[10].

A search failed to disclose any information pertaining to the fatigue of fillet welds connecting the recently developed high strength steels. The lack of such information is further substantiated as follows[4, p. 99]:

> Another type of welded joint which must be considered is the fillet-weld subject to fluctuating stresses. Although considerable information obtained from tests is available on the fatigue strength of this type of joint in structural carbon steel plates, the authors are not aware of similar information on the fatigue strength of fillet welds on T-1 steel plates.

Several studies of fatigue of welded specimens were reviewed. These studies involved welded structural carbon steel plates but many of the more general observations that can be made of these studies are probably valid or at least indicative of what should be expected from welded high strength steels.

Fatigue tests were conducted on welded joints in structural steel plates[17]. The type of weld studied was the butt weld. A summary of the findings indicated that the lowest fatigue strength was observed in the plates tested in the as-welded condition. The fatigue strength was improved by removing the weld reinforcement. Machining it off was more effective than removing it by grinding. Stress-relieving the weld had little, if any, effect on prolonging the life of the specimen. It could be concluded that any slight undercutting at the edge of the reinforcement, internal flaws in the weld or indentation in the surface of the plate would cause a concentration of stress, thus enhancing the possibility of a fatigue fracture at that particular location.

A series of tests was conducted to compare the fatigue and static properties of butt welds produced with E6010 high-cellulose and E7016 low-hydrogen electrodes. Both longitudinal and transverse butt welds were tested. The summary states[6, p. 83s]:

> For the specimens tested with the stress perpendicular to the direction of welding, there was no significant difference in the fatigue strength of welds produced with the two types of electrode and tested with the reinforcement on or off. For the specimens tested with the stress parallel to the direction of welding, the scatter is large, but the E7016 welds with the reinforcement either on or off are generally more resistant to repeated loadings than the E6010 welds, especially if failure occurs after a relatively low number of highly stressed cycles.

About half of the fractures were reported to have originated where the electrode had been changed. It was also observed that welds deposited in the flat position had a significantly greater fatigue strength than those deposited in the horizontal or vertical position.

In discussing the fabrication and fatigue stresses as they pertain to high strength steels[4, p. 90]:

> In considering all fabrication operations on steels of this high strength level, the importance of good and careful workmanship cannot be emphasized too strongly. . . . The ability of steel to withstand such (repeated) stresses is greatly impaired by the presence of surface notches and irregularities. Typical of such stress raisers are rough bolt holes, under-cut or irregular weld beads, and misalignments which will create bending stresses where none were intended.

A study was conducted on the determination of residual cooling stresses in rolled sections[8]. Results indicated that residual cooling stresses were of considerable magnitude. The outside edges of the flanges were found to be in compression as high as 17,500 psi; the center of the flange near the web showed stresses as high as 11,000 psi tension. The web was in residual tension with a maximum stress of 16,000 psi. Further investigation to determine the variation of the residual cooling stresses along the length of a beam showed that they were very nearly constant to within about one foot of each end of the beam. It was noted that the magnitude and distribution of residual stresses present in a long beam were preserved at the center part of a piece cut out of the beam, if the length of the piece was about three times the depth of the section.

In a qualitative study of residual stresses in welds, the following comment is made[11, p. 374s]:

> It is known that the physical properties of certain steels are influenced by their rate of cooling from the molten state. For a high cooling

rate, the yield point, hardness and ultimate strength increase while the ductility is reduced. In welding, the metal is subjected to heating and subsequent cooling. Thermal stresses set up by the large temperature differences near a weld may produce large plastic deformations and result in residual stresses. The residual stresses together with any change in properties of the material due to rapid cooling represent a possible hazardous condition.

The effect of internal stress on the fatigue strength of a metal, is stated as follows[13, p. 204]:

> The presence of internal stress in a metal will be such as to increase the resultant maximum stress above the computed stress, when the applied stress is of the same kind as the internal stress. There will be an apparent reduction of the endurance limit as computed on the basis of the applied load.

With respect to relating laboratory fatigue tests and field fatigue tests, the following was reported on the AASHO Road Test[7, p. 323] about the fatigue of their test bridges:

> Fatigue distress observed in the beams with partial length cover plates of the steel bridges and in the reinforcing bars of the reinforced concrete bridges occurred in the same manner as that observed in laboratory tests. The number of cycles to fatigue distress in these bridge members agreed reasonably well with the number of cycles computed from equations developed from simple laboratory tests of beams. Thus, where a reasonable estimate of the magnitude and number of repetitions of stress can be made, laboratory fatigue data for the material can forecast the life to fatigue failure within reasonable limits.
>
> The steel bridge without cover plates subjected to 392,400 trips of test vehicles withstood the repeated stressing without fatigue cracking. The mean stress ranges at midspan were 19,100 to 21,000 psi and the minimum stress was 12,700 psi. According to laboratory data these beams could have resisted approximately 2,000,000 repetitions of 28,000 psi stress range superimposed on the actual minimum stress. Thus, the laboratory data indicated that a fatigue failure under the conditions existing in the test bridge was improbable.

The literature search failed to uncover any previous work on the flexural fatigue testing of welded prestressed steel members. It also failed to provide much related information on the fatigue characteristics of the newer high strength steels. It did serve to provide a knowledge of fatigue terminology, testing procedures, testing equipment, and the factors that influence the fatigue strength of a metal. All of these are important factors in undertaking an investigation of this type.

FABRICATION AND STATIC TESTS

The detailed information concerning materials, fabrication, and static load tests was available in the form of a report[3] which was supplemented, for the following discussion, by means of direct contacts with personnel of the Iowa State Highway Commission.

MATERIALS

The rolled section, from which the two beams were fabricated, was an 18 inch wide-flange, weighing 50 lbs per foot. Both beams were 26 feet long. One was furnished by the Inland Steel Company and the other by the Bethlehem Steel Company. The rolled sections conformed to the minimum requirements of ASTM-A36-60T[2, pp. 258-260].

The cover plates which were furnished by the United States Steel Corporation, were high strength steel referred to as USS "T-1." The top and bottom cover plates had the nominal cross-section dimensions of 6 x 3/4 inches and 9 x 1/2 inches, respectively. The over-all length of each plate was 24 feet.

The details of the beams, as they were when received for this investigation, are shown in figure 1. Physical properties of the materials are given in table I.

Table I. Properties of materials.*

Property	Top flange	Web	Bottom flange	$9'' \times \frac{1}{2}''$ plate	$6'' \times \frac{3}{4}''$ plate
Material (type of steel)	A36	A36	A36	T-1	T-1
Modulus of elasticity times $(10)^6$ (psi)	29.4	28.6	28.0	27.2	30.4
Yield point (ksi)	35.2	40.6	36.0		
Yield strength for 0.2% offset (ksi)				124.5	116.1
Tensile strength (ksi)	65.3	70.6	64.9	132.1	126.0
Elongation in 8 inches (percent)	28.5	27.9	30.5		
Elongation in 2 inches (percent)				17.5	18.0

*Source: Iowa State Highway Commission[9].

FABRICATION OF TEST BEAMS

The first beam was fabricated in Des Moines, Iowa, at the Pittsburgh-Des Moines Steel Company's plant. It is referred to as the Des Moines Beam. The beam was supported as shown with a 30,000 lb jack load applied at the center of the 25 foot simple span (figures 2, 3). The top and bottom T-1 plates had previously been positioned in an unstressed condition and were held by clamps. While the jack force was still present, the cover plates were welded to the flanges of the beam with continuous

Table II. Welding data for beams.*

Item	Des Moines Beam (automatic)	Ames Beam (semi-automatic)
Wire	L-60 (5/32)	L-60 (3/32)
Flux	760	760
Voltage	34	32
Amperage	450	525
Speed (in./min)	25	25
Fillet weld size (in.)	5/16	5/16
Welding time (hrs)	1 3/4	1 1/4

*Source: Iowa State Highway Commission[9].

fillet welds (table II). This was accomplished by automatic welding in the following sequence:

(1) The bottom (compression) flange and the 9 x $\frac{1}{2}$ inch T-1 plate were welded along side 2.†

(2) Side 1 of the bottom flange and plate were welded.

(3) The top (tension) flange and the 6 x $\frac{3}{4}$ inch T-1 plate were welded along side 1.

(4) Side 2 of the top flange and plate were welded.

The jack force was removed after the welding was completed. The theoretical stresses during fabrication at the center of the span were determined for both beams (table III).

The second beam was fabricated in the Iowa State Highway Commission Laboratory at Ames, Iowa. This beam is referred to as the Ames beam. The prestressing arrangement was as for the Des Moines beam except a 39,000 lb load was used to induce the initial stresses. The cover plates were then tack welded to the flanges sufficiently to transmit the prestressing shear. This amounted to $\frac{1}{8}$ x 4 inch welds placed at 18 inch

†The sides of the beam were arbitrarily numbered 1 and 2 for purposes of reference to the fatigue failures and state of stress study.

Table III. Theoretical stresses during fabrication

Stresses	Top plate	Top flange	Bottom flange	Bottom plate
	Des Moines beam			
Initial stress[*] (ksi)		+25.3[†]	-25.3[‡]	
Prestress[&]	-13.8	+12.5	-12.5	+13.6
	Ames beam			
Initial stress (ksi)		+32.9	-32.9	
Prestress (ksi)	-18.0	+16.2	-16.2	+17.6

[*] After jacking.

[†] Plus denotes tension.

[‡] Minus denotes compression.

[&] After removing jack.

centers. The tack welding proceeded from the center of the beam out to the supports. The jack force was removed after the tacking operation was completed. Two welders, using a semi-automatic process, completed the welding by simultaneously welding both sides of the plate to the top (tension) flange and then to the bottom (compression) flange. The results were continuous fillet welds the full length of the plates.

Both beams were instrumented, at predetermined locations, to record strain and deflections during fabrication. A Whittemore mechanical strain gage was used as the primary means of recording the strains with SR-4 electrical resistance gages providing supplementary information. In addition to strains and deflections, temperature readings on the flanges and cover plates were recorded while welding the Ames beam. The temperatures were measured by means of temperature crayons varying from 100°F to 1000°F (figure 4)[9].

STATIC LOAD TESTS

A series of static load tests were conducted on the Ames beam by the Iowa State Highway Commission. The purpose of these tests was to determine its load-deflection characteristics. Strain measurements and deflection readings were taken as the beam was loaded and unloaded several times. The Des Moines beam was not static tested, since it was unlikely that its fabrication procedure would be adopted. Both beams were subjected to the same loading, however, since the Des Moines beam was used as a "strong back" against which to apply the jack force.

In the static tests two ranges of load were applied to the beam. The load was applied as a single concentration at the center of a 25 foot simple span. The imposed stresses due to a maximum load of 70,000 lbs were 31,600 psi and 30,000 psi tension in the bottom plate and bottom flange, respectively. The corresponding computed net stresses were 49,200 psi tension in the bottom plate and 13,800 psi tension in the bottom flange. The loading tests resulted in a permanent set or loss of camber of 0.06 inch at the centerline of the span. No damage to the beams was apparent as a result of the static tests.

TESTING PROCEDURE

The order of events amounted to a design of a repeated loading which would simulate an actual highway bridge loading, the fatigue tests, and finally the experimental determination of existing stresses in the beams.

DESIGN OF REPEATED LOADING

Since the beams were fabricated to represent bridge members, they were stressed much the same as if they were in an actual bridge. This meant that a minimum load was maintained to simulate the action of the superstructure dead load. The range of repeated loading then would be similar to maximum service loads on the bridge. Other factors to be considered in designing the tests were the desirability of a fatigue failure occurring at about two million cycles, and the load-deflection capabilities of the testing equipment. The significance of two million cycles of design loading in terms of the useful life of a highway bridge is highly speculative and requires considerable future study.

The test beams were adequate to sustain an H-15-44 live load* [1] on a 150 x 24 foot, 3-span continuous I-beam bridge. This assumed the beams to be resisting negative moment (at the piers) as part of the interior stringers for a four stringer cross-section. The calculations made were consistent with accepted bridge design practices, standard specifications[1], and theoretical methods of determining flexural stresses[14].

The stress-load diagram (figure 5) shows the effect of external loads on the stresses in the bottom flange and the bottom cover plate of the beams. The stress relationships for the top flange and top cover plate are the same as shown except opposite in sign. The stress shown for zero load represents the theoretical prestress at the center of the span

*The term live load as used throughout this report includes an allowance for the dynamic effect (impact) of the design vehicle.

(table III) in that particular element of the beam. Since the Ames beam showed the higher prestress value, it was considered as governing in the designed loading. Both beams were planned for the same test loading.

The repeated test loading as it was designed is equal to an H-15 live load (figure 5). This loading, when superimposed upon the dead load, results in theoretical repeated stresses in the bottom cover plate of about 34,000 psi to 52,000 psi tension. The corresponding minimum and maximum loads are 24,400 lbs and 51,900 lbs, respectively. As will be noted later, it was not possible to obtain these exact loads during the actual tests.

Predicting the stress range that would be expected to cause a fatigue failure at about two million cycles involved the consideration of test data on as-received USS T-1 steel plates[4, p. 98] as well as a report on fatigue of steel beams with welded cover plates[16]. Stresses from 34,000 psi to 64,000 psi tension (30,000 psi range) in the bottom cover plate are indicated as being the range for fatigue of as-received T-1 plates (figure 5). This was reduced by 40 percent or 0.67 live load to arrive at the designed range for the tests. This reduction was justified by data available on fatigue tests of steel beams with welded cover plates[16]. These data also indicated that the stress ranges in the A36 flanges due to the designed loading were substantially lower than a critical range for fatigue failure. Therefore, it was reasoned that the stress range chosen for the tests should produce a failure in the bottom T-1 cover plate at about two million cycles.

The load-deflection characteristics of the beams indicated that the testing equipment should very nearly provide the designed loading if the beams were supported on a 22 foot span and the loads were positioned (figure 5). The designed loading for the tests met the requirements.

FATIGUE TESTS

The fatigue tests were performed at the Association of American Railroads Research Center in Chicago, Illinois. The testing equipment used to apply the repeated loading was an Amsler variable speed pulsator (figure 6) and two 50,000 lb hydraulic jacks.

The Des Moines Beam and the Ames Beam were both subjected to the test loading (figures 7, 11). Each beam was supported with a knife edge at one reaction and a roller at the other, thus assuring that simple beam action would result (figure 15). The jack heads, which were free to swivel, transmitted the load directly to the top cover plate of the beam. Web stiffeners were placed on both sides of the web at the end bearings and load points. These were tack welded to the inside of both the top and bottom flanges. The stiffeners that had been placed during fabrication were not removed. Steel straps were placed at the third-points of the test span and provided lateral support for the compression flange. The lateral support seemed advisable at least from a safety standpoint.

The instrumentation involved was four SR-4 electrical resistance strain gages, two placed on the top and two on the bottom cover plate between the load points and a taut wire and scale arrangement to observe the centerline deflection. A check on the static load-strain and static load-deflection characteristics of the beam was maintained by the instrumentation.

The repeated load was applied simultaneously through both hydraulic jacks at the rate of 130 cycles per minute. The pulsator, working at its capacity, varied the load from a minimum of 26,800 lbs per jack to a maximum of 50,000 lbs. Thus, the exact designed loading was not realized (figure 16).

During each test day the repeated loading took 16 consecutive hours. The testing was conducted only on the normal work days. A typical day proceeded in the following manner:

(1) Obtained successive readings for strains and centerline deflections for zero load, 25,000 lbs per jack, and 50,000 lbs per jack. These readings were compared with previous ones to ascertain any possible change in the elastic properties of the beam.

(2) Conducted a visual inspection for fatigue cracks while under the 50,000 lb jack loads.

(3) Started the repeated test loading.

(4) Checked periodically on the maximum and minimum loads as well as for fatigue cracks.

(5) Stopped the pulsator at the close of the test day.

Each of the two beams failed in fatigue due to the applied loading; they were then returned to Iowa State University for an experimental determination of their existing stresses. Damage to the beams was limited to the immediate region of one of the load points where a tensile-type crack occurred.

DETERMINATION OF EXISTING STRESSES

There was much discussion and speculation before and throughout the tests as to the actual state of stress in the beams. It was realized that the actual stresses and computed stresses were not identical. To understand their fatigue strength better it was decided to conduct an experimental investigation of the stresses in the beams after the fatigue tests were completed. A combination of the following would be present: residual cooling stresses, welding stresses, and prestress.

To determine experimentally the state of stress, a method was adopted that required gaging the portions of the beam to be investigated, then measuring the resulting strain after each gaged strip had been removed by sawing[8]. The corresponding average stress was obtained by multiplying the unit strains by the modulus of elasticity for the material.

A Whittemore mechanical strain gage with a 10 inch gage length was used to measure the strains that occurred as a result of the cutting. The least count of the Whittemore was 0.0001 inch; therefore each division of the dial represented a unit strain of 10 micro inches per inch. An invar reference bar was used throughout. The gage holes to accommodate the Whittemore were made by drilling a hole about $\frac{1}{8}$ inch deep with a number 56 drill, then finishing by gently center-punching. This left a uniform shoulder for the points of the gage so that consistent readings could be obtained.

Figure 17 shows the sawing and gaging locations for the three 11.5 inch segments which were gaged and cut from each beam. This was done in an effort to study the variation of the state of stress along the length of the beams. The cross section shows the location of the gage lines and sawing lines as arranged in each segment. This arrangement made it possible to study the distribution of stress up and down the web of the beam as well as across the flanges and cover plates. The basic strip width was $\frac{1}{2}$ inch in the flanges and cover plates and 2 inches in the web. The only exception to this was at the junction of the flange and web and also the extreme outside strips of the cover plates. These outside strips were not gaged on the Ames beam. The T-shaped pieces at the junction of the flange and web each had two gage lines while the other strips had only one.

An identification system was devised to define the location of each gage line completely. This may be illustrated by considering an example such as ABP3-24 (figure 23). The letters ABP denote that the gage line is in the bottom plate of the Ames beam. The number 3 to the left of the hyphen represents the third segment along the beam (figure 17) while the 2 at the right of the hyphen denotes side 2 of the beam. The gage number 4, represents the fourth gage line from the outside of the flange or plate. The gage numbering for the web is from the bottom of the beam to the top (figures 21, 26).

The sequence for taking readings on the gage lines involved an initial set after cutting the beam at the fracture and before sawing into segments, a second set after removal of the three segments, and a final set when the individual strips were completely removed from each of the segments. The air temperature where the readings were taken was very nearly constant at all times; consequently no corrections for temperature were necessary.

To ascertain what effect the fatigue crack had on the state of stress in the individual segments, strain readings were obtained after successively cutting the Des Moines beam into shorter lengths. The initial set of readings was taken in the bottom plate at segment 1 after the beam had been cut at the fracture location to a length of about 15 feet. This is denoted as segment AF (figure 18). Successive sets of readings were taken in the bottom plate of segment 1 following sawing to form segments aF, bF, cF, dF, ee, ff, gg, and hh. Each set of readings was averaged and expressed as a percent of the total change occurring after final cutting. These were plotted (figure 18), and the resulting curve shows that very little change occurred until segment ff, which is 3 feet long, remained. Therefore, it is apparent that the state of stress was not appreciably affected when the fatigue fracture occurred.

RESULTS AND DISCUSSION

The results of this investigation are in four parts. In the first part the behavior of the beams under repeated loading and their subsequent failures is given. The second part presents the results of the determination of existing stresses. In the third phase test data are analyzed and the fatigue strength is evaluated. The fourth part presents some possibilities for future study.

FATIGUE FAILURES

Each beam failed as a result of a sudden fatigue fracture,which was accompanied by a loud report. The daily strain and deflection readings gave no indication of an impending failure or of a change in the elastic properties of the beam (table IV).

Table IV. Summary of theoretical stresses and fatigue failures.

Summary item	Ames beam	Des Moines beam
Minimum tensile stress in T-1 (psi)	35,000	30,900
Maximum tensile stress in T-1 (psi)	50,100	46,000
Minimum tensile stress in A36 (psi)	300	4,000
Maximum tensile stress in A36 (psi)	14,600	18,300
Fatigue crack first occurred	unknown	T-1
Cycles at failure	2,469,100	2,756,100

Observations made several hours before the fracture of the Ames beam failed to detect the formation of any minute cracks. This does not mean that there were no such cracks but that if there were they were not discovered.

A small crack was observed in the Des Moines beam approximately 175,000 cycles before it caused a complete failure. This crack was confined to the portion of the bottom T-1 plate extending beyond the edge of the flange and on side 2 of the beam. The crack appeared to be through the visible portion of the fillet weld but not through the fused portion of the flange and cover plate. In later observations on the Des Moines beam another crack was found on the lower flange of the A36 beam at a point 1 foot from mid-span where a stiffener had been welded. This crack was about 2 inches long and extended into the cover plate. The crack apparently had started in the weld associated with a stiffener placed during fabrication.

In observing the fractures several similarities were noted:

(1) Both fractures occurred in the shear span adjacent to the roller support. The fracture was approximately 6 inches from the load point in the Ames beam (figure 12) and 1 foot from the load point in the Des Moines beam (figure 8).

(2) The fractures formed at points of high stress concentration. In the Ames beam, this was observed to be an abrupt change in the weld area (figures 13, 14), while the Des Moines beam had a small notch on the edge of the bottom cover plate.

(3) The fractures were smooth on one side (figures 10, 13), on the other they were jagged and torn (figures 9, 14). The similarities of the two fractures and the fact that the Des Moines beam first cracked on the smooth side of the flange (side 2) make it reasonable to assume that the Ames beam also cracked first on the smooth side (side 1).

(4) Evidence of yielding was seen in the portion of the web adjacent to the fractures; therefore much of the web did not fail as a result of fatigue.

(5) Discoloration, appearing to be rust, was on certain areas of the fractured welds, indicating a minute fracture before the fatigue tests began.

EXISTING STRESSES

The results of the experimental study to determine the state of stress in the beams were plotted (figures 19 to 28). A modulus of elasticity of 29,000,000 psi was used in converting all the unit strains to unit stresses. Several observations should be noted:

(1) The results obtained after removing the segments from the beam and before cutting the strips from the segments were not plotted.

(2) The welds were areas of extremely high tensile stresses.

(3) An accurate determination of the stress in the high stress areas was not possible due to the rapid change in its magnitude.

(4) Rapid stress variation was evidenced within individual strips by their distortion after sawing.

(5) The strain measured in each strip was assumed to be the average strain for that strip.

(6) Only elastic strains could be measured.

ANALYSIS OF DATA

It has been noted[4] that fatigue failure is determined by three factors: 1st) the maximum stress in the stress cycle, 2nd) the stress ratio or range of stress in the stress cycle, and 3rd) the number of applications of the stress cycle.

There have been various graphical methods devised by which these factors may be related into a useful combination. Before proceeding with the analysis of the data, it is desirable to acquaint the reader with the diagram that will be used to make this analysis. The most direct approach is through its derivation from an elemental S-N diagram (figure 29). For discussion purposes it is assumed that the material in the specimens has an ultimate tensile strength of 80,000 psi. It is important to realize that all the specimens referrred to are of the same material.

The curves in figure 29 represent the fatigue properties of the material as determined by four groups of tests. Curve 1 represents results that might be obtained by subjecting the specimens to a zero to tension stress cycle. The maximum stresses in the stress cycles would be represented by the ordinates of the curve. This curve could be described as the S-N curve for zero to tension loading. Curves 2, 3, and 4 represent a tension to tension loading with each one being established by maintaining a different minimum stress and using various maximum stresses for the different specimens. As before, the ordinates for the curve are the maximum stresses.

Using information from the S-N curves, a diagram can be plotted that will provide the minimum and maximum tensile stresses to which the material may be subjected any given number of times before failure. A fatigue strength diagram of this nature for failure at two million cycles, for example, can be obtained by plotting the information from the S-N curves at points a, b, c, and d (figure 30). The best curve to fit this type of plot has been found to be a straight line[4]. It is evident that the line must pass through the ultimate strength of the material. Any point on line ae has as its coordinates the minimum and maximum stresses to which the specimen could be subjected two million times before a fatigue failure would be expected. A graphical representation of the allowable stress range can be obtained by a vertical line between the two million cycle line and the static strength line. For example, the allowable maximum stress corresponding to a minimum stress of 15 ksi tension is found at point f to be 38 ksi tension. The length of line fg represents 23 ksi which is equal to the allowable stress range. The minimum stress is at g and the maximum stress is at f as indicated.

The presentation of the fatigue information (figure 30) lends itself readily to a pictorial study of the fatigue strength of a specimen. This type of diagram will be used in discussing the fatigue results of the prestressed steel beams.

The right portion of figures 31 and 32 show the fatigue strength diagrams for the Ames beam and the Des Moines beam, respectively. Two materials were involved in this investigation, namely T-1 and A36 steel. Due to the limited fatigue data available only two points on a two million cycle failure line could be established for each steel. These two points were the fatigue strength for zero to tension loading[4, p. 93] and the static ultimate strengths of the materials (table I). Connecting the two known points with a straight line provided suitable information for this study. To the left in each figure is a stress-load diagram. The highest tensile stresses found experimentally at segments 1 and 2 for the bottom cover plate and the bottom flange of the test beams are plotted on the vertical stress scale. These are identified by the strip designation used in the state of stress study. The theoretical stresses at the center-line of the span for the bottom flange and the bottom cover plate are also shown.

The use of these combined diagrams (figures 31, 32) will be illustrated by considering the results shown for DBP1-22. This strip was observed to have an average residual tensile stress of 83,000 psi while it was in the beam (figure 28).

Superimposing the stresses due to the minimum load of 26,800 lbs and the maximum load of 50,000 lbs per jack, results in stresses as shown at points a and b, respectively. When these minimum and maximum repeated stresses are projected horizontally to the fatigue strength diagram (a' and b'), it is indicated that the stress range is critical for failure in less than two million cycles (figures 31, 32; table V). It is interesting to note that the A36 steel in the bottom flange of either beam was not stressed in a critical range for failure at two million cycles. For this reason it is logical to assume that both failures started in either the welds or the bottom T-1 steel cover plates.

The reason the fatigue failures occurred in the beams becomes apparent from the above discussion even when it is realized that the stress

Table V. Critical stresses for the Ames beam and the Des Moines beam.

Gage designation	Residual tensile stress (ksi)	Minimum tensile stress in stress cycle (ksi)	Maximum tensile stress in stress cycle (ksi)	Two million cycle failure expected
ABP1-21*	74.7	92.1	107.2	yes
ABP2-21	78.7	86.4	92.9	no
ABF1-11	9.0	24.5	37.9	no
ABF2-11	32.4	39.3	45.3	no
DBP1-22*	83.2	100.6	115.7	yes
DBP2-12	106.2	113.9	120.4	yes
DBF1-21	7.4	22.9	36.3	no
DBF2-21	27.7	34.6	40.6	no

*This strip is comparable in location and stresses to the actual fatigue fracture.

range applied was only 50 percent of the predicted range for as-received T-1 plates (figure 5). There were high tensile stresses in the vicinity of the welds, thus reducing the allowable repeated range of stress for failure. Determination of the state of stress theoretically provides only the average stress across any section. Such information is inadequate where fatigue failures are of concern. It is the highest stress present at any section, combined with the repeated range of stress at the same location, which determines whether or not that section will fail in fatigue.

SUGGESTED FUTURE STUDY

As a result of this investigation several areas for possible future study are as follows:

(1) Study the fatigue strengths of various members in view of the magnitude of their existing stresses as determined experimentally.

(2) Investigate identical prestressed and unprestressed steel beams to ascertain any possible differences in their fatigue strength.

(3) Study the fatigue strength of fillet welds connecting the new high strength steels.

The method used to make this investigation proved to be very satisfactory. The minor difficulties that arose were mostly due to the size of the test specimens and to the hardness of the T-1 cover plates. The gaging and sawing techniques were time-consuming and expensive, but they were very fruitful. An adequate number of gage lines were used except in the high stress areas. An improvement would be to take narrower strips, but extreme care would be necessary while sawing.

CONCLUSIONS

1. The flexural fatigue strength of a prestressed steel beam can be computed. The fatigue strength depends largely on the maximum residual stress in the tension cover plate.

2. The stress conditions were such that fatigue failure might have occurred in the outside edges of the bottom cover plate at any location within the middle one-third of the span.

3. A stress concentration was the deciding factor in the actual location of each fatigue fracture. In the Ames beam the failure apparently began at a section where there was an abrupt change in the area of the fillet weld. The failure crack in the Des Moines beam started in a small notch at the edge of the bottom cover plate.

4. Although the small number of specimens studied here does not make it conclusive, it appears that prestressing did improve the fatigue performance of the A36 beam by reducing the weld stresses in the bottom flange of the beam.

5. Each of the beams sustained over two million repetitions of load equivalent to 84 percent of an H-15 live load plus impact. This was considered to be an acceptable resistance to fatigue failure.

ACKNOWLEDGMENTS

This investigation has been conducted under the auspices of the Engineering Experiment Station of Iowa State University and was sponsored by the American Institute of Steel Construction, the Pittsburgh-Des Moines Steel Company, the Clinton Bridge Corporation, and the Iowa State Highway Commission. The facilities of the Research Center of the Association of American Railroads in Chicago, Illinois, were used for the fatigue testing of the specimens. The portion of the investigation devoted to the determination of residual stresses was carried out in the Engineering Experiment Station at Iowa State University.

A most pleasing aspect of this work was the ease and competence with which the extensive fatigue tests were conducted at the Association of American Railroads' facilities in Chicago, Illinois. This was due to the efforts of Mr. E. J. Ruble, Executive Vice President for Research, and Mr. F. P. Drew, Research Engineer-Structures. Mr. G. A. Hinueber, engineer-in-charge of the Structures Laboratory, helped in the solution of numerous problems particularly at the outset of the testing. Messers I. A. Eaton and R. J. Trubic, Laboratory Technicians, carried out the actual test operations under the personal supervision of Mr. Drew.

The cooperation and assistance of many persons on the Iowa State Highway Commission staff are appreciated. S. E. Roberts, Head of the Research Department, and C. A. Pestotnik, Bridge Engineer, provided supplementary information concerning the fabrication of the test members as well as administrative help in connection with setting up the project. Others on the Iowa State Highway Commission staff who contributed were P. F. Barnard, John Roland, and Y. H. Gee. The advice and consultation of Dr. Ti-ta Lee, Assistant Professor of Civil Engineering, in connection with the measurement of residual stresses was of vital importance. L. A. Facto, Assistant Professor, took numerous photographs, and Mr. G. E. Blumberg made the drawings. Mr. C. N. Spicer, in charge of the Engineering Shop, supervised the sawing operation for determination of residual stresses.

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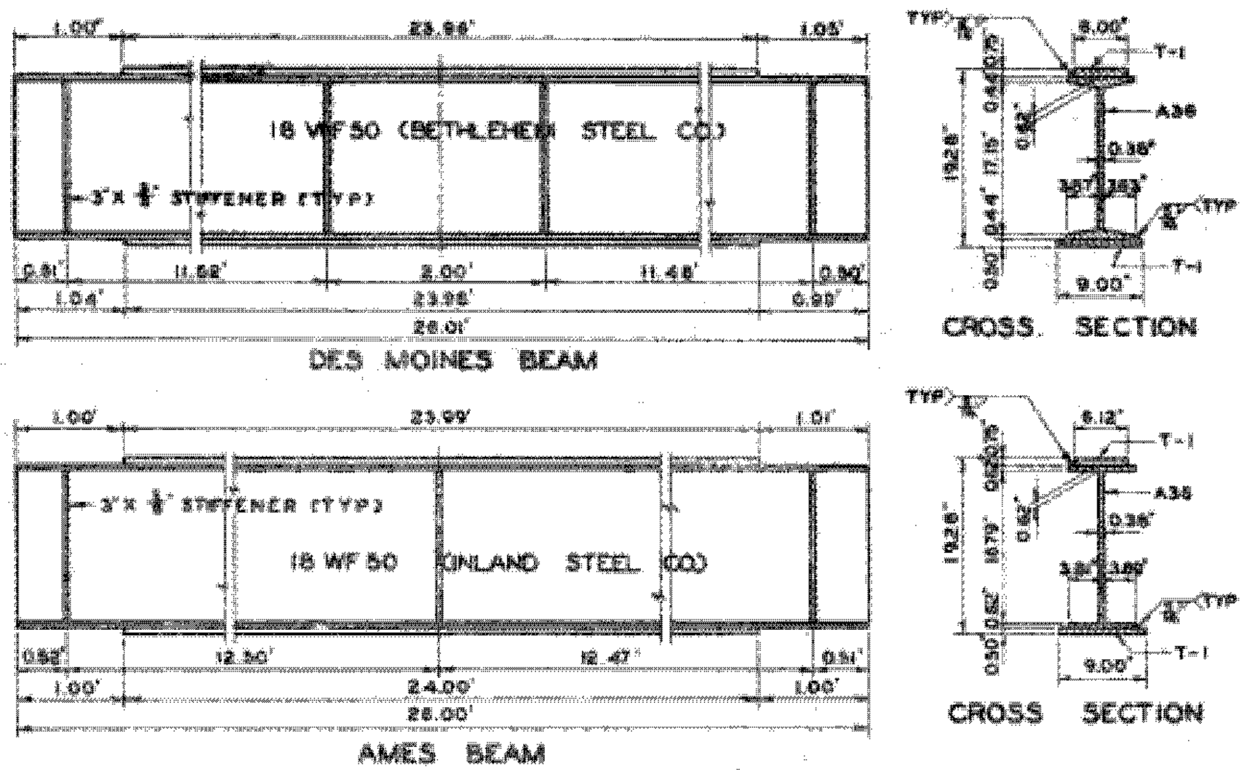

FIG. I. DETAILS OF TEST BEAMS AS-RECEIVED

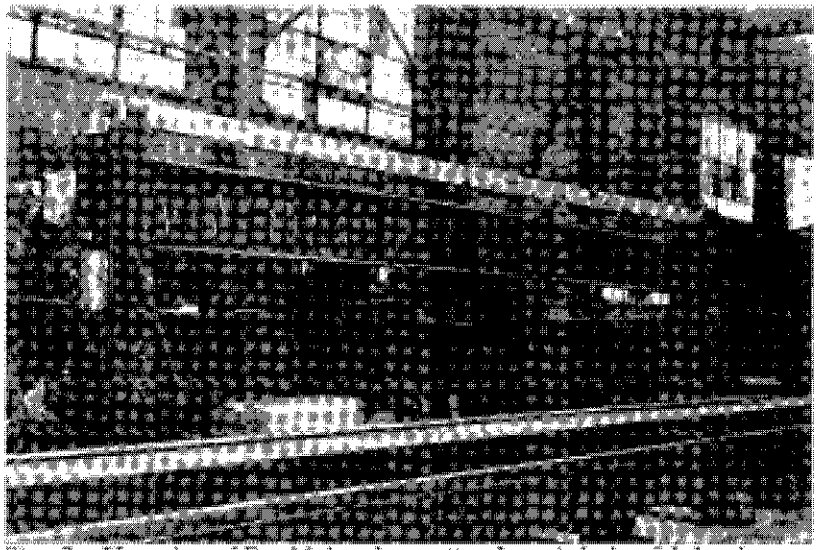

Fig. 2. Elevation of Des Moines beam (top beam) during fabrication.

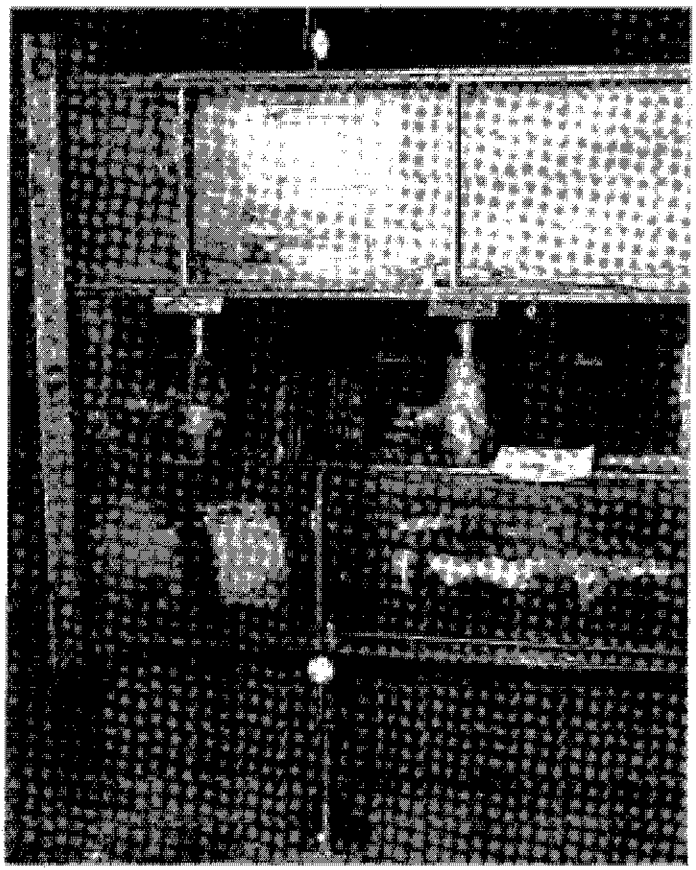

Fig. 3. Elevation at center of span during fabrication of Des Moines beam (single prestressing jack has been replaced by screw jacks).

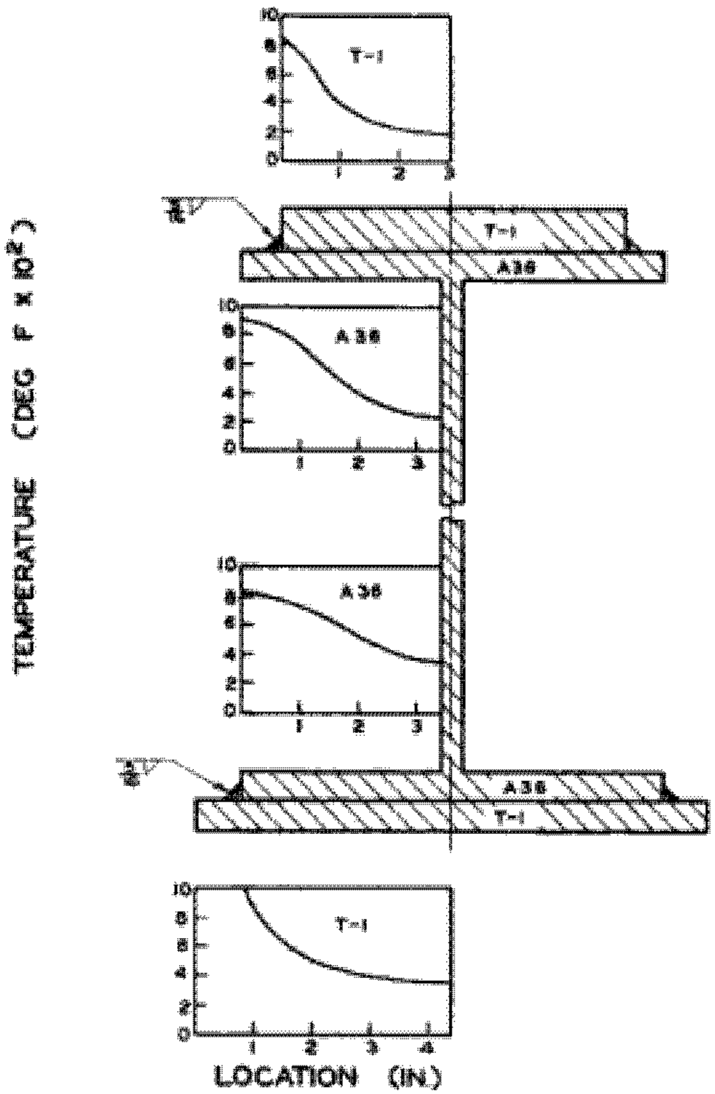

FIG. 4. TEMPERATURE DISTRIBUTION FOR AMES BEAM

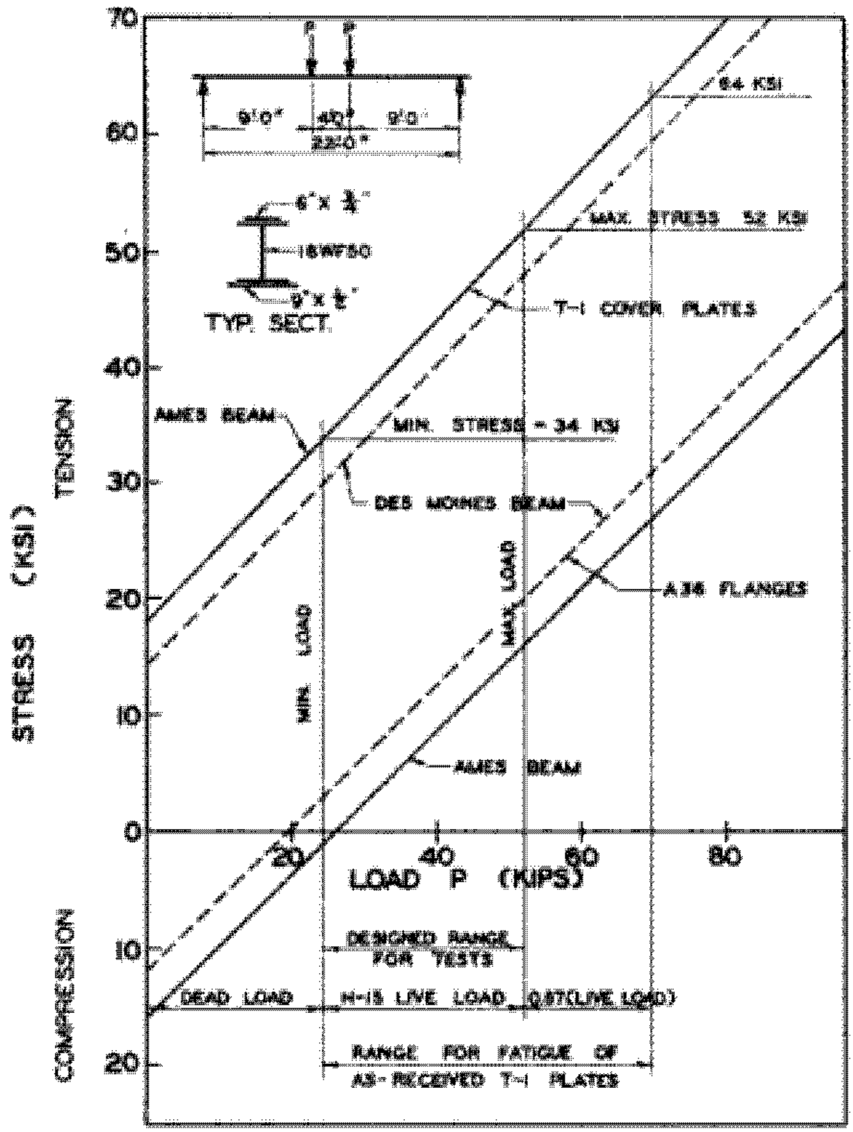

FIG. 5. STRESS - LOAD DIAGRAM FOR BOTTOM FLANGES AND BOTTOM COVER PLATES

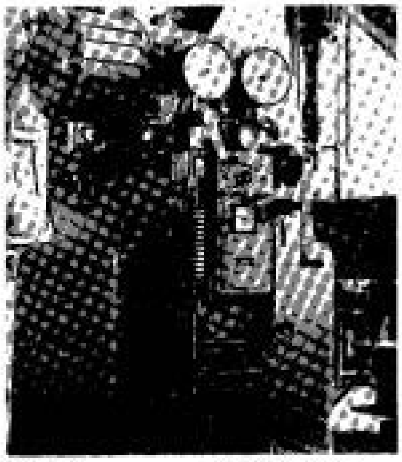

Fig. 6. Amsler variable speed pulsator.

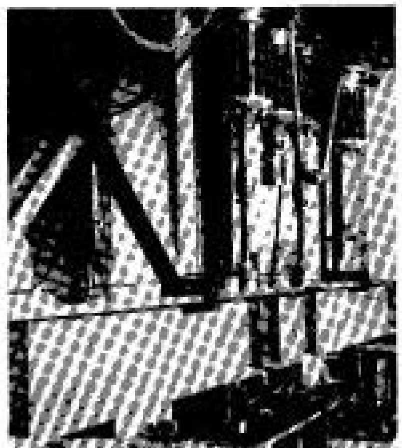

Fig. 7. Elevation of Des Moines beam (side 1) during fatigue test.

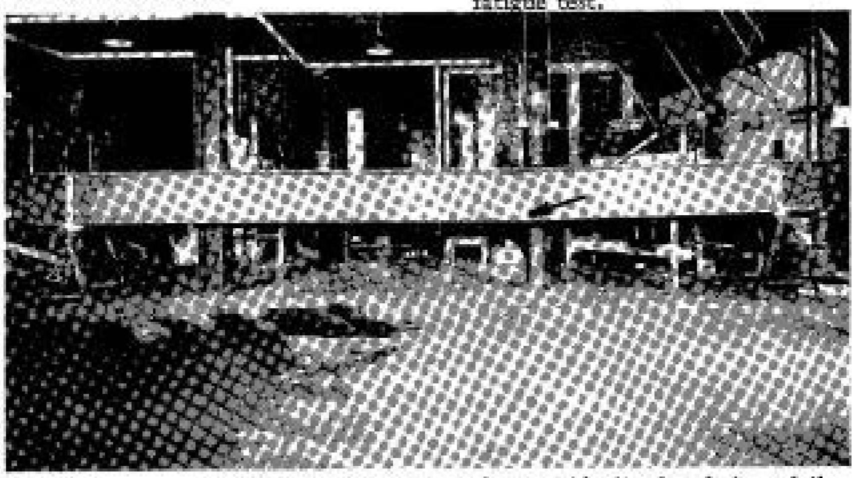

Fig. 8. Elevation of Des Moines beam (side 1) after fatigue failure.

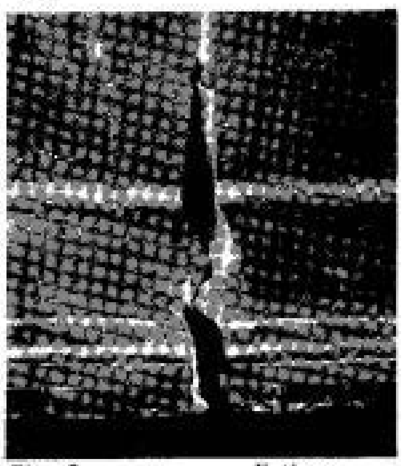

Fig. 9. Fatigue fracture in Des Moines beam (side 1).

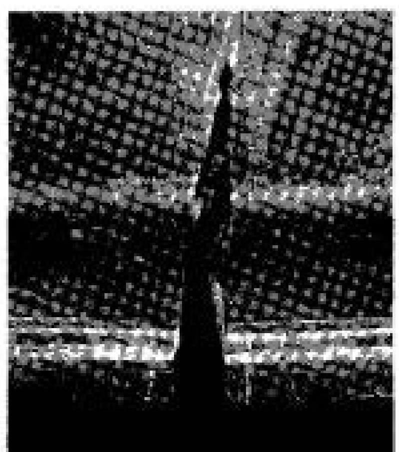

Fig. 10. Fatigue fracture in Des Moines beam (side 2).

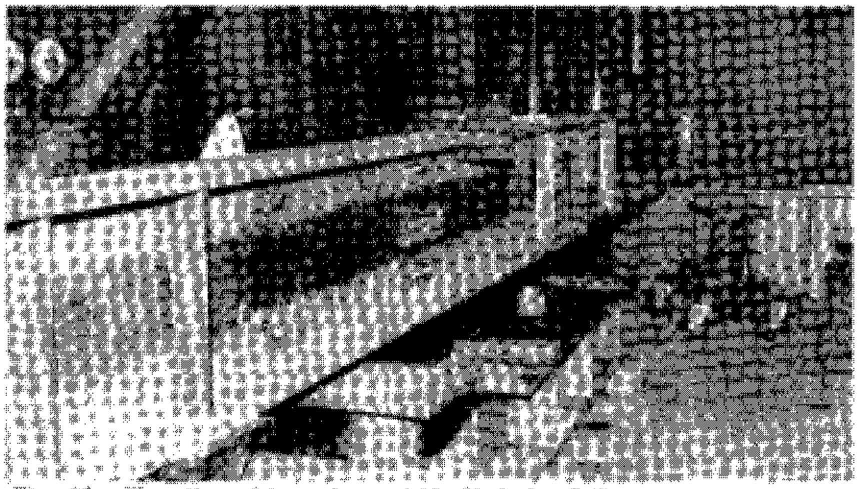

Fig. 11. Elevation of Ames beam (side 1) during fatigue test.

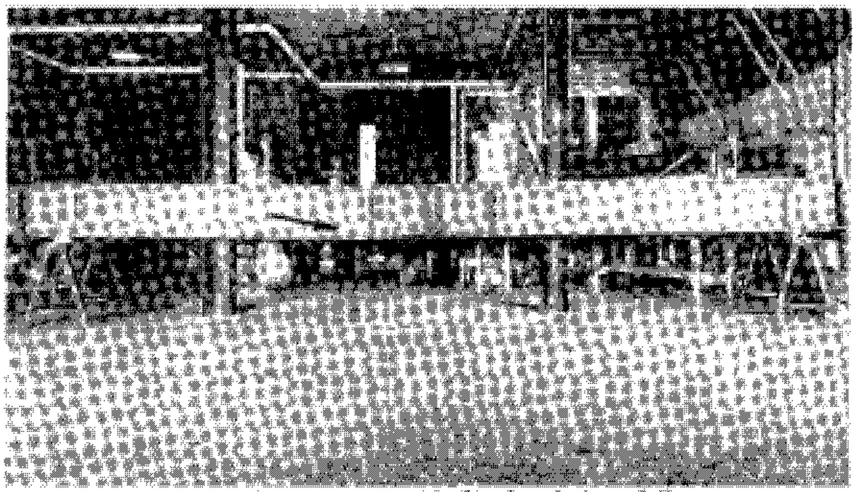

Fig. 12. Elevation of Ames beam (side 2) after fatigue failure.

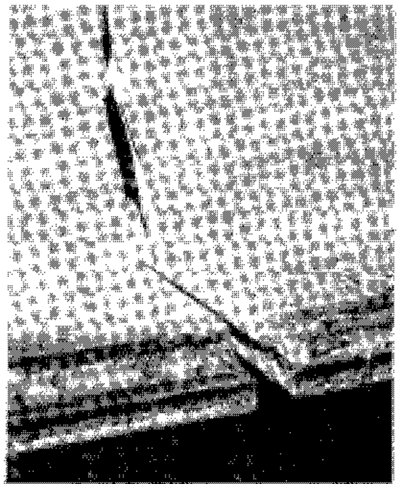

Fig. 13. Fatigue fracture in Ames beam (side 1).

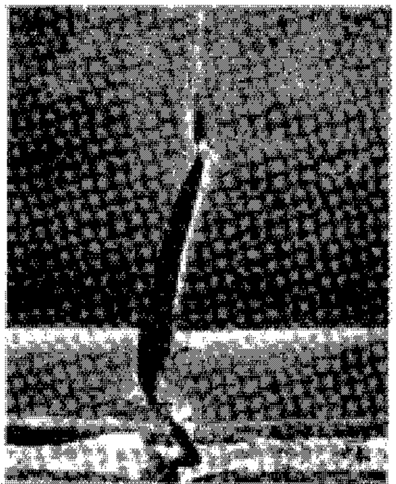

Fig. 14. Fatigue fracture in Ames beam (side 2).

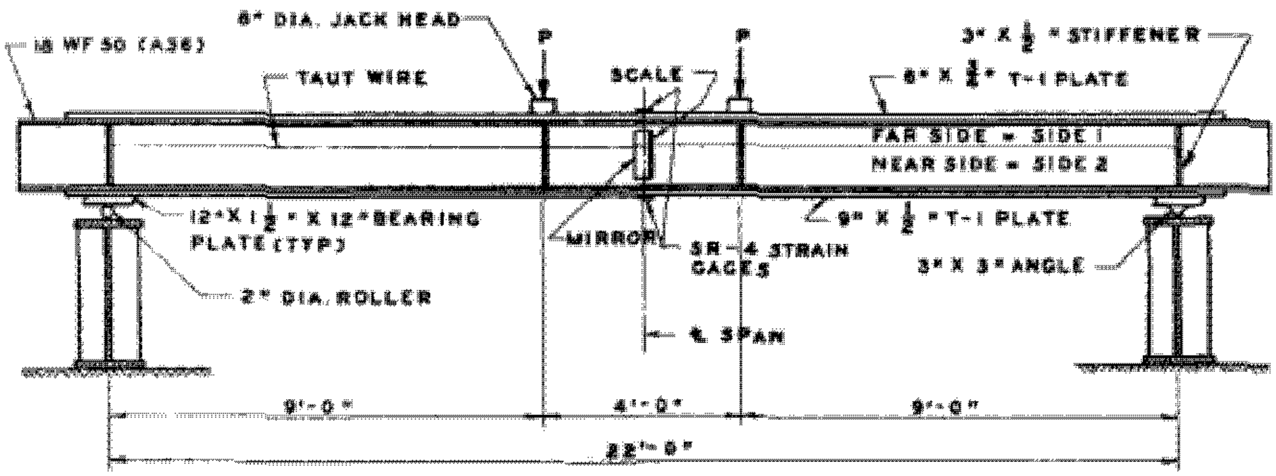

FIG. 15. DETAILS OF TEST LOADING SET-UP

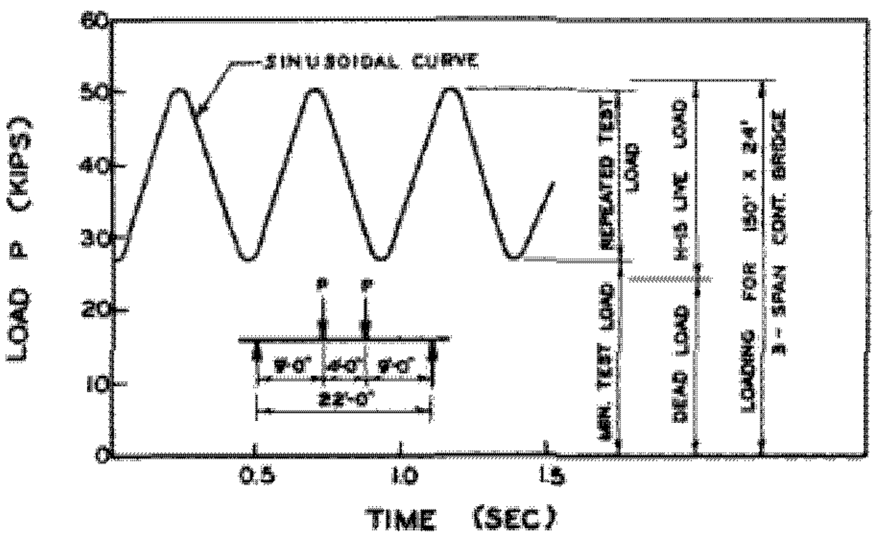

FIG. 16. LOAD-TIME RELATIONSHIP FOR TESTS

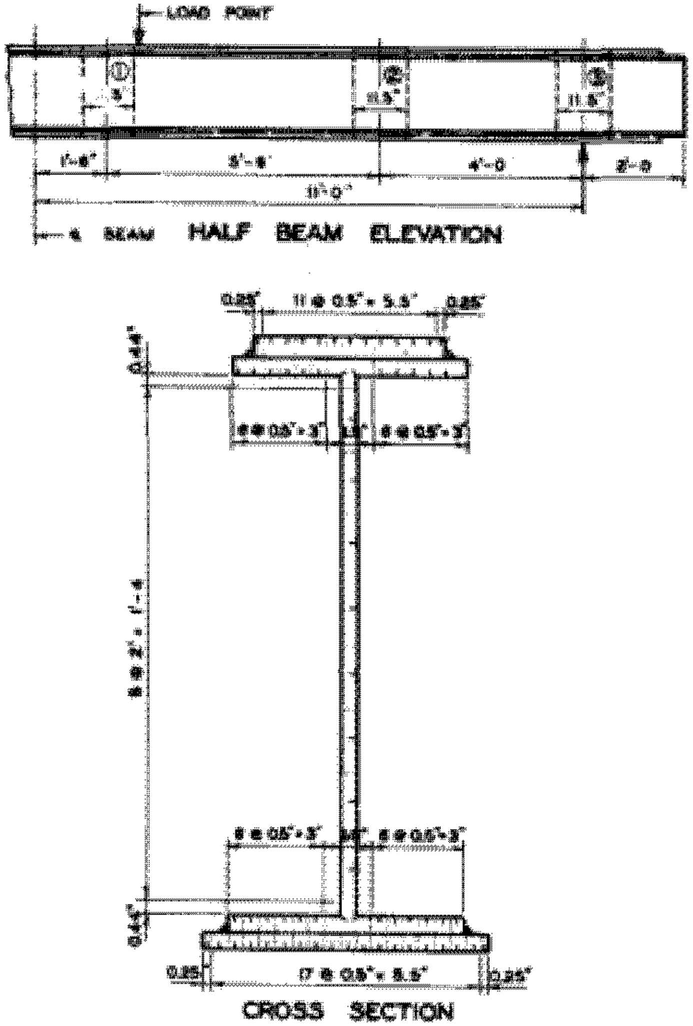

FIG. 17 SAWING AND GAGING LOCATIONS FOR SEGMENTS

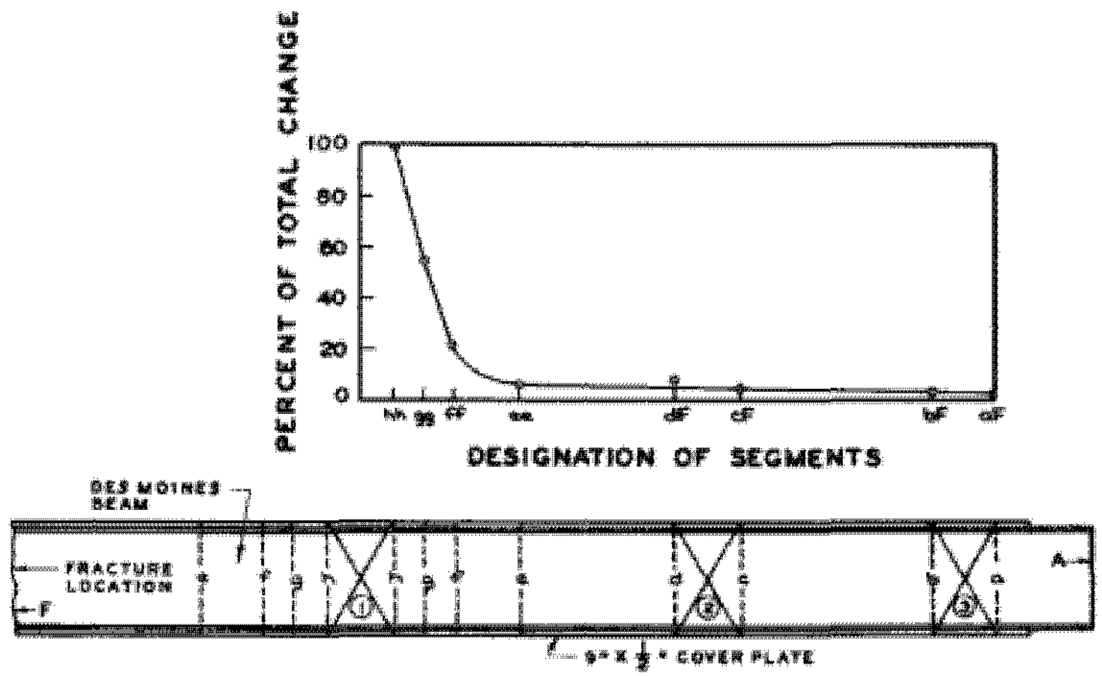

FIG. 18. INFLUENCE OF SAWING ON STRAINS IN BOTTOM PLATE FOR SEGMENT I

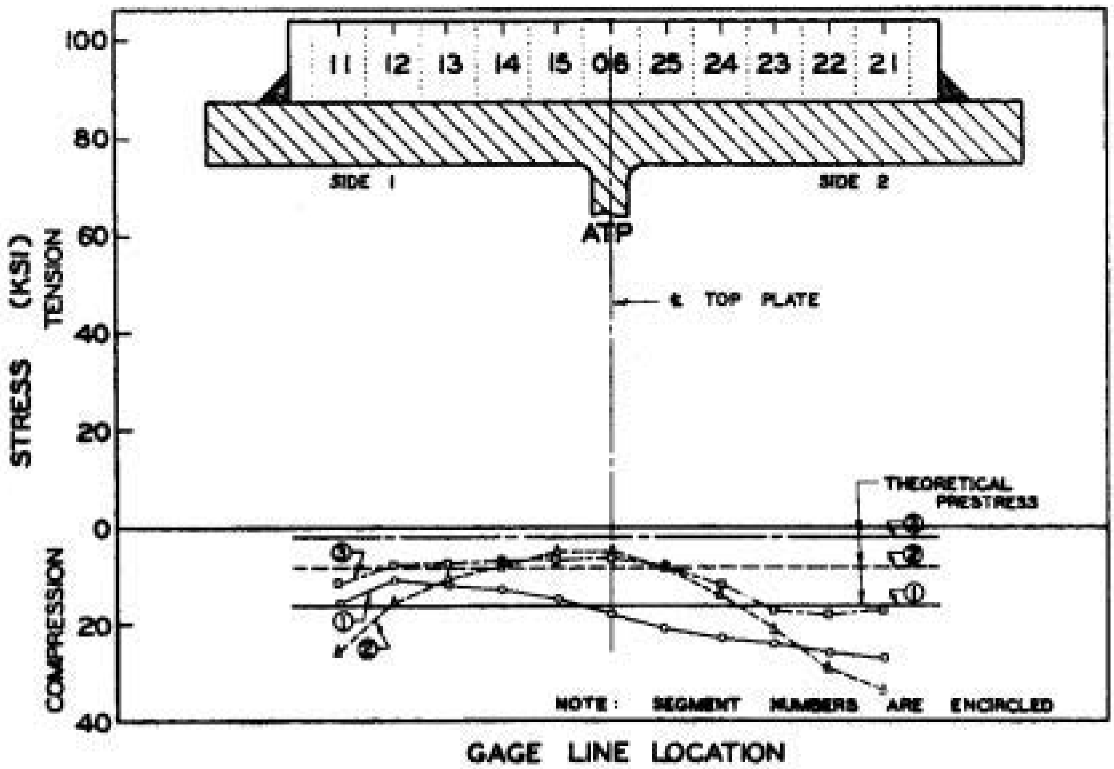

FIG. 19. STATE OF STRESS IN TOP PLATE OF AMES BEAM

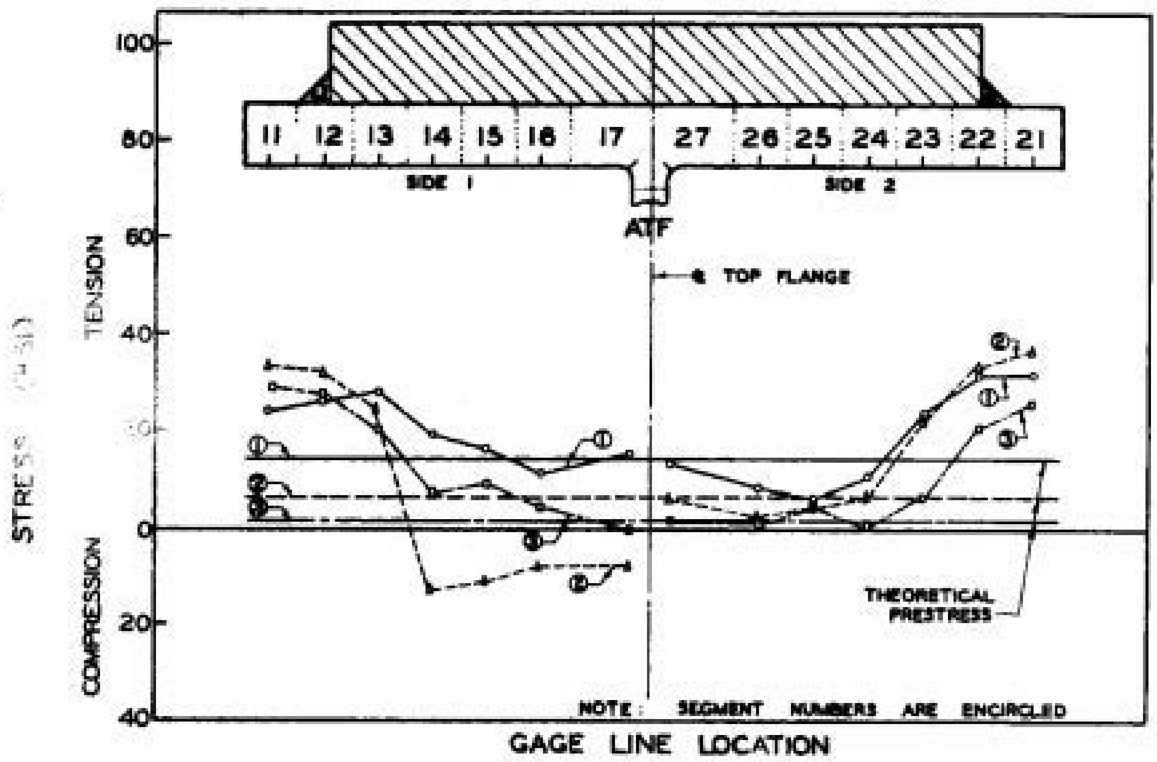

FIG. 20. STATE OF STRESS IN TOP FLANGE OF AMES BEAM

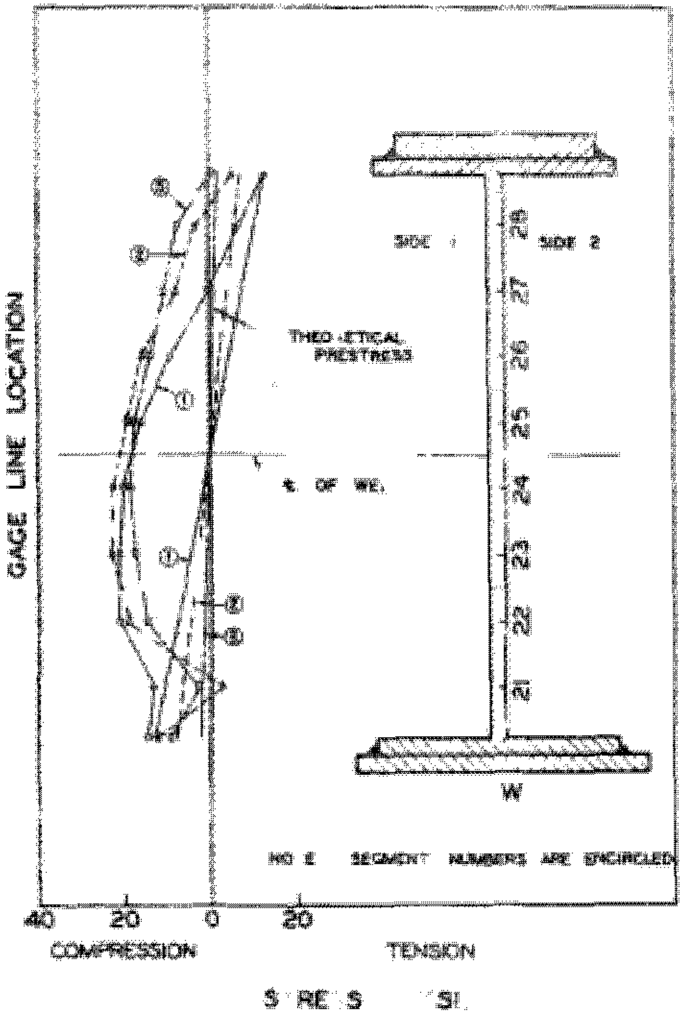

FIG. 21 STATE OF STRESS IN WEB OF AMES BEAM

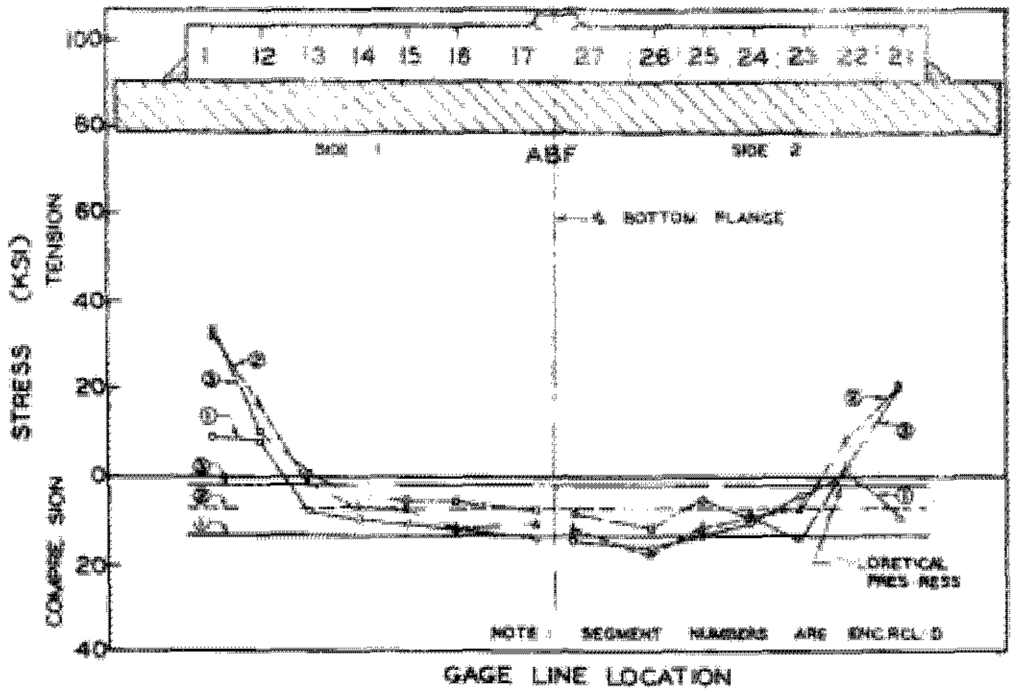

FIG. 22 STATE OF STRESS IN BOTTOM FLANGE OF AMES BEAM

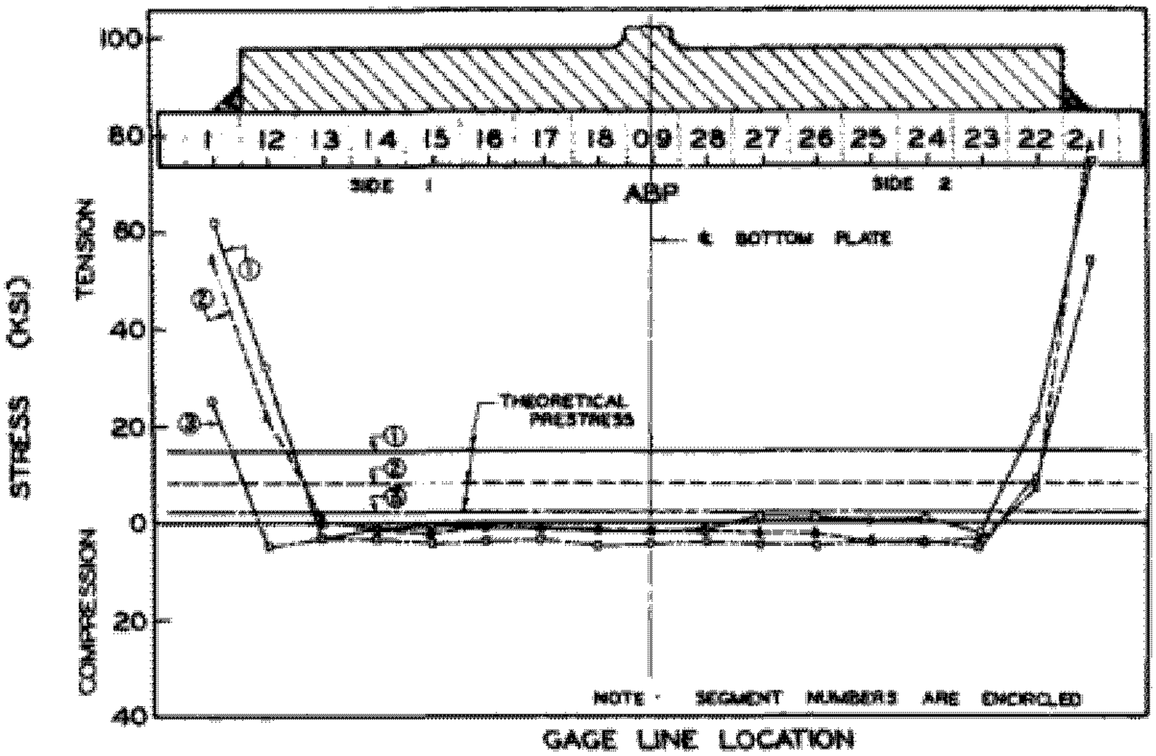

FIG. 23 STATE OF STRESS IN BOTTOM PLATE OF AMES BEAM

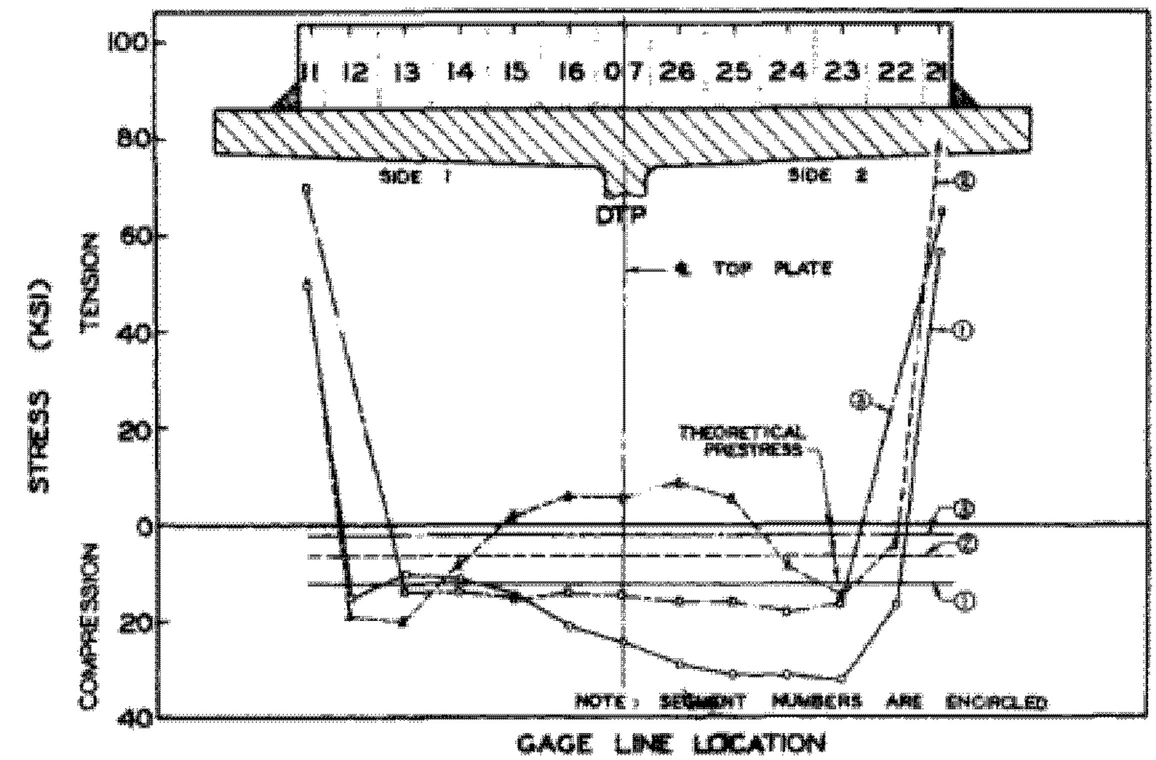

FIG 24. STATE OF STRESS IN TOP PLATE OF DES MOINES BEAM

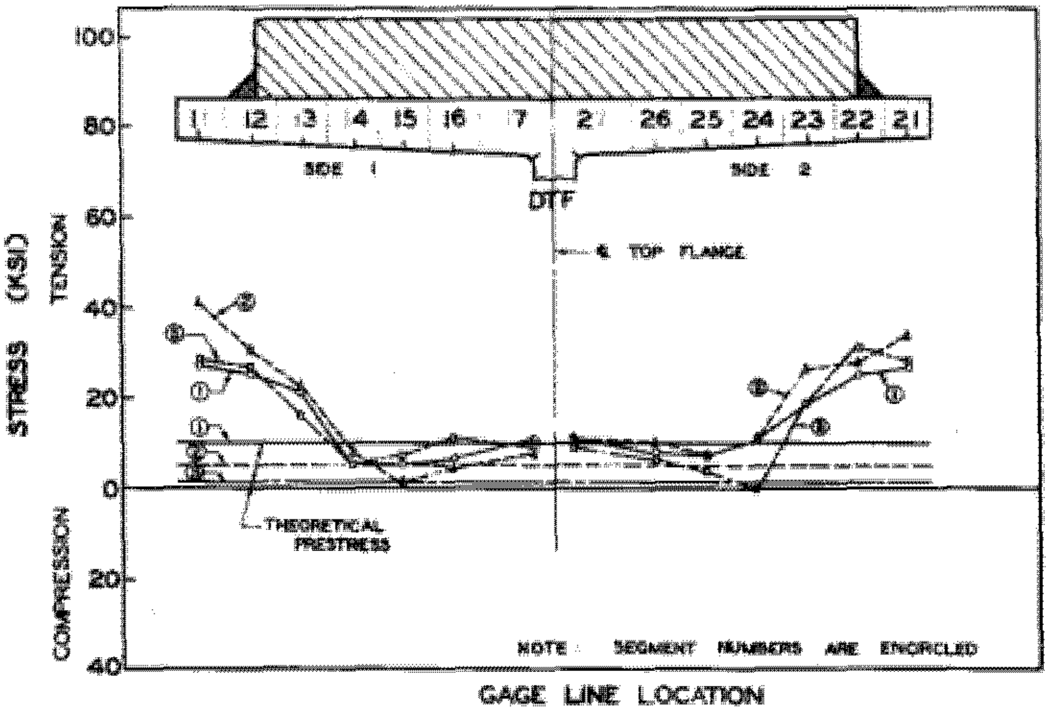

FIG. 25. STATE OF STRESS IN TOP FLANGE OF DES MOINES BEAM

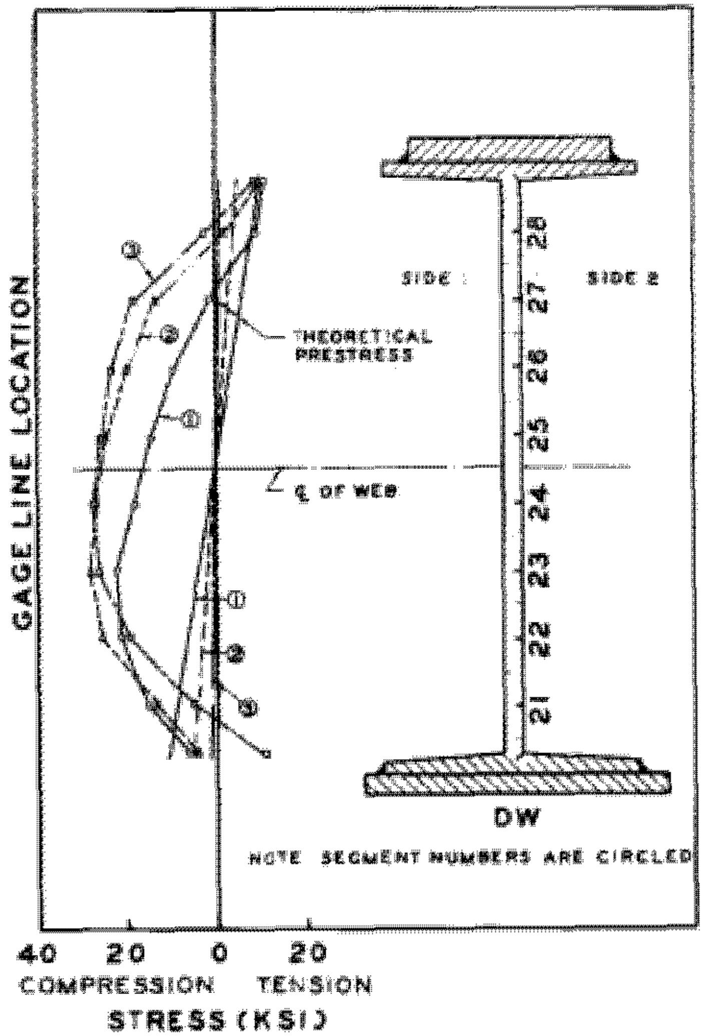

FIG. 26 STATE OF STRESS IN WEB OF DES MOINES BEAM

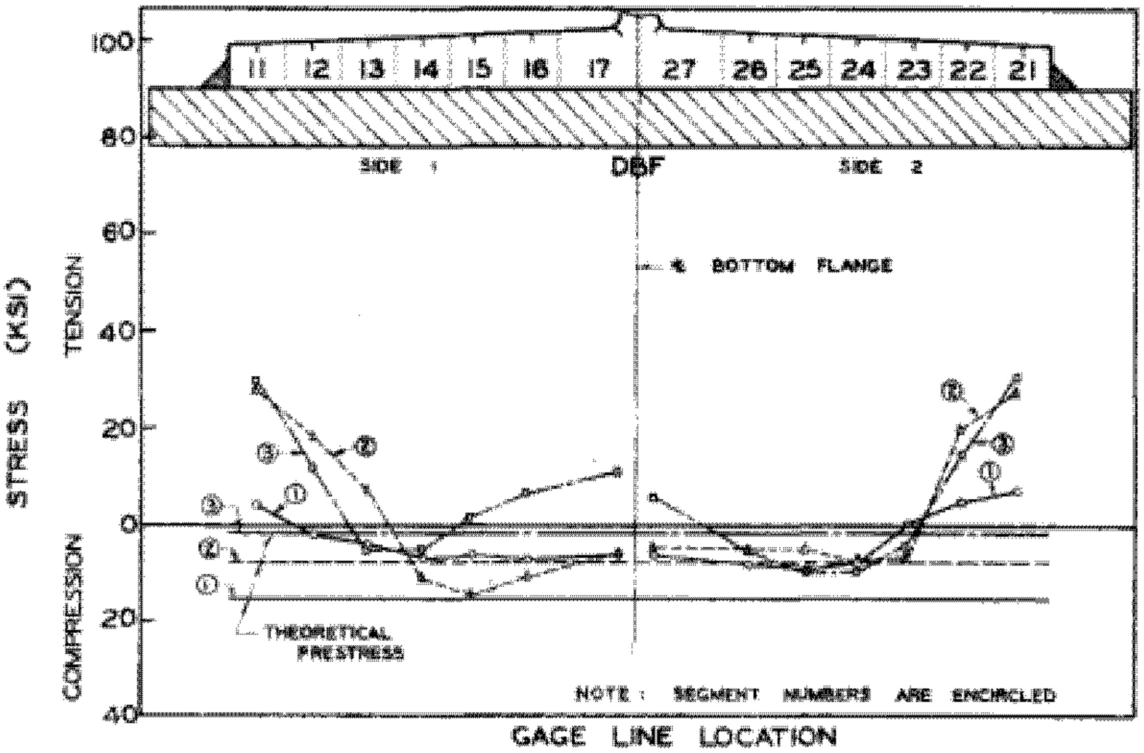

FIG. 27. STATE OF STRESS IN BOTTOM FLANGE OF DES MOINES BEAM

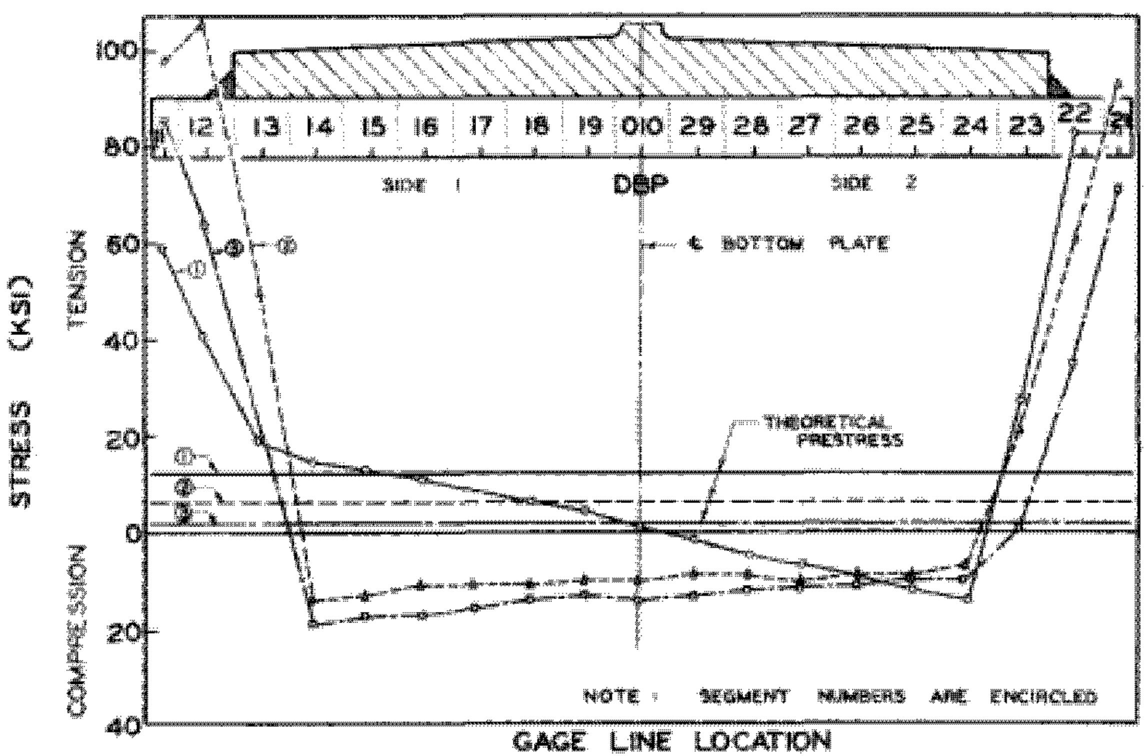

FIG. 28. STATE OF STRESS IN BOTTOM PLATE OF DES MOINES BEAM

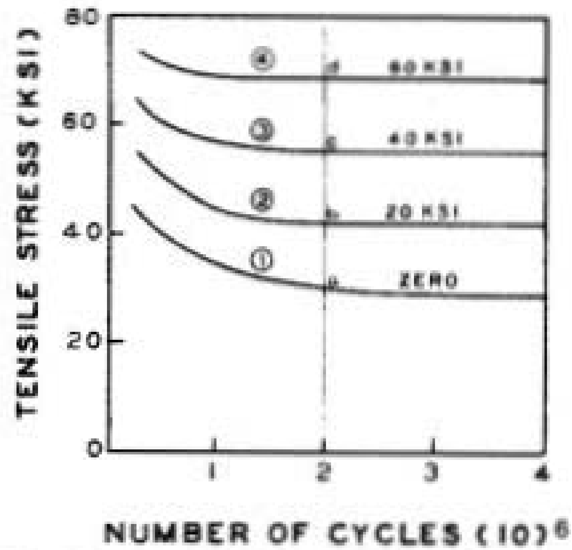

FIG. 29. EXAMPLE S-N DIAGRAM

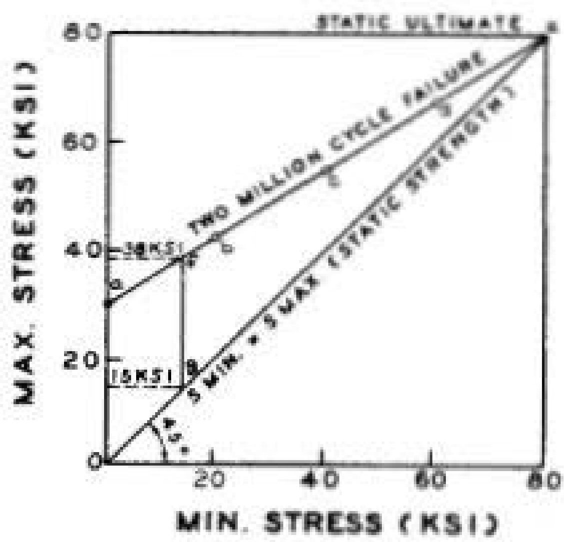

FIG. 30. EXAMPLE FATIGUE STRENGTH DIAGRAM

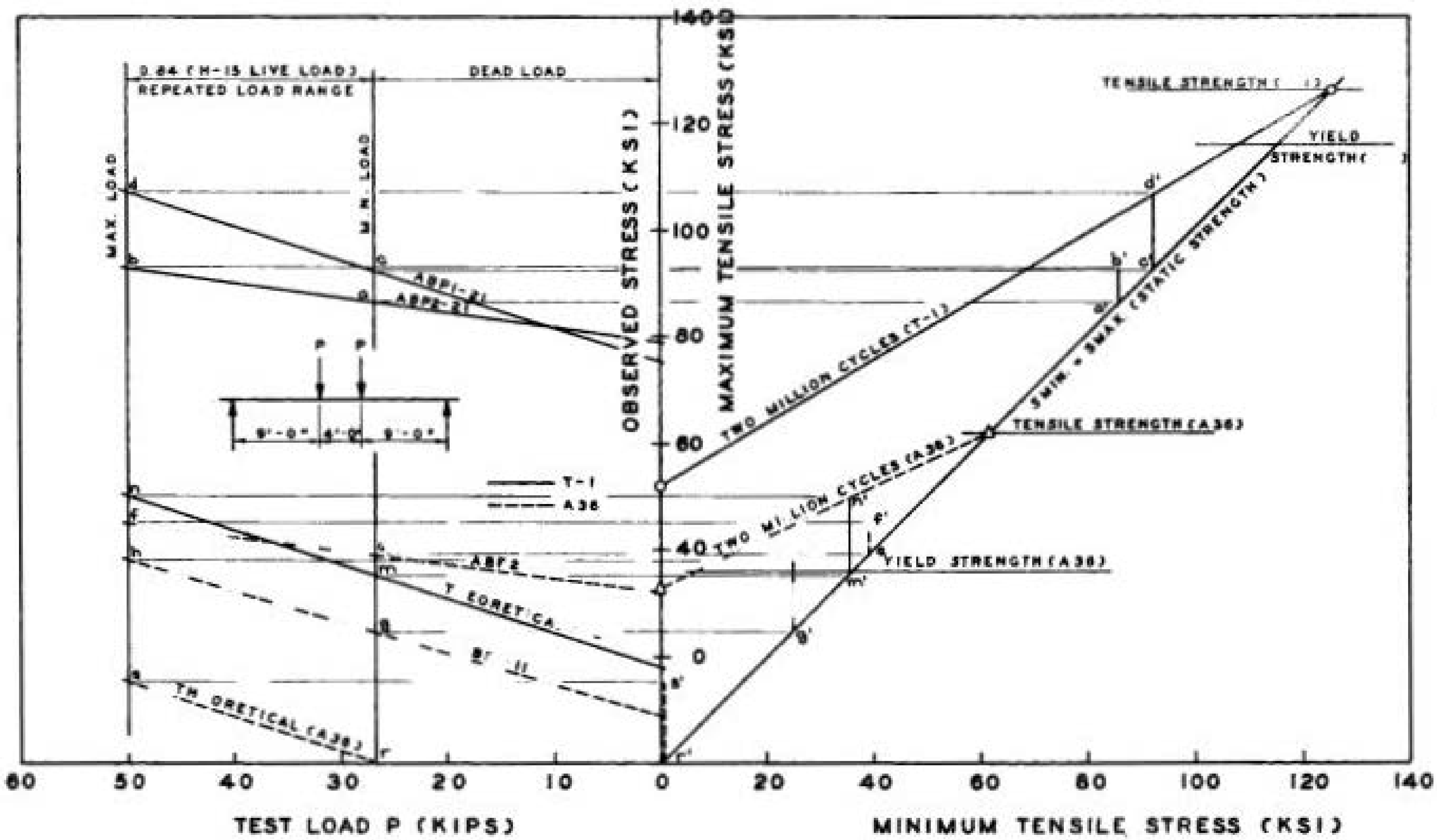

FIG. 31. STRESS-LOAD AND FATIGUE STRENGTH DIAGRAMS FOR AMES BEAM

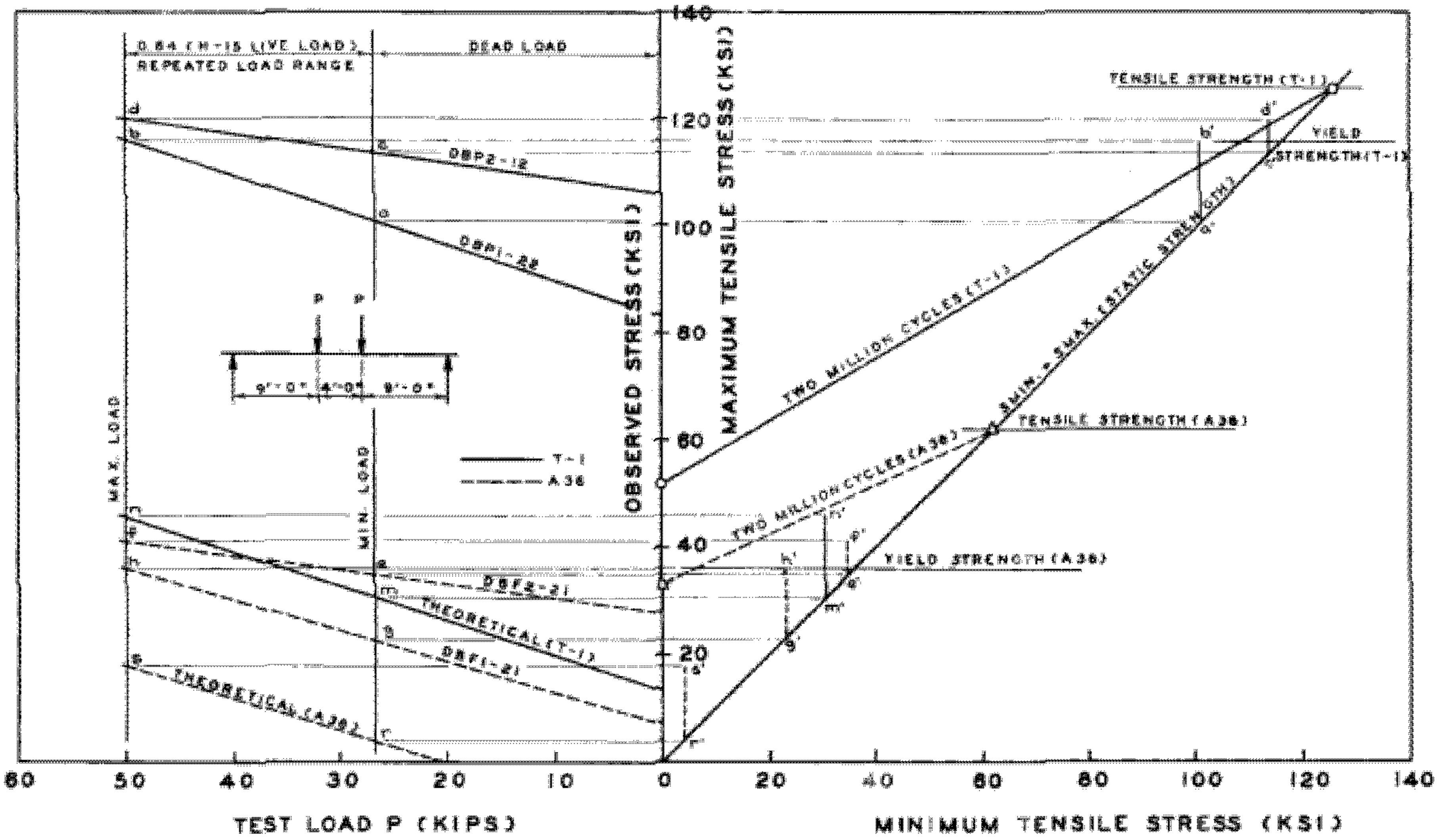

FIG. 32. STRESS-LOAD AND FATIGUE STRENGTH DIAGRAMS FOR DES MOINES BEAM