A STUDY OF THE BEHAVIOR OF REINFORCED
CONCRETE T-BEAMS SUBJECTED TO
REPEATED LOADS.

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BY

ROBERT S. PLATONI

JHRP

JOINT HIGHWAY RESEARCH PROJECT

PURDUE UNIVERSITY AND
INDIANA STATE HIGHWAY COMMISSION

Final Report

A STUDY OF THE BEHAVIOR OF REINFORCED CONCRETE T-BEAMS SUBJECTED TO REPEATED LOADS

TO: J. F. McLaughlin, Director
Joint Highway Research Project

May 6, 1969

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

File No: 7-4-15

Project No: C-36-56 (

Attached is a Final Report "A Study of the Behavior of Reinforced Concrete T-Beams Subjected to Repeated Loads." The report has been authored by Mr. Robert S. Platoni, Graduate Assistant in Research on our staff, under the direction of Professors M. J. Gutzwiller and R. H. Lee.

This research was another of a series of research studies in the structural area designed to provide an improved understanding of the mode of failure in rectangular and T-Beams subjected to repeated and static loads. The current project was concerned with repeated loads on T-Beams. The research indicated failure by fatigue of longitudinal steel reinforcement.

The report is presented to the Board for the record and for acceptance as fulfillment of the objectives of this project.

Respectfully submitted,

Harold L. Michael

Harold L. Michael
Associate Director

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REINFORCED CONCRETE T-BEAMS
SUBJECTED TO REPEATED LOADS**

by

**Robert S. Platoni
Graduate Assistant in Research**

**Joint Highway Research Project
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May 6, 1969**

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LIST OF SYMBOLS

A_s	nominal area of tension steel
a	length of critical shear span (distance from section of maximum moment to point of inflection)
b	width of beam web
C	total internal compression force in concrete
d	effective depth (measured to centroid of compression steel)
d'	distance from compression face to centroid of compression steel
E_s	modulus of elasticity of steel
E_c	initial tangent modulus of concrete
f'_c	concrete compressive strength, 6" x 12" standard cylinder
f_s	stress in longitudinal tension steel
f_y	yield strength of longitudinal steel
F	temperature in degrees fahrenheit
jd	internal moment arm
M	moment at any section
M_s	ultimate shear-compression moment
M_u	ultimate flexural moment
M_1	maximum negative moment
M_2	maximum positive moment
N	number of loading cycles
p	A_s/bd - percentage of longitudinal tension steel
P	total load

P_1	load on overhang
P_2	load between supports
P_f	predicted ultimate load
P_u	total load at failure
T	total force in tension steel
V	total shear at any section
V_a	shear in critical shear span
V_1, V_c	shear force taken by concrete
V_2	transverse shear taken by longitudinal tension reinforcement
v	nominal shearing stress = V/bd
v_c	portion of total shearing stress assigned to concrete
v_u	ultimate shear strength
S.C.	shear-compression failure
D.T.	diagonal tension failure
F.R.	fatigue of reinforcement
MII	strain in micro-inches per inch

ABSTRACT

Platoni, Robert S., MSCE, Purdue University, June, 1969. "A study of the Behavior of Reinforced Concrete T-beams Subjected to Repeated Loads." Major Professors: M. J. Gutzwiller and R. H. Lee.

A total of 14 reinforced concrete T-beams and one beam of rectangular cross-section were tested to failure under repeated loading. The 13 inch deep beams were tested as restrained beams and were designed to be weaker in shear than flexure, with the critical section for shear, or shear span, being the length from the section of zero moment to the section of maximum negative moment. The only variables were the shear span-to-depth ratio and the magnitude of the repeated load. The shear span-to-depth ratio ranged from 2.88 to 5.03 and the rate of loading was 250 cycles per minute.

The object of this research was to compare the mode of failure of repeatedly loaded T-beams to the failure pattern of identical T-beams subjected to static loading. It was also intended to make a comparison with the results of fatigue tests performed on reinforced concrete beams of rectangular cross-section. These companion beams had been tested in two previous investigations at Purdue University.

Two of the test specimens, one T-beam and one rectangular, were cast with iron-constantan thermocouples attached to the reinforcing steel at, or near, the point of maximum negative moment. It was hoped to detect a significant increase of temperature due to repeated

stressing of the steel bars.

It was found that the repeatedly loaded T-beams failed in all cases, regardless of shear span, by fatigue of the longitudinal steel reinforcement if subjected to a sufficient number of loading cycles. This was in direct contrast to the shear type failures of the companion beams of both T-shaped and rectangular cross-section. The stress range was found to have the most influence on the fatigue life of the steel reinforcement while the shear span-to-depth ratio had almost no influence at all. The number of cycles to failure decreased with increasing stress range, and the fatigue limit was found to be approximately 40 ksi. Failure occurred at maximum repeated loads ranging from 52% to 76% of the ultimate static load.

The amount of temperature increase detected during testing was negligible.

INTRODUCTION

Reinforced concrete design practices in the United States have been in a continuous stage of evolution and refinement since the advent of reinforced concrete as a structural element. The resistance of reinforced concrete beams to combined shear and flexure has been both a problem and subject of intense study for almost as long, and the influence of shear has been of particular interest.

Almost a century of research has not been able to put together a thorough explanation of the shear mechanism of failure, but, instead, has continuously aroused more suspicion as to its intrinsic complexity. This is due to the fact that few investigators have included the same combination of variables, such as: size of beam, cross-section, load arrangement, concrete strength, web reinforcement, and longitudinal reinforcement. The nonhomogeneous nature of concrete itself introduces the element of chance, disrupting any order which might be expected from a large number of tests.

In addition to the multiplicity of parameters governing beam strength, consideration must also be given to the type of loading conditions to which the beam is subjected. In particular, the use of reinforced or prestressed concrete members in bridges and other elevated structures to carry large volumes of high speed traffic raises serious questions concerning the reliability and relevance of the results of static load tests, and demands large scale

investigations into the effects of dynamic loading. The recent use of new materials of higher static strength at higher working stresses may also result in a relative reduction in the resistance to repeated loads. A stage in development may therefore be reached where consideration of repeated stresses may form a more important aspect of design than at present.

The purpose of further research in this area is to provide some arbitrary assessment of the reduction in strength that might result from fatigue loading and to show whether there could be a change in the mechanism of failure forming the accepted basis of design.

Cracking Phenomena Under Static Loading

The relative effects of flexure and shear may be exemplified by a simply supported beam loaded symmetrically at two points. The distance from the support to the load is called the shear span. See Figure 1a. If such a beam is subjected to static loading, one of four principal failure modes is noticed. They are;

1. Fracture of the tensile reinforcing steel
2. Diagonal tension failure
3. Compressive failure of the concrete
4. Loss of bond between the concrete and reinforcing steel

The major variables which seem to have the most significant influence on the mode of failure are; compressive strength of the concrete, percentage of longitudinal reinforcement, and the ratio of moment to shear at the critical section. For simply supported beams, the last parameter mentioned may be represented as the ratio of the

shear span to the structural depth of the member. Many investigators have found the shear span-to-depth ratio to be a good indicator of the mode of failure, although no exact values for the ratios which divide one mode of failure from another can be agreed on.

If the loads are applied near the center of the specimen, as in Figure 1a, providing a large shear span-to-depth ratio, shear has little effect and failure will be controlled by flexure. Vertical flexure cracks will form from the tension surface of the beam at intervals along the span according to the magnitude of the bending moment when the limiting tensile strain in the concrete is reached (Figure 1b). Increased loading will lead to further extension of these cracks and either crushing of the concrete or yielding of the tensile reinforcement followed by crushing of the concrete. (Figure 1c).

Moving the loads out from the center of the member decreases the length of the shear span and increases the influence of shear on the failure mode. Figure 2a shows the same sample beam with a moderate shear span-to-depth ratio. After passing through the level of the longitudinal reinforcement, the flexural cracks in the shear span begin to incline toward the load due to the combination of tensile and shearing stresses. As the cracks approach the neutral axis, they become more inclined because of smaller flexural tensile stresses and eventually flatten out after passing through the neutral axis, as can be seen in Figure 2b. Diagonal tension cracks may also form in the shear span between the neutral axis and the longitudinal reinforcement independently of the inclined flexural cracks.

Until the formation of diagonal tension cracks, the behavior

FIGURE 1. TYPICAL FLEXURAL FAILURE

FIGURE 2. TYPICAL DIAGONAL TENSION FAILURE

of the beam is the same as that of beams failing in flexure. The formation of diagonal cracks, however, leads to a redistribution of internal stresses. Figure 2c is a free body diagram of a typical diagonal tension crack and it will be assumed for simplicity that it is the only crack existing in the member. C and V_1 are the compressive and shear forces, respectively, which act on the reduced concrete compression area. T is the tensile force in the reinforcement and V_2 is the transverse shear. By summing vertical forces, it can be seen that the external shear must now be resisted by the reduced compression zone and the longitudinal reinforcement.

Knowing the true contribution of the tension reinforcement to the resistance of shear, or "dowel action", is essential in understanding the diagonal tension mechanism of failure, but, as yet, this has not been achieved. It has generally been assumed, in beams without web reinforcement, that the division of shear force between the compression face and the tension reinforcement depends on the relative stiffness of these elements. The stiffness of the reinforcement is small compared to the stiffness of the compression zone and, thus, the portion of shear transmitted by the tension reinforcement may be neglected. (In beams with web reinforcement the division is more complex but the contribution of the longitudinal reinforcement is still believed to be small). However, there are an increasing number of investigators^{(3,12,14,21,25)*} who attribute greater significance to

* Superscripted numbers in parentheses refer to items in the bibliography.

dowel action. A few of these will be discussed later. If, indeed, the dowel force is substantial, then it can easily be seen how the redistribution of internal forces can lead to a diagonal tension type failure.

After the diagonal crack has penetrated a short distance into the compression zone, (point 1 in Figure 2d), the lower portion of the crack widens and causes the two parts of the beam to rotate about the apex of the crack. At the same time, the tension reinforcement in the crack becomes deformed so that the force in that part of the bar is no longer horizontal. The vertical shearing force, V_2 , aided by the skewed tension force, T , causes the formation of a horizontal crack along the tension steel, or backup cracking (point 2 in Figure 2d). The splitting of the concrete at the level of the steel destroys the bond progressively up to the support, changing the action from that of a beam to a two hinged tied arch. After the horizontal crack has opened, the diagonal tension crack extends at a decreased slope through the compression zone to the top surface of the beam (point 3 in Figure 2d).

In beams of small shear span-to-depth ratio the action is similar. Diagonal tension cracks form, redistribution of internal stresses takes place, and a new equilibrium of internal and external forces is reached. However, horizontal cracking along the longitudinal reinforcement caused by dowel action is prevented by strong vertical compression forces in the vicinity of the nearby support. Further increase of load is possible until the reduced concrete area above the diagonal crack is crushed. The member is then said to have failed by

shear-compression.

Thus, resistance to the dowel force and the associated phenomena of horizontal cracking seems to be the controlling factor in some diagonal tension type failures. D. N. Acharya and K. O. Kemp⁽³⁾ regarded the splitting at the level of the steel due to high shear forces on the longitudinal steel as a primary cause of failure for their specimens. They added⁽⁴⁾ that dowel action may not have significance for all beams but neither should it be totally neglected. D. Watstein and R. G. Mathey⁽²⁵⁾ made an evaluation of the vertical shear transferred by the reinforcement in beams with a shear span-to-depth ratio of approximately two. It ranged from .38 to .74 of the total shear at loads ranging from .42 to .46 of the maximum and decreased continuously thereafter. J. Taub and A. M. Neville⁽²¹⁾ recognized the existence of shear forces in the steel as the cause of horizontal splitting, and William J. Krefeld and Charles W. Thurston⁽¹⁴⁾ concluded that the splitting of the concrete along the tensile reinforcement from the lower end of the diagonal crack triggers the propagation of the upper end of the crack. The last two investigators mentioned also concluded that the shear resistance of the longitudinal reinforcement, acting in conjunction with the embedding concrete, depends on the size of bars, spacing, strength of concrete, and depth of cover.⁽¹⁵⁾

This last factor was verified to some extent in an earlier investigation by A. F. Al-Alusi⁽⁵⁾ which included a series of inverted T-beams and regular T-beams of approximately the same shear span. Although the inverted T-beams did not have substantially greater depth of cover than the regular T-beams, they did have the larger mass of

the concrete flange surrounding the tension reinforcement and resisting horizontal cracking. The test results show that after the formation of a diagonal crack in the inverted T-beams there was a drop of load, but a favorable redistribution of internal stresses enabled the beam to carry additional load. The ultimate shearing stress of the inverted T-beams proved to be higher than that of the regular T-beams, and there was a lack of horizontal splitting along the longitudinal reinforcement, although this was not offered as an explanation at the time.

In the present investigation, the negative moment regions of the test specimens resemble inverted T-beams. The results of these tests in light of horizontal cracking will be discussed later.

Diagonal tension failures may be prevented or delayed through the use of stirrups. The addition of web reinforcement in the shear span increases the resistance to horizontal cracking by tying together the portions of concrete above and below the longitudinal bars and thereby retarding the separation of these two segments, as well as, by delaying the opening of the diagonal crack.

For design purposes, however, web reinforcement is not necessary if the shear stress does not exceed its allowable value. The 1963 ACI Building Code⁽²⁾ redefined the nominal shear stress as merely the average shear stress, $v = V/bd$, which neglects the classical use of the moment ARM, $j d$.

In an attempt to produce members for which ultimate strength tends toward being governed by flexure rather than shear so that members will have a ductile character, ACI-ASCE Committee 426⁽¹⁾ developed the following expression for the ultimate shear stress in a

beam without web reinforcement;

$$v_c = 1.9\sqrt{f'_c} + 2500p\frac{Vd}{M} \leq 3.5\sqrt{f'_c}$$

For working stress design, a safety factor of approximately two is applied to this equation, and appropriate load factors are used for ultimate strength design. It can be seen that the expression contains the factors influencing the shear strength of reinforced concrete beams mentioned previously; concrete strength, percentage of longitudinal reinforcement, and moment to shear ratio.

Cracking Phenomena Under Repeated Loading

The reduced stresses for working stress design and the load factors for ultimate strength design, such as in the previous discussion of shear stress, are based primarily on the behavior and strength characteristics for static loading conditions. It is well known, however, that under repeated loading concrete members collapse at a lower load than under static loading. Repeated stressing causes the strength of the member to be reduced substantially and the member is said to have failed by fatigue.

Present design formulas give consideration to the effects of loads other than static through the use of impact factors which merely increase the live load by some percentage based solely on the length of the member. Experience has shown these values to be safe, but to an unknown degree. It would be wise to become more aware of their relative safety. The more efficient use of structural materials, such as higher permissible stresses or lower load factors, present an even greater need for caution. Despite a wide variety of tests conducted

in the area of fatigue, results on the actual reduction in strength are far from adequate and considerable doubt still exists as to the inherent factors of safety and the method of their evaluation.

In 1958, Nordby⁽¹⁷⁾ reviewed more than 100 of the most important investigations performed on fatigue of concrete up to that time and presented a summary of their findings. For the most part, repeated load tests of plain concrete in compression, flexure, and tension were inconclusive, but they did indicate typical behavior patterns. It was generally agreed that fatigue failures would not occur if the maximum value of the repeated load was less than 60 percent of the static ultimate load. Rates of loading between 70 and 440 cycles per minute had little effect on the fatigue strength, but the range of stress (maximum stress minus minimum stress) was significant; the most severe loading being one in which the mean static load is zero and the varying load causes alternate tension and compression. This is called completely reversed loading. The fatigue limit was also found to be proportional to the concrete strength and age; a higher strength and older concrete being able to resist a higher percentage of the static ultimate load.

Fatigue investigations of reinforced concrete members were equally inconclusive, mainly because they were highly diversified. Most investigators have reported four basic failure modes; fracture of the tensile reinforcement, diagonal tension, shear compression, and loss of bond. It should be noted that these are the same modes of failure exhibited by beams subjected to static loading conditions and, further, they are influenced by the same parameters; i.e., concrete

strength, percentage of tensile and web reinforcement, and moment-to-shear ratio. However, repeated loads introduce an additional set of variables which have such a decisive effect on the mode of failure that identical beams may fail differently depending on whether they are subjected to static or repeated loads. The variables of most concern are: maximum value of repeated load, range, and rate of loading.

Verna and Stelson⁽²³⁾ found that bond is most susceptible to fatigue damage, but only if the member is also statically weak in bond. Chang and Kesler⁽⁷⁾ concluded that bond failures are unlikely under repeated loading with deformed bars meeting ASTM specification A305. This type of failure is not to be confused with the loss of bond associated with diagonal tension failures. Assuming a reinforced concrete member to have sufficient bond strength, then either shear or diagonal tension is most likely to be the cause of failure. However, in almost every investigation, there have been reported a number of specimens whose ultimate fatigue life was controlled by the fatigue life of the steel reinforcement. Van Ornum,⁽¹⁷⁾ who was the first to extensively examine reinforced concrete beams in fatigue, noted that the beams failed through the development of a tension or diagonal tension crack, but, if the crack had penetrated far enough, then the beam failed by steel fatigue. In those beams which failed through the formation of diagonal tension cracks, it appeared that gradual progressive destruction of bond originating near the center of the specimen and moving toward the end caused failure. This may be related to the high dowel forces mentioned in the discussion of static shear

stresses which are aggravated by repeated stressing.

Chang and Kesler^(7,8) ran repeated load tests of concrete beams with various ratios of static shear-to-flexure strength. When the shear and flexure strength were approximately equal, more than half of the specimens failed by fatigue of the reinforcement, with the remaining beams failing equally in diagonal tension or shear compression. They related the type of failure to the magnitude of the repeated load; low loads resulting in flexure failure; high magnitude loads resulting in shear failure. In a later investigation, they found that, generally, a beam weak in shear will crack diagonally, with the crack progressing far enough into the compression zone so that the reduced area will not be able to resist the maximum value of the repeated load. Failure then occurs by destruction of the compression zone. Although the 39 specimens of this test were statically weaker in shear than in flexure, two failed by fatigue of the reinforcing steel.

Recent research projects at Purdue University^(11,19) have also involved reinforced concrete beams which were weaker in shear than in flexure. Results show a definite change of failure mode from static to dynamic loading conditions. Whereas the specimens failed by either shear compression or diagonal tension when subjected to static loads, fracture of the tensile reinforcement was found to be the cause of failure for more than half of the dynamically loaded beams, with the remainder failing by diagonal tension.

The unexpected appearance of flexure failures, and the disproportionate number of flexure failures occurring when they were expected, have led to more complete investigations of the fatigue

strength of reinforcing bars. The Portland Cement Association (PCA) and the American Association of State Highway Officials (AASHTO) have made the most significant contributions in this area to date.

Pfister and Hognestad,⁽¹⁸⁾ working for the PCA, first investigated a number of variables in order to determine their effect on the fatigue limit of reinforcing bars. The fatigue limit is normally expressed as the stress range which would cause failure at two million cycles of load repetition. They found it to be relatively insensitive to the magnitude of the minimum stress, bar yield strength, and beam cross section. It was also reported that the geometric details of bar deformations have strong influences on the fatigue characteristics of the bars. The stress range at the fatigue limit for one bar lot may be 35 percent less than for another lot with another deformation pattern depending on the local geometry where the transverse lugs meet the longitudinal ribs. This was also reported by Kokubu and Okamura⁽¹³⁾ in Japan, who observed the increased radius at the base of the lug results in an increased fatigue strength. They added that sudden changes in the slope of the lug from a point $1/3$ up the height of the lug had little effect on the fatigue strength.

Kenneth Burton⁽⁶⁾ continued the work at the PCA and found two other variables influencing the fatigue life of reinforcing bars. The range of stress had the strongest effect, decreasing fatigue life as the stress range increased. Fisher and Viest,⁽⁹⁾ in a test of bridge materials during the AASHTO Road Test, reported that the stress range was by far the most important independent variable. Burton also concluded that the orientation of the longitudinal ribs had a pronounced

effect on fatigue life of the bar if the transverse lugs merged into the rib. The fatigue life of bars with the ribs placed in a vertical plane, as shown in Figure 3a, was almost 40 percent less than that of bars tested in the same stress range with the ribs in a horizontal plane (Figure 3b), indicating that the stress concentrations at the point where the transverse lug merges with the longitudinal rib (point A in Figure 3a) may be higher than where the lug merges with the main body (point B in Figure 3b). The inverse of this was reported in a later investigation at the PCA by Hanson, Burton, and Hognestad.⁽¹⁰⁾ They concluded that the orientation of the longitudinal bar ribs in a reinforced concrete beam has little effect on fatigue strength when the lugs do not merge into the ribs, but, when the ribs are placed in a horizontal plane, the fatigue strength is not necessarily improved by terminating the transverse lugs before they reach the longitudinal ribs.

The type of deformed bars shown in the diagrams of Figure 3 were used in the AASHTO Road Test.⁽²²⁾ (The photograph was taken of a typical fractured bar from the present investigation). Inspection of the reinforcing bars after failure in the AASHTO test revealed that all cracks formed at, or passed through, the intersection of the two diagonal lugs with a longitudinal rib (point C in Figure 3b), supporting evidence that this junction is a point of fatigue weakness.

Although definite trends in the fatigue behavior of deformed reinforcing bars have been reported, few practical applications have been suggested. However, Hanson, Burton, and Hognestad did conclude that the radius at the base of the transverse lugs should be

FIGURE 3. LONGITUDINAL STEEL REINFORCEMENT

approximately equal to the height of the lug to obtain good fatigue characteristics. All of the investigators agreed that when the stress range in the deformed reinforcing steel exceeds 20,000 psi, fatigue failure becomes possible if the member is subjected to a large number of stress repetitions. Stress ranges of this magnitude may occur in some structural members, particularly if they are reinforced with high strength steel and designed by ultimate strength methods.

In summary, the ultimate load of a reinforced concrete member is reduced if the load is applied repeatedly, and the failure mode may be different than if the beam were subjected to static loads. The failure mode is determined mainly by the maximum value of the repeated load, the range and rate of loading. In many cases the fatigue life of a reinforced concrete beam is limited by the fatigue life of the tension reinforcement which, in turn, depends on the stress range in the steel. The stress range at the fatigue limit (two million cycles of loading), when reduced by a suitable margin of safety, can be used in design as an allowable value for stress due to repeated live load alone. The effects of all dead and live loads can then be accounted for safely by the usual methods of working stress design or ultimate strength design. The geometric details of the deformations also have a pronounced effect on the fatigue characteristics of reinforcing bars. The radii of curvature with which the transverse lugs merge into the main body of the bar are significant, as well as the manner in which transverse lugs meet longitudinal ribs.

PURPOSE OF STUDY

The objective of this study was to continue the research on the behavior of reinforced concrete beams subjected to repeated loads, with specific attention to T-beams. It was intended to determine the effect of shear span-to-depth ratio and magnitude of repeated load on the failure mode of the specimens. The results were then to be compared to those of static load tests of identical T-beams and repeated load tests of beams with similar rectangular cross section.

It was also proposed to evaluate any temperature gradient caused by repeated stressing of the longitudinal reinforcement.

TEST SPECIMENS AND PROCEDURES

The cross section of all test specimens was composed of a 6 inch by 13 inch rectangular web. The T-beams had an additional 3 inch thick flange with an overall width of 30 inches. The beams were simply supported at two points with an overhang to simulate the condition of an interior support of a continuous member. A load (P_1) was applied at the end of the cantilever portion and a second load (P_2) was applied between the supports. The load was distributed to these two points of application in the desired ratio by means of a steel wide flange section. The position and magnitude of P_1 and P_2 produced a maximum negative moment of 2 times the maximum positive moment.

Two high strength No. 6 steel reinforcing bars were used to resist the negative moment and two No. 5 bars to resist the positive moment. All longitudinal reinforcement extended the full length of the test specimen to eliminate the effect of bar cutoff on diagonal tension failure. Web reinforcement was provided only in the cantilever portion of the beam and outside of P_2 , forcing any shear failure to occur within the shear span "a", which extended from the point of zero moment to the point of maximum negative moment.

The length of the shear span and the magnitude of the repeated load were the only intended variables, although there was some variation of concrete strength.

The loads, and shear and moment diagrams are shown in Figure 4,

SERIES I P

SERIES II P

SERIES III P

SERIES IV

FIGURE 4. DETAILS OF SPECIMENS

and the details and properties of the test specimens can be found in Figure 5 and Table 1.

Materials

Concrete Mix

All concrete was made with Type I Portland cement. The concrete strength was intended to be maintained at 5000 psi, but varied from approximately 4000 to 6000 psi at 7 days. Tests were begun when the specimen had an age of at least 7 days, and failure occurred, or the test terminated, at a specimen age of approximately 14 days. Concrete strength at 14 days had an average increase of about 10 percent over 7 day strength. The proportions of the concrete mix by saturated-surface-dry weight were 1:2.06: 3.51 (cement-to-fine aggregate-to-coarse aggregate), with a water-cement ratio (w/c) of .506 by weight and a cement factor of 6.12 sacks/cubic yard.

Aggregates

The aggregates were purchased from Western Indiana Aggregates Corporation, Lafayette. The coarse aggregate was a natural gravel of 1 inch maximum size. After separating into two sizes, 48 percent of No. 4 to $\frac{1}{2}$ inch was combined with 52 percent $\frac{1}{2}$ inch to 1 inch by weight. Average properties of the fine and coarse aggregate are shown below.

	Sp. Gr. *	Absorption	Fineness Modulus
Fine	2.83	1.26%	2.87
Coarse	2.65	1.37%	--- 1" Max. Size

* Based on saturated surface dry weight.

SERIES II P, III P, IV P

SERIES IP

FIGURE 5. DETAILS OF TEST SPECIMENS

Table 1. Properties of Test Specimens

Beam	d	d'	a	a/d	Concrete Compressive Strength (psi)	Age (Days)	Split Tension (psi)	$E \times 10^6$ (psi)
I P-1	11.13"	2.31"	44"	3.95	5270	14	478	2.80
II P-1	"	"	32"	2.88	5900	7	535	4.52
II P-2	"	"	"	"	4150 5150	7 14	467 -	4.30 -
II P-3	"	"	"	"	5380 5830	7 14	511 -	3.91 -
II P-4	"	"	"	"	5800 6100	7 12	548 -	4.10 -
II P-5	"	"	"	"	5800 6400	7 14	540 -	3.93 -
III P-1	"	"	44"	3.95	4700 5520	7 14	485 -	4.27 -
III P-2	"	"	"	"	5350 5780	7 14	386 -	3.98 -
III P-3	"	"	"	"	5750 6210	7 14	340 -	4.16 -
III P-4	"	"	"	"	5450 5880	7 11	436 -	4.09 -
III P-5	"	"	"	"	5550 6150	7 14	483 -	3.90 -

Table 1 (Continued)

Beam	d	d'	a	a/d	Concrete Compressive Strength (psi)	Age (Days)	Split Tension (psi)	$E \times 10^6$ (psi)
III P-6	11.13"	2.31"	44	3.95	5550 6000	7 14	396 -	3.96 -
IV P-1	"	"	56	5.03	5620 6100	7 14	500 -	4.02 -
IV P-2	"	"	"	"	5550	7	435	4.05
IV P-3	"	"	"	"	6180	21	531	4.17

The percentage of longitudinal reinforcement, p , is 0.01318 for all specimens.

The modulus of elasticity for the concrete is the initial tangent modulus.

Gradation of Fine Aggregate

<u>Sieve Size</u>	<u>Percent Retained</u>
No. 4	1.4
8	16.6
16	36.8
30	48.0
50	87.8
100	96.6

Reinforcing Steel

The longitudinal reinforcing steel was a high strength steel, A431. The No. 6 bars had a higher yield stress and ultimate strength than the No. 5 bars. Stress-strain curves for both size bars are shown in Figures 44 and 45, and their properties can be found in Table 2.

Table 2. Properties of Longitudinal Reinforcing Steel

	No. 5	No. 6
Yield Stress	72.1 ksi	96.4 ksi
Ultimate Strength	113.3 ksi	158.0 ksi
Modulus of Elasticity	29.6×10^6 psi	29.1×10^6 psi
Elongation in 8"	-	8.5%

The No. 2 plain bars used for stirrups in the overhang and in the 18 inch interior span were of hard grade steel with an average yield stress of 52,000 psi.

Fabrication and Curing

All specimens were cast in $3/4$ inch plywood forms coated with a polyester resin to prevent deterioration from repeated wetting and drying. The forms are shown partially assembled in Figure 6. Tie rods were inserted through the stem of the specimen and through the base of the form to hold the form together during casting. Those tie rods which passed through the concrete were covered with rubber tubing to facilitate their removal after the concrete had hardened.

The longitudinal reinforcement was wired into a rigid cage by means of the stirrups in the overhang and outside load P_2 . Steel chairs were used to support the cage in the formwork and provided the correct vertical position. This combination of stirrups and chairs allowed a concrete cover to the longitudinal reinforcement of $1\frac{1}{2}$ inches at the top and sides, and 2 inches on the bottom, with a $1\frac{1}{2}$ inch clear spacing between bars.

The concrete was mixed in a stationary rotating drum mixer with a maximum capacity of eleven cubic feet. The size of the specimens required the use of more than one batch of concrete per beam. Therefore, the quantities of materials for each batch were taken from the same stock pile and weighed prior to mixing to insure uniformity. The concrete was vibrated internally as it was placed to prevent honeycombing. Standard test cylinders were also made from the same batch.

Several hours after casting, the top of the specimen and the test cylinders were covered with moist burlap and a sheet of plastic. The forms were removed after six days in order to prepare the specimen for testing.

FIGURE 6. FORMS PRIOR TO CASTING

Instrumentation and Testing Procedures

After the forms were removed, the specimen was allowed to dry for a few hours. The sides of the web and the under-sides of the flange were then painted white with a thin mixture of latex paint. A grid system was drawn over this to allow for easier reproduction of the crack patterns after failure.

An area $3\frac{1}{2}$ inches from the support in the negative moment region was prepared to receive concrete strain gages. The area was sanded, cleaned with acetone, and covered with a layer of Duco-cement. This was allowed to dry for one day before the paperback SR-4 wire gages (Baldwin-Lima-Hamilton Type A-1-S6) were cemented to the surface.

Steel strain gages (Budd Metalfilm C6-111B) had previously been attached to the top reinforcing bars at the point of maximum negative moment. These gages were affixed to the bars using the procedure recommended by William Bean. Eastman 910 was used as an adhesive. The gage assembly was then waterproofed with an epoxy compound (Epoxylite #222). The reinforcement was wired into a cage with its longitudinal ribs lying in a horizontal plane. In order to make room for the strain gage, one set of criss-cross lugs was removed from the top surface of the bar with a disc grinder and the area smoothed with a hand file. For some specimens, gages were placed on both the top and bottom surfaces at the same position. In these cases, lugs were removed from both the top and bottom of the reinforcing bar.

All steel and concrete gages were connected to a switching and balancing unit with three-wire leads. The strain signals were passed from this unit to a Budd P-350 digital read-out strain indicator, from

which the static component of the strain was taken. The dynamic component was evaluated with a calibrated oscilloscope.

Temperature measurement on several early specimens was made with iron-constantan thermocouples, #24 gauge wire, attached to the longitudinal reinforcement at or near the point of maximum negative moment. At first, the thermocouple wires were spot welded to the bars, but this was found not to produce a secure connection. A better connection was obtained by soldering them to a terminal strip which had been glued to the steel surface. In both cases, the wires were protected with the same waterproofing epoxy compound as was used for the steel strain gages. A Leeds and Northrup potentiometer was used in conjunction with the thermocouple wire.

Six of the nine concrete test cylinders were tested just before the beam specimen was loaded. Three were used to determine the compressive strength and the modulus of elasticity, and three others were tested in split tension. The remaining cylinders were tested in compression upon failure of the beam specimen.

The average compressive strength of the cylinders was used to predict the ultimate failure load of the test specimen. The maximum value of the repeated load was taken as some percentage of this ultimate load. In a previous investigation at Purdue University, dealing with repeatedly loaded beams of rectangular cross-section,⁽¹⁹⁾ the predicted failure load was based on an average of the expressions of Moody and Morrow, that of Moody being unconservative, and that of Morrow being conservative. However, static load tests of T-beams,⁽²⁶⁾ identical to those of the present investigation, demonstrated that

the expression of Moody very closely approximated the ultimate failure load, while that of Morrow was highly conservative. Therefore, the maximum value of the repeated load is expressed as a percentage of the failure load using only the equation of Moody. The equations for ultimate shear-moment as derived by both Moody and Morrow, as well as sample calculations, are found in Appendix B. A minimum load of four kips was applied to the specimen at all times to prevent impact. The cycling of maximum and minimum load was delivered to the specimen at a rate of 250 repetitions per minute by means of an Amsler Hydraulic Pulsator. The pulsator produced a sinusoidally varying oil pressure which drove an overhead hydraulic jack. The test setup is shown in Figures 7 and 8.

The specimen was loaded statically to the maximum value for the first cycle. The load was then reduced to the minimum value and the repeated load applied. The range of repeated load was maintained at one level throughout the test until failure occurred. A seismograph was attached to the specimen which automatically shut off the pulsator at the instant of failure. If, after approximately three million cycles, the specimen had not failed, the repeated load was terminated and the specimen loaded statically to failure.

FIGURE 7. TEST SETUP

SERIES	U	V
I	49.2"	40.8"
II	42.2"	29.8"
III	49.2"	40.8"
IV	56.4"	51.6"

FIGURE 8. DETAILS OF TEST SETUP

TEST RESULTS

The test results for all specimens have been summarized in Table 3. The specimens have been grouped in the following sections according to length of shear span, or series, and shape of cross-section. Each section includes a description of individual tests as they occurred, load versus strain curves, and crack pattern diagrams. Photographs of the specimens after failure and close-ups of the steel fractures are also found in this section.

Loads reported are total applied loads, which do not include the dead weight of the specimen nor the weight of the loading apparatus. Complete strain and temperature data may be found in Appendix C, which also includes the position of the gages and thermocouple wires.

Series I P

Series I P consisted of only one beam. The primary purpose of this series was to evaluate any temperature rise in the longitudinal reinforcement during the application of the repeated load. The specimen was identical to those of series III BF in Rogers report⁽¹⁹⁾ which had failed by fracture of the steel reinforcement. The shear span-to-depth ratio was 3.96.

Beam I P-1 (61% Ultimate)

The first flexural crack appeared at a load of 10 kips. After one cycle of loading six flexural cracks had formed in the negative

Table 3. Summary of Test Results

Beam	Applied Load P Max. P Min. (kips)	Percent of P_f^{**}	Fatigue Life N (Cycles)	Ultimate Load P_u^{**} (kips)	Mode of Failure ^{##}	Remarks
I P-1	28.5	3	611,000 t [#]	55	T/D.T.	
II P-1	31	4	3,086,000 t	65	T/S.C.	
II P-2	27	4	4,930,000 t	60	T/S.C.	
II P-3	37	4	2,894,000	37	F/F.P.	
II P-4	43	4	1,123,000	43	F/F.P.	
II P-5	38	4	4,292,000	38	F/F.R.	
III P-1	26	4	3,111,000	26	F/F.R.	
III P-2	32	4	3,369,000 t	74	T/S.C.	
III P-3	39	4	2,540,000	39	F/F.R.	
III P-4	44	4	1,368,000	44	F/F.R.	#5 Bars Failed
III P-5	33	4	2,084,000	33	F/F.R.	
III P-6	41	4	941,000	41	F/F.R.	

Table 3 (Continued)

Beam	Applied Load P Max. P Min. (kips)	Percent of P _f *	Fatigue Life N (Cycles)	Ultimate Load P _u ** (kips)	Mode of Failure##	Remarks
IV P-1	31	4	60	3,490,000	31	F/F.P.
IV P-2	35.5	4	68	896,000	35.5	F/F.R.
IV P-3	33.5	4	62	3,300,000	33.5	F/F.P. #5 Bars Failed

* Predicted ultimate load.

** Total applied load. (Does not include dead weight of specimen and loading apparatus.)

t signifies tests in which repeated load was terminated before the occurrence of fatigue failure.

T - Tested statically to failure after termination of repeated load.

F - Fatigue failure.

S.C. - Shear compression failure.

F.R. - Fatigue of reinforcement.

D.T. - Diagonal tension failure.

II P-1

II P-2

FIGURE 9. BEAMS AFTER TEST - II P-1, II P-2

FIGURE 10. BEAM AFTER TEST — II P-3

FIGURE II. BEAM AFTER TEST — II P-4

FIGURE 12. BEAM AFTER TEST — II P-5

FIGURE 13. BEAM AFTER TEST — III P-I

III P-2

III P-4

FIGURE 14. BEAMS AFTER TEST — III P-2, III P-4

FIGURE 15. BEAM AFTER TEST — III P-3

FIGURE 16. BEAM AFTER TEST — III P-5

FIGURE 17. BEAM AFTER TEST — III P-6

FIGURE 18. BEAM AFTER TEST — IV P-1

IV P-2

IV P-3

FIGURE 19 BEAMS AFTER TEST — IV P-2, IV P-3

moment area, the furthest from the support being 8 inches out on the overhang and 20 inches into the shear span. All of the cracks were inclined toward the support. The crack furthest out on the overhang extended to within 3 inches of the compression face. Two cracks also formed in the positive moment area and stopped 6 inches from the compression face.

The repeated load was applied for 160,000 cycles before the pulsator shut itself off. Up to this point the temperature of the longitudinal reinforcement had remained nearly constant. The pulsator was turned on and allowed to run until 408,000 cycles. A slight variation in temperature was detected but was found only to follow the temperature of the room. The maximum value of the repeated load was therefore raised to 70% of the ultimate load and the specimen was stressed for an additional 200,000 cycles. Again, no temperature increase was observed and the dynamic load was removed. The load was then increased statically to produce failure. At a load of 55 kips a diagonal tension failure occurred with the sudden opening of a diagonal crack running from the support to a point 28 inches inside the shear span and then splitting along the longitudinal reinforcement all the way to the interior load point.

SOUTH SIDE

NORTH SIDE

Cracks during first cycle
 Cracks during repeated load
 Cracks opening wide at failure

FIGURE 20. BEAM IP-1

Series II P

The specimens of this series had a shear span-to-depth ratio of 2.88. They are companion to the beams of series II BF in Rogers' (19) report and identical to beam II T-1 in Wehr's (26) report. The #6 reinforcing bars used in beams II P-1, II P-2 and II P-3 had a yield stress and ultimate strength of 72,000 psi and 113,000 psi respectively, while the #6 bars used in II P-4, and II P-5 and all the beams of the other series had a yield stress and ultimate strength of 96,000 psi and 158,000 psi respectively.

Beam II P-1 (52% Ultimate)

No flexural cracks appeared until a load of 30 kips. At this load a crack opened in the negative moment area approximately 3 inches in the shear span and extended from the top of the flange to within 4 inches of the compression face. At the same time, a flexural crack formed in the web at the section of maximum positive moment. The load was increased to 31 kips with no additional cracking, after which the repeated load was applied. A diagonal crack formed in the overhang portion 8 inches from the support by 96,000 cycles and another one 10 inches inside the shear span at 325,000 cycles. Both cracks began at the chamfer between the web and the flange, and ran through the web toward the support in one direction and extended partially into the flange in the other. This progression was typical of all diagonal cracks to follow. Splitting along the chamfer also began at this time and connected the three cracks. The original crack had the largest opening and now extended to within 2 inches of the compression surface. There was no indication of imminent failure after 3,000,000 cycles. Therefore, the repeated load was removed and the static load increased. At

a load of 65 kips the diagonal crack, which had previously been formed inside the shear span, extended nearly the full length of the shear span splitting the beam from the support to the chamfer, then along the chamfer and up through the flange at the interior load point.

Beam II P-2 (53% Ultimate)

This specimen was identical to Beam II P-1 except for its concrete strength being 2000 psi less. The crack pattern was also almost identical. The first flexural crack formed at the same location at a load of 24 kips and extended to within 4 inches of the compression face as the load was increased to 27 kips. The diagonal crack in the overhang also opened at this load but did not penetrate the flange. After approximately 1,000,000 cycles, a diagonal crack appeared in the shear span 8 inches from the support and extended from slightly through the flange to within 3 inches of the compression face. The other diagonal crack now also entered the flange. The test was allowed to run for nearly 5,000,000 cycles with no appreciable change in the existing cracks. The specimen was then loaded statically and failed by shear-compression at a load of 60 kips. The failure crack began 4 inches from the support, ran diagonally the full length of the shear span, along the chamfer and through the flange at the interior load point.

At the time of failure, the concrete compressive strength had increased by 1000 psi. The ultimate load of the specimen for this concrete strength predicted by the equation of Moody was 56.5 kips, while the actual specimen failed at 60 kips. Therefore, for T-beams, it seems justifiable to express the maximum value of the repeated load as a percentage of the ultimate load predicted by Moody's equation only, as discussed previously.

Beam II P-3 (64% Ultimate)

After the first cycle of loading, a flexural crack had opened 4 inches from the support inside the shear span and extended to within 3 inches of the compression face. The repeated load produced a diagonal tension crack in the overhang at 25,000 cycles. By 1,500,000 cycles, a small diagonal crack had formed 22 inches in the shear span on the south side and 12 inches in the span on the north side. Both cracks were only 3 or 4 inches long and terminated in the chamfer. The original flexural crack had penetrated to within 2 inches of the compression face and remained there until failure of the longitudinal reinforcement at 2,900,000 cycles.

The concrete was stripped back to reveal the fractured bars. Both bars had failed at, or very close to, the intersection of two lugs, and one to two inches inside the shear span. It was impossible to determine which bar had fractured first.

Beam II P-4 (73% Ultimate)

The first flexural crack opened at a load of 29 kips, 2 inches inside the shear span, and ran vertically to within 4 inches of the compression face. After the first cycle, at 43 kips, this crack had reached a point $2\frac{1}{2}$ inches from the bottom and a diagonal crack had formed at the chamfer 6 inches inside the shear span. 50,000 cycles of loading produced a crack in the overhang 4 or 5 inches from the support. After 1,123,000 cycles, at which time failure of the longitudinal reinforcement occurred, this crack opened wide in the flange on the south side of the specimen. The original flexural crack also opened wide at failure. The reinforcing bars failed at a section 2 or

3 inches into the overhang. The crack in the north side bar began at the intersection of two lugs at the top surface, while in the south side bar it began at the intersection of the lugs and the longitudinal rib. Strain readings indicate that the north side bar had failed somewhere between 870,000 and 1,068,000 cycles, after which the entire load was taken by the south side bar until its failure.

Beam II P-5 (64% Ultimate)

A flexural crack formed at a load of 30 kips directly over the support and extended to within 4 inches of the compression face. At the end of the first cycle, (maximum load 38 kips), it was three inches from the compression face. The repeated load was then applied and 8,000 cycles caused two diagonal cracks to open in the shear span on the north side of the beam. The shorter one was 4 inches from the support and the longer one, which stopped 4 inches above the bottom of the specimen, was 10 inches from the support. The latter crack also opened on the south side of the specimen at the same time, along with an extension of the original flexural crack 2 inches from the support into the overhang.

After 4,000,000 cycles of loading, there was very little change in the crack pattern. Steel and concrete strains also remained constant and low. At 4,105,000 cycles the north side #6 reinforcing bar failed very close to the section of maximum moment. Inspection of the bar after failure revealed that the crack had originated at a point where the deformation lugs merged with the longitudinal rib. The south side bar later failed at 4,292,000 cycles.

FIGURE 21. PICTORIAL REPRESENTATION OF SPECIMEN ON CRACK PATTERN SHEET

FIGURE 22. LOAD vs. STEEL STRAIN - SERIES II P - FIRST CYCLE

FIGURE 22. CONTINUED

FIGURE 23. LOAD vs. CONCRETE STRAIN — SERIES II P — FIRST CYCLE

FIGURE 25. BEAM II P-2

FIGURE 26. BEAM II P-3

FIGURE 27. BEAM IP-4

FIGURE 28. BEAM IP-5

Series III P

The specimens of this series had a shear span-to-depth ratio of 3.95. They are similar to the beams of series III BF in Rogers' (19) report and to beam III T-1 in Wahr's (26) report.

Beam III P-1 (52% Ultimate)

A load of 25 kips produced a flexural crack directly over the support extending to within 4 inches of the compression face. A similar crack opened under the interior load and terminated in the chamfer. Neither crack lengthened during the remainder of the first cycle. By 300,000 cycles the crack in the negative moment region had penetrated to within 2 inches of the compression face. A small diagonal tension crack opened on both sides of the beam at 400,000 cycles. It appeared in the shear span 8 inches from the support on the south side and 6 inches from the support on the north. The north side crack lengthened slightly after 3,000,000 cycles while the crack on the south side remained unchanged.

During the early stages of repeated loading, strain gages attached to the longitudinal reinforcement showed both bars to have nearly equal static components of strain, but the south side bar to have a much higher dynamic component. Both gages were lost after 771,000 cycles. At 3,005,000 cycles, concrete strains slightly decreased on the south side of the specimen and sharply increased on the north side indicating that the south side bar had fractured. At 3,111,000 cycles, the north side bar also failed. Both bars failed at sections in the overhang; the north side bar 1 inch from the support and the south side bar 2 inches from the support.

Beam III P-2 (60% Ultimate)

As in the previous specimen, the flexural crack formed directly over the support. This occurred at 28 kips and after the first cycle had been completed, total load 32 kips, the crack was 4 inches from the bottom surface. After 10,000 cycles, diagonal tension cracks had opened at points 6 inches on either side of the support and the chamfer had split between them. Another diagonal crack formed 12 inches inside the shear span at 346,000 cycles and penetrated the flange by 681,000 cycles. At 1,465,000 cycles, the other two diagonal cracks were halfway through the flange and the flexural crack had extended to within 2 inches of the compression face. 3,369,000 cycles of loading produced no additional cracking. Therefore, the repeated load was removed in order to fail the specimen statically. The static load was increased to 55 kips before further cracking was observed in the negative moment area. At a load of 60 kips, concrete strain gages indicated a compressive stress of approximately 4000 psi at positions $\frac{1}{2}$ inch and 1 inch above the bottom surface and increasing rapidly. Concrete gages at all locations were lost simultaneously after that reading. A shear-compression failure occurred at a load of 74 kips. The crack pattern at failure was almost identical to that of beam III T-1 in Harvey's report. Prior to failure, the diagonal cracks inside the shear span had not penetrated the neutral axis and gage readings indicated high compressive stresses at least as far as 2 inches above the bottom surface. At failure, the web spalled badly along a diagonal line from the support to a point 12 inches inside the shear span. The chamfer then split open from that point to the

interior load, with the flange cracking at three other points in between.

Beam III P-3 (71% Ultimate)

At a load of 30 kips, the flange cracked 4 inches inside the shear span. After passing through the flange, the crack inclined toward the support and stopped $2\frac{1}{2}$ inches from the compression face on the south side and 6 inches from the face on the north side. After the first cycle, with a total load of 39 kips, the south side crack remained unchanged while the north side crack extended to within $3\frac{1}{2}$ inches of the support. A diagonal tension crack also formed 9 inches from the support in the shear span at the chamfer and extended 3 inches into both the flange and the web. 5,000 repetitions of load caused a diagonal crack to form 6 inches out on the overhang. The length of all cracks increased slightly with additional loading. At 2,449,000 cycles, the diagonal crack in the overhang had split the flange completely and opened wide on the south side due to failure of the longitudinal reinforcement on that side. The bar was later found to have cracked at a section 2 inches from the support in the overhang. The crack originated at a point on the bar where the lugs merged with the longitudinal rib.

Beam III P-4 (82% Ultimate)

As in the previous specimen, the first flexural crack formed at a load of 30 kips. It was located 4 inches in the overhang, inclined toward the support after passing through the chamfer and stopped 3 inches from the compression face. A diagonal tension crack opened at a load of 35 kips at a point 6 inches from the support in

the shear span. By 45,000 cycles of loading this crack had split the flange completely and extended to within 2 inches of the bottom surface. A parallel crack formed 10 inches into the shear span, at the same time, as well as a diagonal crack 10 inches out on the overhang on the north side. This last crack opened on the south side at the same position with 659,000 cycles.

Steel strain gages indicated high stress, increasing in both #6 reinforcing bars until the gages were lost sometime after 659,000 cycles. However, at 1,368,000 cycles, there was a fatigue failure of the #5 reinforcing bars placed at the bottom of the web to resist the positive moment. Repeated loading had caused the flexural crack in the positive moment area to penetrate deeply into the compression flange. At 200,000 cycles it was less than $\frac{1}{2}$ inch from the extreme fiber on the south side. This excessive penetration may have been the cause of failure even though the positive moment was only half of the negative moment.

Beam III P-5 (61% Ultimate)

At a load of 26 kips, the tension flange cracked 2 inches from the support in the shear span. The crack extended to within 4 inches of the compression face and did not penetrate any deeper with additional static loading. At the end of the first cycle, 33 kips, a short diagonal crack had opened 8 inches in the shear span at the chamfer. By 5,000 cycles this crack had lengthened in both directions and a diagonal crack formed 6 inches from the support in the overhang. After 1,000,000 cycles, the cracks remained nearly the same except for the original flexural crack which was now only 2 inches from the bottom

surface. The north side #6 reinforcing bar failed at 2,070,000 cycles and the south side bar followed at 2,084,000 cycles. The section of failure was 2 inches inside the shear span which was the location of the flexural crack. Thermocouples attached to the bars very close to the point of fracture gave no indication of temperature increase.

Beam III P-6 (76% Ultimate)

A flexural crack formed at a load of 25 kips. The crack appeared in the flange directly over the support on the south side and 6 inches from the support in the shear span on the north side. When the crack reached the chamfer it was 2 inches in the shear span on both sides. With a load of 40 kips it had extended to within 3 inches of the compression face and two diagonal tension cracks had formed. One was in the shear span 10 inches from the support and the other was 5 inches out on the overhang. By 5000 repetitions of loading all cracks had lengthened slightly and an additional diagonal crack opened in the shear span 15 inches from the support. After 100,000 cycles, extensive splitting along the chamfer was noticed and a small diagonal crack opened 10 inches in the overhang. The flexural crack on the southside and a diagonal tension crack on the north side had penetrated to within $1\frac{1}{2}$ inches of the compression face by this time.

Sometime after 300,000 cycles, the indicated strain on the top surface of the south side #6 reinforcing bar at the location of the strain gage began to decrease in both its static and dynamic components. The strain on the bottom surface, and the strains on both surfaces of the north side bar increased slowly but continuously. A fatigue failure of the south side bar occurred at 941,000 cycles.

FIGURE 29. LOAD vs. STEEL STRAIN - SERIES III P - FIRST CYCLE

FIGURE 29. CONTINUED

FIGURE 30 CONTINUED

FIGURE 33. BEAM III P-3

NORTH SIDE

SOUTH SIDE

FIGURE 34 BEAM III P-4

FIGURE 35. BEAM III P-5

FIGURE 36 BEAM III P-6

Series IV P

The specimens of this series had a shear span-to-depth ratio of 5.03. They are companion to the beams of series IV T in Wehr's⁽²⁶⁾ report.

Beam IV P-1 (60% Ultimate)

The first cycle of loading produced the same crack pattern as in the shorter beams. A flexural crack formed over the support at a load of 25 kips, stopping 4 to 5 inches above the compression face. At 30 kips a diagonal crack opened in the shear span 10 inches from the support and lengthened slightly when the first cycle was completed at 31 kips. By 58,000 cycles, two diagonal cracks had opened in the overhang; 4 inches and 8 inches from the support. After 1,839,000 cycles, there was little change in the crack pattern but steel strains had increased greatly. A strain gage attached to the top surface of the north side bar indicated a stress of approximately 90,000 psi $\pm$ 15,000 psi before being lost. The gage attached to the bottom surface at the same position remained intact, however, and indicated a stress of only 20,000 psi $\pm$ 11,250 psi for almost 3,000,000 additional cycles of loading. Concrete strains also increased steadily during this time, reaching approximately 4200 psi $\pm$ 760 psi after 3,391,000 cycles.

Very short diagonal cracks began to form over almost the entire length of the shear span in the late stages of loading. They were located at approximately 6 inch intervals but extended into the web only two or three inches from the chamfer.

At 3,490,000 cycles there was a fatigue failure of the south side #6 reinforcing bar at the point of maximum moment where the

strain gages had been attached. Failure occurred at the intersection of the deformation lugs and the longitudinal rib. Although the top and bottom surfaces of the bar had been smoothed to receive the strain gages, that part of the lug which merged with the rib had not been removed.

Beam IV P-2 (68% Ultimate)

The tension flange cracked directly over the support at a load of 24 kips. At 30 kips the crack had reached a point 3 inches above the compression face, and a diagonal crack began to form 10 inches in the shear span. With the application of 35 kips load, the diagonal crack lengthened considerably and another crack opened 5 inches from the support in the overhang. The dynamic load was applied but had little effect on the crack pattern. As in the previous specimen, steel strains and related stresses increased rapidly. The static component of stress on the top surface of the north side #6 bar had reached approximately 106,000 psi after only 45,000 cycles. The dynamic component of the same stress was 27,000 psi. Again, the stress on the bottom surface remained at approximately 20,000 psi $\pm$ 10,500 psi, only one-fifth of the top surface stress.

At 822,000 cycles, the south side bar failed at a point slightly in the cantilever portion as did the north side bar at 896,000 cycles. The concrete strains were low when compared to those of the previous specimen due to the far smaller number of cycles required to produce failure of the longitudinal reinforcement.

Beam IV P-3 (62% Ultimate)

A flexural crack formed at a load of 26 kips and extended to within 4 inches of the compression face. After the first cycle had been completed, total load 33.5 kips, a diagonal tension crack opened 12 inches from the support in the shear span. This crack lengthened with 6,000 cycles of load and the flexural crack had reached a point 2 inches above the support. A diagonal crack also opened 6 inches out on the overhang and another one 18 inches inside the shear span. No significant additional cracking was noticed until 1,873,000 cycles. As in beam IV P-1, very short cracks had formed throughout the entire length of the shear span. They were spaced 4 to 6 inches apart and all terminated in the chamfer.

Contrary to the results of the two previous specimens, the steel strains remained constant for the duration of the test. The strains on the bottom surfaces of the longitudinal reinforcement were not much lower than those on the top surfaces, the ratio being approximately $4/5$, indicating much less local bending than in beams IV P-1 and IV P-2. At 3,300,000 cycles there was a fatigue failure of both #5 reinforcing bars in the positive moment area.

FIGURE 37. LOAD vs. STEEL STRAIN — SERIES IV P — FIRST CYCLE

FIGURE 39. BEAM IX P-1

FIGURE 40. BEAM IV P-2

FIGURE 41. BEAM IV P-3

DISCUSSION OF TEST RESULTS

Temperature

Limited time was spent on the investigation of heat generated by repeated stressing of the longitudinal steel reinforcement. Two specimens were equipped with iron-constantan thermocouples attached to the steel bars and in neither case was there a significant temperature increase. A net increase of 5 or 6 degrees Fahrenheit was detected over a period of more than 1,000,000 cycles of loading and nearly all of this was due to variation in the room temperature. Through frequent monitoring of the thermocouples, it was found that the specimens developed more heat during the curing period (heat of hydration) than during the application of the repeated load.

Modes of Failure

Given a sufficient number of loading cycles, the restrained reinforced concrete T-beams failed by fatigue of the longitudinal steel reinforcement regardless of the shear span-to-depth ratio or the magnitude of the repeated load.

Beams II P-1, II P-2 and III P-2 were able to withstand at least 3,000,000 repetitions of loading and were therefore tested statically to failure after the repeated load was terminated. These specimens exhibited shear-compression type failures at loads comparable to the predicted ultimate loads based on the expression of Moody and

using the concrete strength at the time of failure. It can be assumed that, if the repeated load had been continued, failure would have been caused by fatigue of the longitudinal reinforcement since there were too few diagonal cracks in the shear span to indicate imminent diagonal tension failure and the compressive stresses were not high enough at that time to cause shear-compression failures. Beams III P-4 and IV P-3 failed by fatigue of the steel reinforcement in the positive moment area even though the positive moment for all specimens was only one-half of the negative moment. This may have been caused by the excessive penetration of the flexural crack through the compression flange at the point of maximum positive moment and, again, if this had not occurred, it is presumed that fatigue of the longitudinal reinforcement in the negative moment area would have been the mode of failure.

The nine remaining specimens, with at least two in each series, had steel fatigue failures in the negative moment region after 896,000 to 4,290,000 cycles of loading. The maximum value of the repeated load at failure ranged from 52% to 76% of the static ultimate load. Identical T-beams subjected to static loading conditions, as found in the report of Wehr,⁽²⁶⁾ failed by either shear-compression or diagonal tension for all shear span lengths. Similar reinforced concrete beams of rectangular cross-section subjected to repeated loading by Rogers⁽¹⁹⁾ displayed diagonal tension failures when the shear span-to-depth ratio was small, and steel fatigue failures for larger shear span-to-depth ratios.

Thus, there is a definite change in the mode of failure when a

reinforced concrete beam without stirrups is exposed to cyclical loading rather than static loading. The presence of the tension flange in the restrained T-beams also has an effect on the failure mode under repeated loading. Under the above conditions, the fatigue life of the specimen is limited by the fatigue life of the longitudinal reinforcement.

Factors Affecting Fatigue Life of Longitudinal Reinforcement

The fact that there were relatively few flexural and diagonal tension cracks contributed greatly to reducing the fatigue limit of the steel reinforcing bars. A typical T-beam during testing, even after numerous loading cycles, contained only one major flexural crack close to the point of maximum negative moment, and, at most, only two diagonal tension cracks within the shear span. These diagonal cracks appeared no more than sixteen inches from the support location and were usually extensions of smaller flexural cracks. Some diagonal cracks formed independently in the shear span but were prevented from penetrating as far as the level of the tensile steel reinforcement by the large concrete tension flange and low magnitude of load. Under static loading conditions, additional diagonal cracks would have formed further out in the shear span as the load approached its ultimate value. Under repeated loading, however, the maximum value of load was not high enough to produce these cracks nor did a large number of repetitions have that effect. Thus, very high steel strains were localized at the section of the flexural crack. Steel strains were noticed to jump suddenly upon the formation of the flexural crack and to reach higher values than in beams of rectangular cross-section subjected to

comparable loads but having many more cracks. This was especially evident in series III P which is similar to series III BF in Rogers,⁽¹⁹⁾ report. It was also observed during testing that the width of the flexural crack at the top of the flange was approximately one-tenth of an inch while the diagonal cracks hardly opened.

The cracking of the flange at only one location also caused bending to be concentrated at that point. In contrast to the curvature of beams cracked at many places, the specimens and, likewise, the steel reinforcement were composed of two linear sections rotated about the support and resembled broken pencils. Strain gages attached to both top and bottom surfaces of the longitudinal reinforcement indicated a higher ratio of top-to-bottom stress than can be explained by the difference in distance from the compression face. This was particularly evident in series IV P. Beams IVP-1 and IV P-2 developed ratios of approximately 4:3 under static loading, and as much as 4:1 and 5:1 under repeated loading while the ratio of the distances from the compression face to the respective surfaces is only approximately 12:11. (This was also reported in an investigation by Pfister and Hognestad⁽¹⁸⁾ in which the strain on the top surface was consistently 10 to 20 percent higher than that on the bottom surface.) Beam IV P-3 was stressed at a much higher level than IV P-1 but was able to withstand nearly as many repetitions of loading (probably more since the test was terminated by fracture of the #5 bars) due to the fact that the ratio of the stresses on the top surface of the bars to those on the bottom was only 20:19 under static loading, and 5:4 after more than three million load cycles. Thus, bending stresses were superposed on the

already high tensile stresses.

As expected, the longitudinal reinforcement fractured at either of two places; one passing through the intersection of two diagonal lugs at the top surface of the bar, and the other passing through the point where two lugs merge with the longitudinal rib. These have previously been noted as points of high stress concentration (B and C in Figure 3b). All specimens were cast with bars positioned as shown in Figure 3b. Inspection of bars whose failure planes passed through the intersection of two lugs with the longitudinal rib revealed that the cracks had originated at point C rather than at the top surface which is further from the compression face. This is a strong indication that a poor deformation pattern, perhaps caused by the rolling process itself, could decrease the fatigue limit of longitudinal steel reinforcement. That is, the stress range required to produce failure at two million cycles of loading could be less than for a bar whose deformations merged more smoothly.

The stress range was found to have the most influence on the fatigue life of the steel reinforcement and the shear span-to-depth ratio to have little effect, if any. Figure 42 is a plot of the stress range versus the number of cycles to failure (plotted on logarithm paper) for each series. The stress range is defined as the maximum stress minus the minimum stress as calculated from $f_s = M/A_sjd$, assuming $j = 7/8$. The number of cycles to failure is seen to decrease with increasing stress range and the fatigue limit for all shear span lengths is 40 ksi. In Figure 43, the number of cycles for all specimens are plotted together on the same set of axes

FIGURE 42. STRESS RANGE vs. NUMBER OF CYCLES

versus percent of static ultimate load. The overlapping of points from each series indicates that neither the fatigue limit nor the fatigue life is influenced by the length of the shear span.

Resistance to Diagonal Tension and Shear-Compression Failures

Each of the preceding factors, i.e.; localized steel stress and excessive bar bending caused by the small number of flexural and diagonal cracks, points of high stress concentration, and stress range, limit the fatigue life of the longitudinal steel reinforcement to some degree. They do not explain, however, the lack of diagonal tension and shear-compression failures. Chang and Kesler⁽⁸⁾ reported on the fatigue behavior of 6 inch deep reinforced concrete beams and related the mode of failure to the magnitude of the repeated load. They concluded, for their particular specimens, that the beam would generally fail by fatigue of the reinforcement under two conditions:

1. If the maximum repeated load would not cause diagonal cracking during loading, i.e.; V/V_c is approximately equal to or less than 0.60.

2. If the maximum repeated load does not cause shear-compression failure before the reinforcement breaks, i.e.; M/M_s must be equal to or less than 0.70.

It can be seen from Table 4 that the repeated load was well above that required to produce diagonal tension failure for all specimens and, in some cases, was sufficient to cause shear-compression type failures. The absence of a major diagonal crack, obviously, prevented either type failure from occurring. It is here that the presence of a large tension flange influences the mode of failure. As

Table 4. Comparison of Test Strengths With Calculated Values

Beam	Applied Load P Max. (kips)	Calculated* Shear Moment (kips)	M/M _s	v _c Test (psi)	v _c ** Calc. (psi)	V/V _c
II P-3	37	P = 57.4	0.64	172	158	1.09
II P-4	43	P = 59.3	0.73	200	164	1.22
II P-5	38	P = 59.3	0.64	176	164	1.07
III P-1	26	P = 50.0	0.52	96	142	0.68
III P-3	39	P = 55.0	0.71	144	156	0.92
III P-5	33	P = 53.7	0.61	122	154	0.79
III P-6	41	P = 53.7	0.76	151	154	0.98
IV P-1	31	P = 52.0	0.60	95	151	0.63
IV P-2	35.5	P = 52.0	0.68	109	151	0.72

* Based on the expression of Moody. Appendix B

** $v_c = 1.9 \sqrt{f'_c} + 2500 \frac{nV_d}{M}$

discussed previously in the Introduction, inverted T-beams have been found to resist higher shearing stresses than regular T-beams and to resist the horizontal splitting of the concrete along the tensile reinforcement associated with diagonal cracking. Since the reinforcing bars are embedded in the concrete flange, there is an additional area of concrete surrounding the bars which seems to act in much the same way as stirrups in resisting dowel forces. The masses of concrete above and below the steel are prevented from separating as they would in a beam of rectangular cross-section, delaying the formation of diagonal failure cracks and allowing the longitudinal reinforcement to fail by fatigue.

SUMMARY AND CONCLUSIONS

1. The restrained reinforced concrete T-beams without web reinforcement subjected to repeated loading in this investigation were weak with respect to shear but failed by fatigue of the longitudinal steel reinforcement.

2. The stress range had the most significant effect on the fatigue life of the longitudinal reinforcement. Increasing the stress range decreased the fatigue life. The fatigue limit, or stress range at two million cycles of loading, was found to be 40 ksi.

3. The concrete tension flange seemed to produce the same effect as would be expected from web reinforcement by preventing horizontal concrete splitting along the tensile reinforcement thereby delaying diagonal cracking.

4. High steel stresses and excessive bar bending were localized at the section of maximum moment due to the small number of cracks.

5. The steel reinforcing bars failed on planes passing through points of high stress concentration. The cracks were found to have initiated at these points.

6. The shear span-to-depth ratio did not influence the mode of failure or the fatigue limit of the longitudinal steel reinforcement.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. ACI-ASCE Committee 426, "Shear and Diagonal Tension", ACI Journal, January, February, March, 1962, Proceedings, Vol. 59.
2. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-63)", ACI Standard, June 1963.
3. Acharya, D. N. and Kemp, K. O., "Significance of Dowel Forces on the Shear Failure of Rectangular Reinforced Concrete Beams Without Web Reinforcement", ACI Journal, October 1965, Proceedings, Vol. 62², P. 1265.
4. Acharya, D. N. and Kemp, K. O. - Authors closure to discussion of reference 3. ACI Journal, June 1966, Proceedings, Vol. 62², P. 1771.
5. Al-Alusi, A. F., "Diagonal Tension Strength of Reinforced Concrete T-Beams with Varying Shear Span", ACI Journal, May 1957, Proceedings, Vol. 53, P. 1067.
6. Burton, K. T., "Fatigue Tests of Reinforcing Bars", Journal of the PCA Research and Development Laboratories, Vol. 7 No. 3, September 1965; PCA Development Department, Bulletin D. 93.
7. Chang, T. S. and Kesler, C. E., "Static and Fatigue Strength in Shear of Beams With Tensile Reinforcement", ACI Journal, June 1958, Proceedings, Vol. 54, P. 1033.
8. Chang, T. S. and Kesler, C. E., "Fatigue Behavior of Reinforced Concrete Beams", ACI Journal, August 1958, Proceedings, Vol. 55. P. 245.
9. Fisher, J. W. and Viest, I. M., "Fatigue Tests of Bridge Materials of the AASHTO Road Test", AASHTO Road Test Technical Staff Papers, Special Report 66, 132-147, Highway Research Board (1961).
10. Hanson, J. M., Burton, K. T. and Hognestad, E., "Fatigue Tests of Reinforcing Bars - Effect of Deformation Pattern", Journal of the PCA Research and Development Laboratories, Vol. 10, No. 3, September 1968.
11. Harvey, W. N., "A Study of Diagonal Tension Failure in Reinforced Concrete Beams", M.S. Thesis, Purdue University, 1964.

12. Jones, R., "The Ultimate Strength of Reinforced Concrete Beams in Shear", Magazine of Concrete Research, August 1956, Vol. 8, No. 23, P. 69.
13. Kokubu, M. and Okamura, H., "Fundamental Study on Fatigue Behavior of Reinforced Concrete Beams Using High Strength Deformed Bars", Transactions of the Japan Society of Civil Engineers, No. 122, October 1965.
14. Krefeld, W. J. and Thurston, C. W., "Contribution of Longitudinal Steel to Shear Resistance of Reinforced Concrete Beams", ACI Journal, March, April 1966, Proceedings, Vol. 63, P. 325, 451.
15. Krefeld, W. J. and Thurston, C. W. - Authors closure to discussion of reference 14. ACI Journal, December 1966, Proceedings, Vol. 63², P. 1474.
16. Murdock, J. W. and Kesler, C. E., "Effect of Range of Stress on Fatigue Strengths of Plain Concrete Beams", ACI Journal, August 1958, Proceedings, Vol. 55, P. 221.
17. Nordby, G. M., "Fatigue of Concrete - A Review of Research", ACI Journal, August 1958, Proceedings, Vol. 55, P. 191.
18. Pfister, J. F. and Hognestad, E., "High Strength Bars as Concrete Reinforcement - Part 6. Fatigue Tests", Journal of the PCA Research and Development Laboratories, Vol. 6, No. 1, January 1964; PCA Development Department, Bulletin D 74.
19. Rogers, W. A., "A Study of the Behavior of Reinforced Concrete Beams Subjected to Repeated Loads", M.S. Thesis, Purdue University, 1968.
20. Stelson, T. E. and Cernica, J. N., "Fatigue Properties of Concrete Beams", ACI Journal, August 1958, Proceedings, Vol. 55, P. 255.
21. Taub, J. and Neville, A. M., "Resistance to Shear of Reinforced Concrete Beams", ACI Journal, August 1960, Proceedings, Vol. 57, P. 193.
22. The AASHTO Road Test, Report 4, Bridge Research, Special Report 61 D, Highway Research Board (1962).
23. Verna, J. R. and Stelson, T. E., "Failure of Small Reinforced Concrete Beams Under Repeated Loads", ACI Journal, October 1962, Proceedings, Vol. 59, P. 1489.
24. Verna, J. R. and Stelson, T. E., "Repeated Loading Effect on Ultimate Static Strength of Concrete Beams", ACI Journal, June 1963, Proceedings, Vol. 60, P. 743.

25. Watstein, D. and Mathey, R. G., "Strains in Beams Having Diagonal Cracks", ACI Journal, December 1958, Proceedings, Vol. 55, P. 717.
26. Wehr, K. E., "Shear Strength of Reinforced Concrete T-Beams", M. S. Thesis, Purdue University, 1967.

APPENDIX A

APPENDIX A

Stress-Strain Properties of the Reinforcement

Several coupons of the steel reinforcement were selected and tested to determine the properties. The resultant stress-strain properties of an average test for both size longitudinal steel are shown in Figures 44 and 45.

FIGURE 44. TYPICAL STRESS-STRAIN PROPERTIES - #6 LONGITUDINAL STEEL

FIGURE 45. TYPICAL STRESS-STRAIN PROPERTIES - # 5 LONGITUDINAL STEEL

APPENDIX B

APPENDIX B

Sample Calculations for Ultimate Load Prediction

The static ultimate load of a specimen of rectangular cross-section was estimate as the average of the values predicted by the shear-moment expressions of Moody and Morrow. For T-beams, the static ultimate load was taken as that predicted by the expression of Moody only. The statically loaded T-beams in Wehr's⁽²⁶⁾ investigation showed this value to be a good indication of the failure load and the expression of Morrow to be very conservative. The effect of compression steel has been neglected.

Moody developed the following expression.

$$M_s = pf_s \left(1 - \frac{k_2}{k_1 k_3} \frac{pf_s}{f'_c} \right) bd^2$$

where $k_2 = .42, \quad k_1 k_3 = 1.121 - .0485 \frac{f'_c}{1000}$

and
$$f_s = \frac{\frac{3M}{Vd} - 0.45}{\frac{3M}{Vd} + 0.55} \left[6.9 \times 10^{-4} E_s \left(-1 + \sqrt{1 + \frac{1450}{pE_s/k_1 k_3 f'_c}} \right) \right]$$

Morrow developed similar expressions for beams without stirrups.

$$M_s = pf_s \left(1 - \frac{k_2}{k_1 k_3} \frac{pf_s}{f'_c} \right) bd^2$$

where $\frac{k_2}{k_1 k_3} = .44, \quad k_1 k_3 = \frac{800 + f'_c}{70 + f'_c}$

and
$$f_s = \frac{1}{2} E_s k_{\epsilon_u} \left(-1 + \sqrt{1 + \frac{4 k_1 k_3 f'_c}{p E_s k_{\epsilon_u}}} \right)$$

$$10^4 k_{\epsilon_u} = \frac{1.116 a/d + .174}{a/d - .872}$$

For all cases the calculated load was based on the moment, M_s , developed at the edge of the support block.

Series I: $P_f = \frac{\text{Average } M_s}{10.18}$

Series II: $P_f = \frac{M_s}{9.32}$

Series III: $P_f = \frac{M_s}{10.18}$

Series IV: $P_f = \frac{M_s}{10.75}$

Sample Calculations

For Beam I P-1

$$f'_c = 5270 \text{ psi}$$

$$a/d = 3.95$$

$$E_s = 30 \times 10^6 \text{ psi}$$

$$p = .01318$$

Moody:

$$\frac{M}{Vd} = \frac{10.18 P}{.247 P (11.13)} = 3.7$$

$$k_1 k_3 = 1.121 - .0485 \left(\frac{5270}{1000} \right) = .866$$

$$f_s = \frac{3(3.7) - .45}{3(3.7) + .55} \left[6.9 \times 10^{-4} (30 \times 10^6) \left(-1 + \sqrt{1 + \frac{1450(.866)(5270)}{.01318(30 \times 10^6)}} \right) \right]$$

$$f_s = 61,000 \text{ psi}$$

$$M_s = .01318(61,000) \left(1 - \frac{.42(.01318)61,000}{.866 \cdot 5270} \right) 6 (11.13)^2$$

$$M_s = 554,000 \text{ in.-lb.}$$

Morrow:

$$k_1 k_3 = \frac{800 + 5270}{70 + 5270} = 1.136$$

$$10^4 k \epsilon_u = \frac{1.116(3.95) + .174}{3.95 - .872} = 1.486$$

$$f_s = \frac{1}{2}(30 \times 10^6)(1.486 \times 10^{-4}) \left(-1 + \sqrt{1 + \frac{4(1.136)5270}{(.01318)(30 \times 10^6)(1.486 \times 10^{-4})}} \right)$$

$$f_s = 42,8000 \text{ psi}$$

$$M_s = .01318(42,800) \left(1 - \frac{.44(.01318)42,800}{5270} \right) 6(11.13)^2$$

$$M_s = 399,000 \text{ in.-lb.}$$

$$\text{Ave. } M_s = \frac{554,000 + 399,000}{2} = 476,500 \text{ in.-lb.}$$

$$P_f = \frac{476,500}{10.18} = 46.9^k$$

$$P_{\text{test}} = 28.5^k$$

$$\% \text{ of } P_f = \frac{28.5}{46.9} = 61\%$$

APPENDIX C

Table Cl. Steel Temperature - Beam I P-1 (All Distances Measured from Support)

N (Cycles)	Load (kips)		No. 6 Bar - N*		No. 5 Bar - S**		Room Temperature °F
	Max.	Min.	0" 1½" East °F	1½" West	0" 3" East °F	3" West	
1	0	-	82	82	82	82	87
1	5	-	82	82	82	82	87
1	10	-	82	82	82	82	87
1	15	-	82	82	82	82	87
1	20	-	82	82	82	82	87
1	25	-	82	82	82	82	87
1	28.5	-	83	83	83	83	88
4,000	28.5	3	84	84	84	84	89
11,000	"	"	87	87	87	87	89
160,000	Test Interrupted - No Readings						
228,000	Test Resumed						
234,000	28.5	3	86	86	86	86	90
260,000	"	"	86	86	86	86	89
278,000	"	"	87	87	86	86	90

Table C1 (Continued)

N (Cycles)	Load (kips) Max.	Min.	No. 6 Bar - N*		No. 6 Bar - S**		Room Temperature °F
			0" East of	1½" West	0" East of	3" West	
287,000	28.5	3	88	88	87	87	91
310,000	"	"	88	88	88	88	88
337,000	"	"	89	89	89	89	90
408,000	"	"	88	88	88	88	87
415,000	33.1	4	88	88	88	88	87
614,000	"	"	82	82	82	82	82

* N - North Side of Beam

** S - South Side of Beam

Table C2. Steel and Concrete Strains - Beam II P-1

N (Cycles)	Load (kips)		No. 6 Bar (MII) At Support	$\frac{1}{2}$ " From Support	Compression Zone - $3\frac{1}{2}$ " From Support		
	Max.	Min.			From Bottom	1" From Bottom	2" From Bottom
1	5	-	26		-12	-17	-22
1	10	-	35		-38	-45	-50
1	15	-	51		-73	-71	-79
1	20	-	68		-115	-104	-102
1	25	-	84		-140	-138	-150
1	26	-	88		-155	-143	-160
1	31	-	675		-312	-235	-185
3,000	31	4	-		-186 $\pm$ 125	-138 $\pm$ 100	-115 $\pm$ 75
8,000	"	"	-		-204 $\pm$ 125	-154 $\pm$ 100	-128 $\pm$ 75
23,000	"	"	-		-228 $\pm$ 125	-175 $\pm$ 100	-145 $\pm$ 75
30,000	"	"	-		-252 $\pm$ 125	-185 $\pm$ 100	-175 $\pm$ 75
96,000	"	"	-		-265 $\pm$ 125	-190 $\pm$ 100	-185 $\pm$ 75
487,000	"	"	-		-298 $\pm$ 125	-188 $\pm$ 100	-180 $\pm$ 75
880,000	"	"	-		-313 $\pm$ 125	-195 $\pm$ 100	-171 $\pm$ 75

Table C2 (Continued)

N (cycles)	Load (kips)		No. 6 Bar (MII) At Support	Compression Zone - 3½" From Support		
	Max.	Min.		½" From Bottom	1" From Bottom	2" From Bottom
3,086,000	31	4	-	-	-197 ± 100	-154 ± 75
			Repeated Load Removed			
	35	-	-	-	-329	-252
	40	-	-	-	-362	-271
	45	-	-	-	-396	-295
	50	-	-	-	-435	-320
	55	-	-	-	-485	-345
	60	-	-	-	-527	-335
	65	-	-	-	-572	-492

Table C3. Steel and Concrete Strains - Mean II P-2

N (Cycles)	Load (kips) Max.	No. 6 Bar (No.) At Support	Compression Data - 3" From Support - 1"		Compression Data - 3" From Support - 1"		Compression Data - 3" From Support - 1"		Compression Data - 3" From Support - 1"	
			1" From Bottom	2" From Bottom	1" From Bottom	2" From Bottom	1" From Bottom	2" From Bottom	1" From Bottom	2" From Bottom
1	5	10	-115	-59	-75	-72	-100	-69		
1	10	26	-84	-103	-124	-125	-150	-106		
1	15	69	-126	-111	-160	-162	-194	-135		
1	20	107	-168	-175	-199	-203	-216	-164		
1	25	156	-210	-204	-176	-167	-185	-210		
1	27	1225	-273	-325	-280	-115	-160	-260		
1,000	27	650 ± 160	-334 ± 150	-290 ± 150	-250 ± 110	-266 ± 120	-300 ± 100	-120 ± 50		
7,000	"	1650 ± 1170	-324 ± 175	-308 ± 110	-275 ± 100	-320 ± 100	-376 ± 90	-163 ± 50		
352,000	"	-	-305 ± 175	-434 ± 150	-394 ± 100	-310 ± 125	-700 ± 100	-250 ± 50		
1,000,000	"	-	-330 ± 175	-480 ± 150	-440 ± 100	-315 ± 125	-750 ± 100	-270 ± 50		
1,130,000	"	-	-355 ± 175	-515 ± 150	-465 ± 100	-345 ± 125	-780 ± 100	-294 ± 50		
1,810,000	"	-	-365 ± 175	-550 ± 150	-490 ± 100	-350 ± 125	-800 ± 100	-310 ± 50		
2,190,000	"	-	-365 ± 175	-555 ± 150	-500 ± 100	-365 ± 125	-800 ± 100	-320 ± 50		
2,265,000	"	-	-390 ± 175	-575 ± 150	-515 ± 100	-400 ± 125	-820 ± 100	-335 ± 50		
2,580,000	"	-	-360 ± 175	-560 ± 150	-505 ± 100	-380 ± 125	-800 ± 100	-320 ± 50		
3,100,000	"	-	-350 ± 175	-580 ± 150	-510 ± 100	-335 ± 125	-700 ± 100	-310 ± 50		
4,130,000	"	-	-315 ± 175	-575 ± 150	-510 ± 100	-335 ± 125	-770 ± 100	-315 ± 50		
4,280,000	"	-	-360 ± 175	-590 ± 150	-520 ± 100	-395 ± 125	-790 ± 100	0325 ± 50		
4,550,000	"	-	-370 ± 175	-595 ± 150	-520 ± 100	-395 ± 125	-795 ± 100	-325 ± 50		
4,930,000	"	-	-380 ± 175	-600 ± 150	-530 ± 100	-400 ± 125	-800 ± 100	-370 ± 50		
Repeated Load Removed										
30	-	830	-600	-760	-660	-550	-900	-385		
35	-	1015	-675	-805	-695	-605	-915	-415		
40	-	1223	-715	-860	-735	-657	-980	-425		
45	-	1435	-850	-925	-775	-730	-1040	-450		
50	-	1600	-926	-975	-820	-807	-1095	-478		
55	-	1755	-1060	-1040	-835	-885	-1160	-525		
60	-	1865	-1117	-1096	-862	-1000	-1223	-510		

* R - North Side of Beam

** S - South Side of Beam

Table CII. Steel and Concrete Strains - Beam II P-3

N (Cycles)	Load (kips) Max. Min.	No. 6 Bar (MII) At Support	Compression Zone - 3" from Bottom 1" from Bottom	Compression Zone - 3" from Support - 1" from Bottom	Compression Zone - 3" from Bottom 1" from Bottom	Compression Zone - 3" from Support - 1" from Bottom
1	5	20	-28	-44	-38	-24
1	10	32	-44	-74	-58	-48
1	15	42	-51	-105	-90	-78
1	20	58	-134	-138	-122	-106
1	25	80	-174	-172	-157	-140
1	30	102	-217	-206	-187	-173
1	35	1160	-474	-334	-175	-247
1	37	1232	-515	-360	-153	-258
3,000	37	790 ± 100	-270 ± 175	-210 ± 110	-102 ± 60	-180 ± 125
15,000	"	765 ± 100	-222 ± 150	-220 ± 100	-126 ± 66	-165 ± 125
25,000	"	703 ± 100	-235 ± 175	-215 ± 100	-112 ± 60	-175 ± 125
305,000	"	820 ± 100	-175 ± 150	-270 ± 100	-226 ± 60	-180 ± 100
700,000	"	865 ± 100	-155 ± 150	-255 ± 100	-235 ± 60	-160 ± 100
1,170,000	"	870 ± 350	-160 ± 175	-270 ± 125	-290 ± 75	-165 ± 75
1,500,000	"	850 ± 200	-130 ± 175	-230 ± 125	-270 ± 75	-185 ± 100
1,815,000	"	-	-110 ± 150	-186 ± 125	-250 ± 60	-160 ± 100
2,213,000	"	-	-110 ± 150	-175 ± 125	-270 ± 50	-90 ± 100
2,600,000	"	-	-135 ± 150	-180 ± 125	-290 ± 50	-90 ± 75
2,670,000	"	-	-135 ± 150	-190 ± 125	-305 ± 50	-80 ± 75
2,891,000	"	-	Failure			-135 ± 100

* N - North Side of Beam

** S - South Side of Beam

Table 05. Steel and Concrete Strains - Beam II P-4

N (cycles)	Load (kips) Max.	No. 6 Bar - 1" At Support	No. 6 Bar - 1" At Support	Compressive Strain - Top of Support - 1" from Bottom			Tensile Strain - Top of Support - 1" from Bottom		
				1" from Bottom	1" from Top	1" from Top	1" from Bottom	1" from Top	1" from Bottom
1	5	-	18	-44	-37	-38	-48	-43	-44
1	10	-	28	-75	-60	-60	-60	-72	-67
1	15	-	39	-108	-84	-86	-116	-104	-94
1	20	-	56	-147	-128	-114	-167	-144	-125
1	25	-	83	-190	-168	-146	-208	-187	-167
1	30	-	1035	-240	-252	-168	-244	-240	-257
1	35	-	1295	-314	-314	-208	-308	-375	-275
1	40	-	1440	-590	-380	-240	-530	-422	-307
1	43	-	1536	-590	-443	-273	-630	-480	-336
6,000	43	910 ± 500	465 ± 295	-120 ± 250	-290 ± 175	-180 ± 100	-	-335 ± 175	-210 ± 125
11,000	"	875 ± 550	355 ± 400	-105 ± 250	-270 ± 175	-175 ± 100	-	-300 ± 175	-220 ± 125
23,000	"	905 ± 550	350 ± 350	-145 ± 250	-290 ± 175	-155 ± 100	-	-305 ± 175	-235 ± 125
50,000	"	-	320 ± 400	-145 ± 250	-245 ± 200	-200 ± 100	-	-305 ± 175	-255 ± 125
179,000	"	-	330 ± 395	-395 ± 275	-245 ± 200	-210 ± 125	-	-310 ± 175	-265 ± 125
270,000	"	-	350 ± 395	-440 ± 275	-395 ± 200	-215 ± 125	-	-320 ± 175	-270 ± 125
430,000	"	-	350 ± 395	-440 ± 275	-395 ± 200	-215 ± 125	-	-335 ± 175	-270 ± 125
870,000	"	-	460 ± 400	-120 ± 275	-345 ± 200	-245 ± 125	-	-350 ± 175	-260 ± 125
1,068,000	"	-	1110 ± 300	-350 ± 250	-280 ± 150	-190 ± 75	-	-435 ± 175	-325 ± 125
1,073,000	"	-	1190 ± 300	-350 ± 250	-285 ± 150	-190 ± 75	-	-450 ± 175	-340 ± 125
1,076,000	"	-	1210 ± 300	-350 ± 250	-285 ± 150	-190 ± 75	-	-450 ± 175	-340 ± 125
1,080,000	"	-	1250 ± 300	-350 ± 250	-285 ± 150	-190 ± 75	-	-450 ± 175	-340 ± 125
1,085,000	"	-	1345 ± 395	-350 ± 250	-285 ± 150	-190 ± 75	-	-445 ± 175	-345 ± 125
1,087,000	"	-	1380 ± 395	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,089,000	"	-	1440 ± 395	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,091,000	"	-	1450 ± 395	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,092,500	"	-	1475 ± 395	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,095,500	"	-	1555 ± 395	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,100,000	"	-	1710 ± 395	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,102,000	"	-	1855 ± 375	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,102,700	"	-	1895 ± 375	-350 ± 250	-295 ± 150	-190 ± 75	-	-465 ± 200	-345 ± 125
1,103,300	"	-	1955 ± 375	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 200	-345 ± 125
1,105,000	"	-	2055 ± 375	-350 ± 250	-285 ± 150	-190 ± 72	-	-465 ± 200	-345 ± 125

Table 55. Steel and Concrete Strains - Beam II F-4

N (cycles)	Load (kips) max.	No. 6 Bar - A At Support	No. 6 Bar - B At Support	Vertical Displacement in mm	Top Bar Strain	Bottom Bar Strain	Top Bar Strain	Bottom Bar Strain	Top Bar Strain	Bottom Bar Strain
1	5	-	20	18	-44	-37	-38	-48	-43	-41
1	10	-	34	28	-75	-62	-60	-80	-72	-67
1	15	-	42	39	-108	-94	-86	-118	-104	-94
1	20	-	63	56	-147	-128	-114	-157	-141	-125
1	25	-	86	83	-190	-168	-146	-205	-187	-167
1	30	-	1035	945	-340	-282	-185	-344	-340	-257
1	35	-	1205	1140	-413	-314	-205	-468	-375	-275
1	40	-	1410	1342	-500	-380	-240	-530	-422	-307
1	43	-	1560	1536	-500	-443	-273	-630	-480	-336
6,000	43	4	910 ± 500	645 ± 255	-420 ± 250	-290 ± 175	-180 ± 100	-	-335 ± 175	-210 ± 125
11,000	"	"	875 ± 550	355 ± 400	-400 ± 250	-270 ± 175	-175 ± 100	-	-300 ± 175	-220 ± 125
23,000	"	"	905 ± 550	350 ± 350	-425 ± 250	-290 ± 175	-165 ± 100	-	-305 ± 175	-235 ± 125
50,000	"	"	-	320 ± 400	-415 ± 250	-285 ± 200	-200 ± 100	-	-305 ± 175	-255 ± 125
178,000	"	"	-	310 ± 325	-395 ± 275	-285 ± 200	-210 ± 125	-	-310 ± 175	-265 ± 125
270,000	"	"	-	350 ± 325	-440 ± 275	-385 ± 200	-215 ± 125	-	-320 ± 175	-270 ± 125
430,000	"	"	-	350 ± 325	-410 ± 275	-385 ± 200	-240 ± 125	-	-335 ± 175	-270 ± 125
370,000	"	"	-	160 ± 400	-420 ± 275	-345 ± 200	-245 ± 125	-	-350 ± 175	-280 ± 125
1,068,000	"	"	-	1110 ± 300	-350 ± 250	-280 ± 150	-190 ± 75	-	-435 ± 175	-325 ± 125
1,073,000	"	"	-	1190 ± 300	-350 ± 250	-285 ± 150	-190 ± 75	-	-450 ± 175	-340 ± 125
1,076,000	"	"	-	1210 ± 300	-350 ± 250	-285 ± 150	-190 ± 75	-	-450 ± 175	-340 ± 125
1,080,000	"	"	-	1250 ± 300	-350 ± 250	-285 ± 150	-190 ± 75	-	-450 ± 175	-340 ± 125
1,085,000	"	"	-	1345 ± 325	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,087,000	"	"	-	1380 ± 325	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,089,000	"	"	-	1410 ± 325	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,091,000	"	"	-	1450 ± 325	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,092,500	"	"	-	1475 ± 325	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,095,500	"	"	-	1555 ± 325	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,100,000	"	"	-	1710 ± 325	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,102,000	"	"	-	1855 ± 375	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 175	-345 ± 125
1,102,700	"	"	-	1895 ± 375	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 200	-345 ± 125
1,103,300	"	"	-	1955 ± 375	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 200	-345 ± 125
1,105,000	"	"	-	2055 ± 375	-350 ± 250	-285 ± 150	-190 ± 75	-	-465 ± 200	-345 ± 125

Table 05 (Continued)

N (cycles)	Load (kips) Max.	No. 6 Bar - N [*] At Support	No. 4 Bar - max At Support	Compression Zone - 3' From Bottom 1/2" From Bottom	Compression Zone - 3' From Support - 1/2" 1/2" From Bottom	Compression Zone - 3' From Bottom 1/2" From Bottom	Compression Zone - 3' From Support - 1/2" 1/2" From Bottom
1,105,600	43	-	2200 ± 425	-350 ± 250	-285 ± 150	-190 ± 75	-315 ± 125
1,106,400	"	-	2330 ± 425	-350 ± 250	-285 ± 150	-190 ± 75	-315 ± 125
1,107,300	"	-	2440 ± 500	-350 ± 250	-285 ± 150	-190 ± 75	-315 ± 125
1,107,600	"	-	3230 ± 650	-350 ± 250	-285 ± 150	-190 ± 75	-315 ± 125
1,123,000	"	Failure					

* N - North Side of Beam

** S - South Side of Beam

Table 06. Steel and Concrete Strains - Beam II 1-5

N (Cycles)	Load (kips) Max.	No. 6 Bar - Top No. - At Support	No. 6 Bar - Bottom No. - At Support	No. 6 Bar - Top Max. - At Support	No. 6 Bar - Bottom Max. - At Support	Compression Zone - 3/4" Spacing		Tension Zone - 3/4" Spacing		Compression Zone - 1/2" Spacing		Tension Zone - 1/2" Spacing	
						Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
1	5	-	23	20	16	-20	-35	-32	-20	-20	-30	-44	-
1	10	-	36	28	24	-46	-45	-55	-43	-60	-62	-80	-
1	15	-	55	39	36	-70	-70	-80	-75	-94	-94	-110	-
1	20	-	75	55	50	-100	-90	-105	-105	-125	-125	-145	-
1	25	-	104	80	77	-130	-120	-135	-135	-160	-160	-180	-
1	30	-	140	133	-	-	-	-	-	-	-	-	-
1	30	-	894	704	655	-205	-225	-200	-200	-260	-260	-235	-
1	35	-	1070	835	780	-305	-350	-225	-325	-325	-325	-285	-
1	38	-	1170	905	845	-350	-275	-230	-355	-355	-355	-315	-
4,000	38	4	760 ± 425	555 ± 350	600 ± 350	-425 ± 160	-465 ± 125	-435 ± 100	-205 ± 160	-205 ± 160	-205 ± 160	-210 ± 100	-
100,000	"	"	835 ± 450	670 ± 350	615 ± 350	-430 ± 175	-445 ± 110	-50 ± 100	-180 ± 160	-250 ± 110	-250 ± 110	-380 ± 100	-
300,000	"	"	820 ± 450	575 ± 350	605 ± 350	-430 ± 175	-440 ± 125	-35 ± 90	-160 ± 160	-275 ± 110	-275 ± 110	-380 ± 100	-
485,000	"	"	800 ± 450	575 ± 350	620 ± 385	-435 ± 175	-430 ± 125	-45 ± 90	-170 ± 160	-280 ± 110	-280 ± 110	-360 ± 100	-
660,000	"	"	825 ± 450	580 ± 350	620 ± 385	-435 ± 175	-435 ± 125	-35 ± 90	-175 ± 160	-285 ± 110	-285 ± 110	-390 ± 100	-
790,000	"	"	840 ± 425	580 ± 370	620 ± 385	-430 ± 175	-430 ± 125	-30 ± 90	-175 ± 160	-290 ± 110	-290 ± 110	-420 ± 100	-
1,063,000	"	"	910 ± 425	550 ± 350	630 ± 385	-440 ± 175	-440 ± 125	-20 ± 90	-175 ± 160	-305 ± 110	-305 ± 110	-450 ± 100	-
1,235,000	"	"	910 ± 425	510 ± 350	625 ± 385	-440 ± 175	-445 ± 125	-10 ± 90	-175 ± 160	-310 ± 110	-310 ± 110	-510 ± 100	-
1,445,000	"	"	975 ± 425	510 ± 340	630 ± 385	-450 ± 175	-445 ± 125	0 ± 90	-180 ± 160	-315 ± 110	-315 ± 110	-560 ± 100	-
1,850,000	"	"	825 ± 410	535 ± 325	640 ± 385	-445 ± 175	-450 ± 125	-25 ± 90	-160 ± 160	-325 ± 110	-325 ± 110	-630 ± 100	-
2,610,000	"	"	775 ± 425	545 ± 325	640 ± 385	-485 ± 175	-470 ± 125	-80 ± 90	-200 ± 160	-355 ± 110	-355 ± 110	-695 ± 100	-
2,780,000	"	"	780 ± 440	530 ± 350	610 ± 385	-495 ± 175	-470 ± 125	-95 ± 90	-190 ± 160	-360 ± 110	-360 ± 110	-705 ± 100	-
3,000,000	"	"	840 ± 450	530 ± 350	650 ± 350	-500 ± 175	-475 ± 125	-95 ± 90	-200 ± 160	-360 ± 110	-360 ± 110	-700 ± 120	-
3,180,000	"	"	770 ± 490	530 ± 350	655 ± 350	-500 ± 175	-475 ± 125	-110 ± 90	-200 ± 160	-365 ± 110	-365 ± 110	-710 ± 120	-
3,375,000	"	"	805 ± 490	530 ± 350	645 ± 350	-505 ± 175	-465 ± 125	-120 ± 90	-205 ± 160	-370 ± 110	-370 ± 110	-715 ± 120	-
3,750,000	"	"	770 ± 490	550 ± 350	640 ± 350	-500 ± 175	-460 ± 125	-145 ± 90	-215 ± 160	-385 ± 110	-385 ± 110	-740 ± 120	-
3,833,000	"	"	925 ± 400	535 ± 375	645 ± 375	-520 ± 175	-460 ± 125	-155 ± 90	-215 ± 160	-390 ± 110	-390 ± 110	-745 ± 120	-
4,105,000	"	"	345 ± 150	-130 ± 100	-	-270 ± 150	-440 ± 100	-110 ± 50	-300 ± 190	-470 ± 160	-470 ± 160	-825 ± 125	-
4,292,000	"	"	Failure	-	-	-	-	-	-	-	-	-	-

* N - North Side of Beam

** S - South Side of Beam

Table C7. Steel and Concrete Strains - Near I-15-1

N (cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar - N*		No. 6 Bar - S**		Compression Zone - 3/8" from Support - N*		Compression Zone - 3/8" from Support - S**		Compression Zone - 3/8" from Bottom		Compression Zone - 3/8" from Bottom	
			At Support	At Support	At Support	At Support	3/8" from Bottom	3/8" from Bottom	3/8" from Bottom	3/8" from Bottom	3/8" from Bottom	3/8" from Bottom	3/8" from Bottom	3/8" from Bottom
1	5	-	14	12	-36	-27	-16	-24	-20	-20	-24	-20	-24	-20
1	10	-	30	30	-70	-45	-42	-50	-45	-45	-50	-45	-55	-45
1	15	-	42	46	-110	-66	-74	-80	-75	-75	-80	-75	-84	-75
1	20	-	67	72	-165	-110	-110	-115	-110	-110	-115	-110	-120	-110
1	25	-	908	1018	-350	-262	-170	-240	-150	-150	-240	-150	-250	-150
1	26	-	934	1066	-384	-253	-193	-248	-156	-156	-248	-156	-254	-156
4,000	20	4	670 ± 350	755 ± 425	-264 ± 170	-190 ± 130	-120 ± 90	-135 ± 120	-80 ± 75	-80 ± 75	-135 ± 120	-80 ± 75	-135 ± 110	-80 ± 75
12,000	"	"	690 ± 375	770 ± 425	-280 ± 175	-195 ± 135	-120 ± 100	-150 ± 120	-95 ± 60	-95 ± 60	-150 ± 120	-95 ± 60	-145 ± 100	-95 ± 60
204,000	"	"	810 ± 350	895 ± 425	-300 ± 160	-160 ± 125	-70 ± 100	-150 ± 110	-150 ± 60	-150 ± 60	-150 ± 110	-150 ± 60	-140 ± 100	-150 ± 60
400,000	"	"	855 ± 350	915 ± 425	-320 ± 160	-170 ± 125	-75 ± 100	-160 ± 110	-155 ± 60	-155 ± 60	-160 ± 110	-155 ± 60	-140 ± 100	-155 ± 60
680,000	"	"	945 ± 350	1000 ± 425	-320 ± 160	-170 ± 125	-60 ± 90	-165 ± 110	-160 ± 60	-160 ± 60	-165 ± 110	-160 ± 60	-140 ± 100	-160 ± 60
771,000	"	"	1040 ± 300	1025 ± 450	-320 ± 175	-170 ± 110	-65 ± 100	-172 ± 110	-165 ± 60	-165 ± 60	-172 ± 110	-165 ± 60	-140 ± 100	-165 ± 60
1,104,000	"	"	-	-	-325 ± 175	-170 ± 110	-60 ± 100	-165 ± 110	-165 ± 60	-165 ± 60	-165 ± 110	-165 ± 60	-135 ± 100	-165 ± 60
1,475,000	"	"	-	-	-348 ± 160	-182 ± 125	-60 ± 90	-185 ± 100	-175 ± 60	-175 ± 60	-185 ± 100	-175 ± 60	-145 ± 100	-175 ± 60
1,551,000	"	"	-	-	-370 ± 160	-200 ± 125	-60 ± 90	-200 ± 100	-190 ± 60	-190 ± 60	-200 ± 100	-190 ± 60	-155 ± 100	-190 ± 60
2,606,000	"	"	-	-	-405 ± 150	-225 ± 125	-80 ± 90	-240 ± 100	-210 ± 50	-210 ± 50	-240 ± 100	-210 ± 50	-180 ± 100	-210 ± 50
3,095,000	"	"	-	-	-600 ± 200	-345 ± 150	-150 ± 90	-295 ± 75	-155 ± 25	-155 ± 25	-295 ± 75	-155 ± 25	-110 ± 70	-155 ± 25
3,111,000	"	"	Failure	-	-	-	-	-	-	-	-	-	-	-

* N - North Side of Beam

** S - South Side of Beam

Table C8. Steel and Concrete Strains - Beam III P-2

N (Cycles)	Load (kips) kips	No. 6 Bar - 1# At Support	No. 6 Bar - 2# At Support	Compression Zone - 3" from Support - 1#		Compression Zone - 3" from Support - 2#		Compression Zone - 3" from Support - 3#	
				1" from Bottom	2" from Bottom	1" from Bottom	2" from Bottom	1" from Bottom	2" from Bottom
1	5	-	17	-34	-32	-28	-32	-28	-28
1	10	-	30	-66	-68	-59	-69	-62	-62
1	15	-	50	-109	-103	-92	-106	-98	-98
1	20	-	80	-150	-145	-126	-146	-135	-135
1	25	-	125	-200	-186	-167	-194	-176	-176
1	30	-	124.8	-246	-225	-214	-278	-287	-287
1	32	-	1317	-350	-250	-100	-394	-290	-290
2,000	32	4	880 ± 175	-225 ± 160	-215 ± 100	-255 ± 175	-215 ± 150	-170 ± 110	-170 ± 110
10,000	"	"	940 ± 175	-255 ± 175	-170 ± 110	-255 ± 175	-238 ± 150	-155 ± 100	-155 ± 100
25,000	"	"	870 ± 175	-280 ± 175	-185 ± 110	-255 ± 175	-235 ± 150	-155 ± 100	-155 ± 100
346,000	"	"	925 ± 175	-330 ± 175	-250 ± 110	-320 ± 175	-280 ± 150	-215 ± 100	-215 ± 100
681,000	"	"	940 ± 150	-375 ± 175	-285 ± 110	-355 ± 175	-310 ± 150	-215 ± 100	-215 ± 100
737,000	"	"	965 ± 150	-385 ± 175	-295 ± 110	-360 ± 175	-315 ± 150	-250 ± 100	-250 ± 100
753,000	"	"	970 ± 150	-390 ± 175	-290 ± 110	-370 ± 175	-320 ± 150	-255 ± 100	-255 ± 100
1,077,000	"	"	995 ± 150	-415 ± 160	-320 ± 110	-380 ± 160	-330 ± 150	-260 ± 100	-260 ± 100
2,113,000	"	"	1000 ± 150	-420 ± 160	-320 ± 110	-375 ± 160	-330 ± 150	-260 ± 100	-260 ± 100
2,465,000	"	"	1000 ± 100	-440 ± 150	-350 ± 100	-420 ± 150	-365 ± 140	-275 ± 100	-275 ± 100
1,424,000	"	"	1010 ± 100	-440 ± 150	-315 ± 100	-420 ± 150	-372 ± 140	-275 ± 100	-275 ± 100
1,532,000	"	"	1020 ± 100	-445 ± 150	-350 ± 100	-425 ± 150	-380 ± 140	-280 ± 100	-280 ± 100
1,592,000	"	"	1020 ± 100	-460 ± 150	-360 ± 100	-430 ± 150	-385 ± 140	-285 ± 100	-285 ± 100
1,837,000	"	"	1025 ± 100	-470 ± 150	-375 ± 100	-435 ± 150	-400 ± 140	-290 ± 90	-290 ± 90
1,862,000	"	"	1025 ± 100	-488 ± 150	-395 ± 100	-445 ± 150	-415 ± 140	-305 ± 90	-305 ± 90
1,713,000	"	"	1025 ± 100	-490 ± 150	-400 ± 100	-460 ± 150	-425 ± 140	-305 ± 90	-305 ± 90
2,235,000	"	"	1045 ± 100	-510 ± 150	-420 ± 100	-470 ± 150	-425 ± 140	-295 ± 90	-295 ± 90
2,594,000	"	"	1045 ± 100	-520 ± 150	-435 ± 100	-480 ± 150	-435 ± 140	-240 ± 90	-240 ± 90
2,656,000	"	"	1045 ± 100	-535 ± 140	-450 ± 80	-495 ± 150	-448 ± 140	-225 ± 90	-225 ± 90
2,687,000	"	"	1050 ± 100	-540 ± 140	-445 ± 80	-495 ± 150	-450 ± 140	-220 ± 90	-220 ± 90
2,771,000	"	"	1060 ± 100	-550 ± 140	-465 ± 80	-480 ± 150	-445 ± 140	-195 ± 90	-195 ± 90
3,031,000	"	"	1060 ± 100	-550 ± 140	-465 ± 80	-480 ± 150	-445 ± 140	-195 ± 90	-195 ± 90
3,051,000	"	"	1070 ± 100	-550 ± 140	-465 ± 80	-480 ± 150	-445 ± 140	-195 ± 90	-195 ± 90
3,063,000	"	"	1070 ± 100	-560 ± 140	-470 ± 80	-500 ± 150	-450 ± 140	-195 ± 90	-195 ± 90
3,159,000	"	"	1070 ± 100	-560 ± 140	-470 ± 80	-500 ± 150	-450 ± 140	-195 ± 90	-195 ± 90

Table CB (Continued)

N (Cycles)	Load (kips) Max.	4	1065 ± 100	No. 6 Bar - V _a At Support	No. 6 Bar - S _{ss} At Support	Compression Zone - 3" From Support - 1" From Bottom		Compression Zone - 3" From Support - 2" From Bottom		-200 ± 90
						V _a From Bottom	V _a From Bottom	V _a From Bottom	V _a From Bottom	
Repeated Load Removed										
10	-	-	900	5150	-470	-525	-140	-140	-115	-165
20	-	-	1220	5790	-550	-615	-515	-555	-505	-220
30	-	-	1520	6115	-610	-705	-580	-660	-600	-275
35	-	-	1670	6275	-695	-760	-605	-710	-660	-315
40	-	-	1815	6420	-730	-795	-645	-755	-695	-325
45	-	-	2020	6650	-750	-810	-625	-770	-710	-315
50	-	-	2155	6790	-805	-860	-655	-820	-765	-350
55	-	-	2325	6945	-895	-910	-690	-890	-800	-
60	-	-	2380	6965	-1100	-1035	-735	-980	-880	-
65	-	-	2500	7035	-	-	-	-	-	-
74	-	-	Failure	-	-	-	-	-	-	-

* N - North Side of Beam

** S - South Side of Beam

Table 09. Steel and Concrete Strains - Beam III P-3

N (cycles)	Load (kips) max.	No. 6 Bar - No. At Support	No. 6 Bar - No. At Support	Compression Zone - 3" From Support - See		Compression Zone - 3" From Support - See		Compression Zone - 3" From Support - See	
				1" From Bottom	2" From Bottom	1" From Bottom	2" From Bottom	1" From Bottom	2" From Bottom
1	5	-	13	-8	-35	-24	-46	-40	-24
1	10	-	28	-70	-70	-50	-82	-76	-50
1	15	-	43	-106	-106	-75	-125	-115	-78
1	20	-	65	-148	-145	-106	-179	-159	-112
1	25	-	91	-192	-188	-146	-237	-204	-144
1	30	-	118	-244	-208	-186	-320	-220	-70
1	35	-	146	-300	-250	-226	-400	-320	-135
1	39	-	174	-352	-294	-266	-470	-345	-172
5,000	39	4	820 ± 500	-406 ± 260	-350 ± 225	-165 ± 125	-380 ± 250	-265 ± 190	-104 ± 100
8,000	"	"	810 ± 500	-415 ± 260	-358 ± 225	-175 ± 125	-380 ± 250	-262 ± 190	-102 ± 100
22,000	"	"	810 ± 475	-420 ± 260	-375 ± 225	-175 ± 125	-375 ± 250	-250 ± 190	-98 ± 100
72,000	"	"	815 ± 475	-420 ± 260	-390 ± 225	-180 ± 125	-385 ± 250	-265 ± 190	-105 ± 90
118,000	"	"	900 ± 475	-390 ± 250	-400 ± 225	-175 ± 125	-405 ± 250	-285 ± 190	-100 ± 90
700,000	"	"	945 ± 460	-390 ± 250	-420 ± 210	-195 ± 110	-450 ± 250	-315 ± 190	-110 ± 90
784,000	"	"	945 ± 460	-380 ± 250	-420 ± 210	-190 ± 110	-445 ± 250	-320 ± 190	-105 ± 90
834,000	"	"	970 ± 460	-395 ± 250	-430 ± 210	-190 ± 110	-450 ± 250	-320 ± 190	-110 ± 90
1,086,000	"	"	1035 ± 475	-395 ± 250	-440 ± 210	-200 ± 110	-470 ± 250	-340 ± 190	-120 ± 90
1,178,000	"	"	1065 ± 475	-400 ± 250	-450 ± 210	-210 ± 110	-475 ± 250	-348 ± 190	-125 ± 90
1,498,000	"	"	1115 ± 475	-405 ± 250	-465 ± 210	-215 ± 110	-485 ± 250	-365 ± 190	-130 ± 90
1,836,000	"	"	"	-395 ± 250	-465 ± 210	-225 ± 120	-500 ± 250	-380 ± 200	-125 ± 100
1,947,000	"	"	"	-400 ± 250	-470 ± 210	-225 ± 120	-510 ± 250	-385 ± 200	-130 ± 100
2,250,000	"	"	"	-385 ± 250	-470 ± 210	-225 ± 120	-510 ± 250	-390 ± 200	-120 ± 100
2,449,000	"	"	"	-620 ± 300	-680 ± 250	-320 ± 125	-125 ± 210	-300 ± 150	-40 ± 50
2,540,000	"	"	Failure						

* N - North Side of Beam

** S - South Side of Beam

Table C10. Steel and Concrete Strands - Beam III F-4

N (Cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar - N ^o At Support	No. 6 Bar - S ^o At Support	Compression Zone - 32' from Support - N ^o 1" from Bottom	Compression Zone - 32' from Support - S ^o 1" from Bottom	Compression Zone - 32' from Support - N ^o 1" from Bottom	Compression Zone - 32' from Support - S ^o 1" from Bottom
1	5	-	25	20	-18	-22	-15	-25
1	10	-	44	19	-52	-54	-40	-67
1	15	-	62	57	-88	-86	-68	-95
1	20	-	80	74	-122	-122	-94	-133
1	25	-	99	95	-156	-157	-125	-172
1	30	-	96.5	850	-270	-251	-154	-250
1	35	-	1225	1165	-383	-362	-210	-292
1	40	-	1422	1355	-465	-430	-250	-325
1	44	-	1590	1580	-515	-508	-285	-375
4,000	44	-	1015 ± 500	1020 ± 550	-385 ± 250	-385 ± 290	-105 ± 100	-200 ± 150
45,000	"	"	1035 ± 490	1025 ± 525	-430 ± 250	-460 ± 290	-105 ± 100	-180 ± 150
298,000	"	"	1215 ± 490	1110 ± 525	-460 ± 250	-490 ± 200	-85 ± 100	-170 ± 150
353,000	"	"	1230 ± 490	1150 ± 525	-470 ± 250	-500 ± 200	-90 ± 100	-170 ± 150
659,000	"	"	1080 ± 580	1315 ± 580	-520 ± 290	-395 ± 250	-100 ± 100	-185 ± 150
743,000	"	"	-	-	-525 ± 275	-370 ± 250	-100 ± 110	-190 ± 150
833,000	"	"	-	-	-540 ± 275	-355 ± 250	-100 ± 110	-190 ± 150
1,072,000	"	"	-	-	-580 ± 275	-345 ± 250	-120 ± 110	-205 ± 150
1,368,000	"	"	Failure	-	-	-	-	-

• N - North Side of Beam

•• S - South Side of Beam

Table C12. Steel Temperature - Beam III P-5 (All Distances Measured From Support)

(Cycles)	Load (kips)		No. 6 Bar - N*		No. 6 Bar - S**		Room Temperature of
	Max.	Min.	3" East	1" West of	3" East	1" West of	
1	33	4	71	71	71	71	76
3,000	"	"	72	72	72	72	76
9,000	"	"	72	72	72	72	76
16,000	"	"	74	74	74	73	76
26,000	"	"	76	75	74	75	76
100,000	"	"	75	75	74	75	76
300,000	"	"	74	74	74	74	76
1,000,000	"	"	77	77	76	76	76
1,380,000	"	"	69	69	69	69	76

* N - North Side of Beam

** S - South Side of Beam

Table 113. Steel and Concrete Strains - Beam III P-6

N (Cycles)	Load (kips) max.	Load (kips) Min.	No. 6 Bar - Top		No. 6 Bar - Bottom		No. 6 Bar - Tip		No. 6 Bar - Bottom		Compression Zone - 1" from Bottom		Compression Zone - 1" from Bottom		Compression Zone - 1" from Bottom	
			At Support	See - At Support	At Support	See - At Support	At Support	See - At Support	At Support	See - At Support	At Support	See - At Support	At Support	See - At Support	At Support	See - At Support
1	5	-	23		15		18		15		-10		-6		-21	
1	10	-	39		26		35		26		-11		-11		-43	
1	15	-	50		33		48		35		-87		-80		-75	
1	20	-	73		56		72		62		-130		-124		-115	
1	25	-	104		90		115		105		-180		-135		-165	
1	25	-	740		625		915		752		-375		-336		-225	
1	30	-	1010		880		1130		950		-385		-310		-275	
1	35	-	1175		1020		1280		1085		-410		-310		-355	
1	40	-	1340		1155		1440		1210		-470		-335		-445	
1	42	-	1385		1160		1485		1190		-480		-335		-470	
5,000	41	4	920 ± 500		700 ± 110		1095 ± 550		685 ± 100		-335 ± 225		-220 ± 150		-110 ± 260	
100,000	"	"	1000 ± 500		720 ± 125		1120 ± 550		715 ± 100		-300 ± 250		-165 ± 175		-125 ± 260	
300,000	"	"	1025 ± 475		720 ± 110		1065 ± 490		775 ± 100		-285 ± 250		-130 ± 160		-105 ± 260	
413,000	"	"	1030 ± 475		725 ± 110		790 ± 375		790 ± 100		-280 ± 250		-120 ± 160		-125 ± 250	
674,000	"	"	1075 ± 475		760 ± 100		390 ± 175		810 ± 100		-285 ± 250		-100 ± 175		-145 ± 250	
780,000	"	"	1105 ± 500		785 ± 100		310 ± 125		810 ± 100		-265 ± 250		-115 ± 175		-160 ± 250	
911,000	"	"	Failure													

• N - North Side of Beam

• S - South Side of Beam

Table C11. Steel and Concrete Strains - Beam IV P-3

N (cycles)	Load (kips) Max.	No. 6 Bar - Top		No. 6 Bar - Bottom		No. 6 Bar - Top		No. 6 Bar - Bottom		Compression Zone - 1" from Bottom		Compression Zone - 3" from Support - No		Compression Zone - 3" from Support - Max	
		Min.	Max	Min.	Max	Min.	Max	Min.	Max	Min.	Max	Min.	Max	Min.	Max
1	5	-	16	11	12	18	12	-28	-28	-28	-40	-40	-40	-40	-40
1	10	-	26	19	18	28	18	-67	-67	-67	-80	-80	-80	-80	-80
1	15	-	46	35	28	46	28	-112	-112	-112	-120	-120	-120	-120	-120
1	20	-	72	46	40	76	40	-152	-152	-152	-157	-157	-157	-157	-157
1	25	-	1136	756	700	1115	700	-290	-290	-290	-215	-215	-215	-215	-215
1	30	-	1316	840	975	1354	975	-400	-400	-400	-275	-275	-275	-275	-275
1	31	-	1300	885	1010	1400	1010	-448	-448	-448	-300	-300	-300	-300	-300
2,000	31	4	810 ± 175	-	615 ± 100	940 ± 500	615 ± 100	-370 ± 190	-370 ± 190	-370 ± 190	-180 ± 100	-180 ± 100	-180 ± 100	-180 ± 100	-180 ± 100
56,000	*	*	1025 ± 175	-	635 ± 100	1040 ± 475	635 ± 100	-530 ± 200	-530 ± 200	-530 ± 200	-240 ± 100	-240 ± 100	-240 ± 100	-240 ± 100	-240 ± 100
63,000	*	*	1040 ± 175	-	635 ± 100	1155 ± 475	635 ± 100	-540 ± 200	-540 ± 200	-540 ± 200	-250 ± 100	-250 ± 100	-250 ± 100	-250 ± 100	-250 ± 100
64,500	*	*	1050 ± 175	-	635 ± 100	1220 ± 400	635 ± 100	-540 ± 200	-540 ± 200	-540 ± 200	-255 ± 100	-255 ± 100	-255 ± 100	-255 ± 100	-255 ± 100
305,000	*	*	2720 ± 500	-	725 ± 375	-	725 ± 375	-725 ± 200	-725 ± 200	-725 ± 200	-375 ± 100	-375 ± 100	-375 ± 100	-375 ± 100	-375 ± 100
400,000	*	*	2915 ± 500	-	740 ± 375	-	740 ± 375	-760 ± 200	-760 ± 200	-760 ± 200	-385 ± 100	-385 ± 100	-385 ± 100	-385 ± 100	-385 ± 100
717,000	*	*	-	-	760 ± 360	-	760 ± 360	-825 ± 375	-825 ± 375	-825 ± 375	-445 ± 100	-445 ± 100	-445 ± 100	-445 ± 100	-445 ± 100
1,073,000	*	*	-	655 ± 375	805 ± 375	-	805 ± 375	-890 ± 190	-890 ± 190	-890 ± 190	-490 ± 100	-490 ± 100	-490 ± 100	-490 ± 100	-490 ± 100
1,458,000	*	*	-	655 ± 375	805 ± 375	-	805 ± 375	-905 ± 190	-905 ± 190	-905 ± 190	-505 ± 100	-505 ± 100	-505 ± 100	-505 ± 100	-505 ± 100
1,839,000	*	*	-	610 ± 375	825 ± 375	-	825 ± 375	-930 ± 190	-930 ± 190	-930 ± 190	-520 ± 100	-520 ± 100	-520 ± 100	-520 ± 100	-520 ± 100
2,224,000	*	*	-	605 ± 375	835 ± 375	-	835 ± 375	-965 ± 190	-965 ± 190	-965 ± 190	-545 ± 100	-545 ± 100	-545 ± 100	-545 ± 100	-545 ± 100
2,603,000	*	*	-	640 ± 375	845 ± 375	-	845 ± 375	-1010 ± 190	-1010 ± 190	-1010 ± 190	-585 ± 100	-585 ± 100	-585 ± 100	-585 ± 100	-585 ± 100
2,904,000	*	*	-	585 ± 375	845 ± 375	-	845 ± 375	-1035 ± 190	-1035 ± 190	-1035 ± 190	-605 ± 100	-605 ± 100	-605 ± 100	-605 ± 100	-605 ± 100
3,125,000	*	*	-	680 ± 375	850 ± 375	-	850 ± 375	-1040 ± 190	-1040 ± 190	-1040 ± 190	-620 ± 100	-620 ± 100	-620 ± 100	-620 ± 100	-620 ± 100
3,391,000	*	*	-	725 ± 375	870 ± 375	-	870 ± 375	-1060 ± 175	-1060 ± 175	-1060 ± 175	-630 ± 100	-630 ± 100	-630 ± 100	-630 ± 100	-630 ± 100
3,400,000	*	*	Failure	-	-	-	-	-	-	-	-	-	-	-	-

* N - North Side of Beam

** S - South Side of Beam

Table G15. Steel and Concrete Strains - Beam IV P-2

N (Cycles)	Load (kips) Max.	No. 6 Bar - Top		No. 6 Bar - Bottom		Compression Zone - Top		Compression Zone - Bottom		Compression Zone - Top		Compression Zone - Bottom	
		At Support	At Support	At Support	At Support	% From Bottom	% From Bottom	% From Bottom	% From Bottom	% From Bottom	% From Bottom	% From Bottom	% From Bottom
1	5	-	24	26	17	-34	-38	-43	-35	-40	-35	-30	
1	10	-	42	42	24	-73	-75	-83	-82	-88	-82	-70	
1	15	-	70	70	38	-115	-118	-115	-132	-138	-132	-106	
1	20	-	136	151	63	-104	-104	-156	-184	-192	-184	-144	
1	25	-	1210	1308	902	-108	-374	-273	-276	-334	-276	-244	
1	30	-	1145	1170	1083	-182	-437	-313	-356	-432	-356	-182	
1	35	-	1542	1594	1144	-545	-498	-350	-492	-598	-492	-234	
1	35.5	-	1553	1574	1168	-578	-510	-330	-517	-624	-517	-247	
3,000	35.5	4	1110 ± 900	940 ± 550	705 ± 375	-330 ± 225	-265 ± 200	± 30 ± 60	-454 ± 250	-130 ± 112			
12,000	*	*	2380 ± 900	975 ± 550	705 ± 375	-330 ± 200	-285 ± 175	± 90 ± 140	-475 ± 225	-150 ± 100			
18,000	*	*	2610 ± 900	1345 ± 550	695 ± 350	-330 ± 200	-285 ± 175	± 80 ± 140	-475 ± 225	-155 ± 100			
22,000	*	*	3010 ± 900	2300 ± 800	700 ± 350	-330 ± 200	-285 ± 175	± 80 ± 140	-475 ± 225	-155 ± 100			
45,000	*	*	3550 ± 900	-	710 ± 375	-330 ± 225	-300 ± 160	± 45 ± 50	-465 ± 250	-160 ± 100			
280,000	*	*	-	-	735 ± 375	-305 ± 225	-340 ± 160	± 20 ± 50	-470 ± 250	-170 ± 100			
366,000	*	*	-	-	735 ± 375	-300 ± 225	-340 ± 160	± 10 ± 50	-470 ± 250	-170 ± 100			
645,000	*	*	-	-	765 ± 375	-310 ± 225	-365 ± 160	± 10 ± 50	-480 ± 250	-175 ± 100			
822,000	*	*	-	-	82 ± 30	-460 ± 250	-510 ± 200	± 20 ± 75	-395 ± 200	-65 ± 50			
896,000	*	*	Failure	-	-	-	-	-	-	-	-		

* K - North Side of Beam

** S - South Side of Beam

Table 116. Steel and Concrete Strains - Beam IV P-3

N (Cycles)	Load (kips) Max.	No. 6 Bar - Top		No. 6 Bar - Bottom		No. 6 Bar - Top		No. 6 Bar - Bottom		Compression Strain - 3" from Support - 1/2" from Bottom		Compression Strain - 1" from Support - 1/2" from Bottom	
		N - At Support	N - At Support	N - At Support	N - At Support	S - At Support	S - At Support	S - At Support	S - At Support	by From Bottom	by From Bottom	by From Bottom	by From Bottom
1	5	-	23	15	25	17	-20	-31	-25	-30	-21	-25	-25
1	10	-	38	28	40	26	-52	-75	-42	-70	-50	-42	-42
1	15	-	58	39	55	38	-88	-128	-105	-118	-85	-102	-102
1	20	-	83	54	78	51	-123	-180	-150	-164	-120	-143	-143
1	25	-	128	80	119	77	-164	-243	-204	-225	-160	-190	-190
1	26	-	856	788	842	746	-372	-450	-288	-436	-262	-385	-385
1	30	-	1068	950	985	896	-432	-500	-330	-508	-395	-509	-509
1	33.5	-	1115	1070	1100	1040	-440	-535	-350	-550	-430	-525	-525
6,000	33.5	1	1025 ± 400	835 ± 380	1000 ± 400	880 ± 380	-335 ± 200	-405 ± 180	-245 ± 110	-380 ± 225	-290 ± 150	-45 ± 110	-45 ± 110
89,000	"	"	1015 ± 400	830 ± 375	1030 ± 400	865 ± 375	-350 ± 200	-415 ± 180	-255 ± 110	-405 ± 225	-220 ± 150	-75 ± 110	-75 ± 110
266,000	"	"	1058 ± 400	845 ± 375	-	863 ± 375	-372 ± 200	-436 ± 190	-267 ± 110	-428 ± 225	-211 ± 150	-90 ± 110	-90 ± 110
427,000	"	"	1088 ± 400	867 ± 375	-	893 ± 375	-380 ± 200	-438 ± 190	-270 ± 110	-434 ± 225	-236 ± 150	-78 ± 110	-78 ± 110
670,000	"	"	1080 ± 400	860 ± 375	-	875 ± 375	-390 ± 200	-455 ± 190	-275 ± 110	-450 ± 225	-255 ± 150	-81 ± 110	-81 ± 110
755,000	"	"	1090 ± 400	865 ± 375	-	875 ± 375	-400 ± 200	-460 ± 190	-280 ± 110	-455 ± 225	-255 ± 150	-85 ± 110	-85 ± 110
1,077,000	"	"	1090 ± 400	875 ± 350	-	875 ± 350	-410 ± 190	-465 ± 175	-285 ± 100	-470 ± 210	-270 ± 140	-85 ± 110	-85 ± 110
1,460,000	"	"	1090 ± 400	865 ± 350	-	865 ± 350	-425 ± 190	-470 ± 175	-295 ± 100	-480 ± 210	-260 ± 140	-90 ± 110	-90 ± 110
1,873,000	"	"	1100 ± 400	875 ± 350	-	865 ± 350	-440 ± 190	-480 ± 175	-295 ± 100	-500 ± 210	-290 ± 140	-95 ± 110	-95 ± 110
2,224,000	"	"	1095 ± 400	860 ± 350	-	860 ± 350	-450 ± 190	-490 ± 175	-300 ± 100	-500 ± 210	-275 ± 140	-90 ± 110	-90 ± 110
2,322,000	"	"	1090 ± 400	830 ± 350	-	830 ± 350	-470 ± 190	-495 ± 175	-290 ± 100	-480 ± 210	-270 ± 140	-70 ± 110	-70 ± 110
2,618,000	"	"	1075 ± 400	830 ± 350	-	830 ± 350	-445 ± 190	-475 ± 175	-300 ± 100	-490 ± 210	-290 ± 140	-85 ± 110	-85 ± 110
2,833,000	"	"	Power Failure										
3,200,000	"	"	1080 ± 400	830 ± 350	-	840 ± 350	-470 ± 190	-510 ± 175	-385 ± 100	-500 ± 210	-290 ± 140	-90 ± 110	-90 ± 110
3,300,000	"	"	Failure										

• N - North Side of Beam

• S - South Side of Beam