Reinforced Concrete Beams Investigation
of Stresses in Web Reinforcement and
in Longitudinal Reinforcement

Theoretical and Applied Mechanics

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REINFORCED CONCRETE BEAMS:
INVESTIGATION OF STRESSES IN WEB REINFORCEMENT
AND IN LONGITUDINAL REINFORCEMENT

BY

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B. S. University of Illinois, 1912

THESIS
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I HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

LEWIS HERINGBRR FISHER

ENTITLED REINFORCED CONCRETE BEAMS: INVESTIGATION OF STRESSES
IN WEB REINFORCEMENT AND IN LONGITUDINAL REINFORCEMENT

BE ACCEPTED AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF MASTER OF SCIENCE IN THEORETICAL AND APPLIED MECHANICS.

In Charge of Major Work

Head of Department

Recommendation concurred in:

Committee

on

Final Examination
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I. INTRODUCTION.
1. Preliminary.

The question of the proper reinforcement for web stresses has long been a mooted one among reinforced concrete designers. Many theories have been advanced by various men as to the action of web reinforcement and as many different types of reinforcement have been proposed. It was with the hope of throwing some further light on this subject that the present series of tests was planned.

For a number of years tests have been made at the Engineering Experiment Station of the University of Illinois under the direction of Professor A. L. Talbot, to study the nature of web stresses in reinforced concrete beams. This matter received some attention in Bulletin No. 4 and was the principal subject treated in Bulletin No. 29. Since the publication of Bulletin No. 29, further investigation along this line has been carried on as thesis work by students of the University of Illinois, namely, Mitchell (1910) and Brooks and Haeffner (1911) and Schmitz (1912). The early work and methods were not entirely adequate, but the results are important, in that they showed where improvements in the methods of testing were required.

The first work on this line, in which the Berry type of extensometer was used, was done by Brooks and Haeffner but since that time a great deal of advancement has been made in the design and use of this instrument. It was with the idea
of measuring the stirrup stresses directly with this instrument, it is evident that if the steel stresses could be measured, a great deal of information could be gathered as to the part the stirrups play in the resisting of web stresses.

3. Scope of Investigation.

This investigation was planned to secure as much information as possible from the limited number of test specimens available (sixteen) on the action of web members in a reinforced concrete beam under load. Nine beams were made with web members inclined at 45°, two were made with vertical web members, two had two of the longitudinal bars bent up, and two had no web reinforcement. One beam was an odd one, reinforced with a Gabriel unit frame left over from previous series. In the beams with inclined web members, five different arrangements or amounts of steel were used so that some idea as to the effectiveness of various arrangements of the steel could be obtained. The two beams reinforced with vertical members were alike and give some data as to the effectiveness of vertical as compared with inclined web members. The beams with no web reinforcement and those with bent up bars enable further comparisons to be made.

In all cases where web reinforcement was used, web members were firmly welded to the longitudinal bars as shown in the detail drawings. The word "stirrup" is descriptive of and has been used generally to denote a stirrup-
like loop of same kind made from reinforcing steel and designed to hook around the horizontal reinforcing bar. Usually in practice, though not always, such stirrups have been placed vertically in the beam. To use the word "stirrup" to denote a rigidly attached web member, which in no sense resembles a stirrup, is not logical. Furthermore the use of a distinctive term for a member so rigidly attached as to become integral with the reinforcing unit assists in distinguishing the two types of web reinforcement. For this reason the term web member is used in this thesis to denote any unit of web reinforcement (whether vertical or inclined) which is so rigidly attached to the horizontal bar as to necessitate that the two act simultaneously in taking stress deformation.

3. Acknowledgment.

These tests were made as part of the research work of the Engineering Experiment Station of the University of Illinois under the general direction of Professor A. H. Talbot. The specimens were made under the supervision of Mr. D. ... ...., Associate in the Engineering Experiment Station. Mr. W. N. Slater, Assistant in the Engineering Experiment Station, and Mr. N. F. Schanerman, Instructor in T. & N. ... gave many helpful suggestions and much valuable assistance in both the making of the tests and the working up of the data. Mr. J. Montgomery, Research Fellow in the Engineering Experiment Station assisted in making the tests. Five of the beams were tested by Mr. E. A. Schmitz of the class of 1912, who did the first
work on the testing of this series of beams. The report of these tests is to be found in Mr. Schmitz's thesis.

4. Analysis or Theory.

In a reinforced concrete beam under load, there are four kinds of stress:—(1) tension in the longitudinal steel; (2) compression in the concrete; (3) bond between concrete and steel; and (4) diagonal tension. The first three of these stresses lend themselves readily to analysis if certain reasonable and safe assumptions are made, such as no tensile strength in the concrete and no slipping of the bars relative to the concrete, but any complete theoretical analysis known to the writer giving the stresses which will occur in web members due to diagonal tension involves assumptions so unreliable in character as to render the analysis of doubtful value. Because of the fact that web stresses in reinforced concrete beams do not lend themselves readily to exact analysis, the empirical method limiting the web stress by limiting the allowable shearing unit stress has come to be almost generally adopted.

(a) Stress in Beams with Web Reinforcement: - In view of the general practice of limiting web stress by limiting the vertical shear per square inch, formulas for stress in web members are derived according to which the stress is proportional to the shear. (See Bulletin No. 20, University of Illinois).

At any vertical section of the beam, the value of the resisting moment is given by the equation

\[ M = A f d \]

where \( M \) is the resisting moment, \( A \) is the total area-
sectional area of the steel, \( F \) is the area of the steel, and \( d \) is the effective depth of the beam. Differentiating the above equation, we have 
\[
\frac{dm}{dx} = \frac{df}{dx} + \frac{1}{N} \frac{dN}{dx}
\]
but \( \frac{dm}{dx} = V \) the total vertical shear on the section, then \( A \frac{df}{dx} \) represents the rate of change in the total tensile stress \( \frac{dm}{dx} \) in the reinforcement at the section under consideration. Now if \( m \) be taken as the total unit strain developed, \( \delta \) as the circumference of each bar and \( m \) the number of bars, we get \( m \cdot \delta \) as the total strain developed, or the total change in tensile stress over a unit length. Equating this to \( V \) as derived above at \( m \frac{df}{dx} = \frac{V}{\delta} \). If \( \nu \) is taken as the horizontal unit shearing stress, the shearing resistance per unit length of beam is \( h \nu \) and this equals the change in tensile stress \( m \frac{df}{dx} \) and so the final equation for the horizontal shear per square inch is \( h \nu = \frac{V}{\delta} \) and \( \nu \) as the vertical and horizontal unit shears are equal this equation gives the intensity of the vertical shear at any point.

From the equation for diagonal tension
\[
t = 2s + \sqrt{4s^2 + v^2}
\]
(see Merklin's "Mechanics of materials", page 122) we have \( t = v \) if \( s \) the horizontal tensile stress is zero. In this equation \( t \) is the maximum diagonal tensile unit stress and when \( s = 0 \) its direction makes an angle of 45° with the horizontal. The above conditions are fulfilled at the neutral axis. If the action of the web members be considered to depend upon these conditions and the further condition that the web members carry
all the diagonal tension, the following formulae are readily derived. The intensity of this diagonal tension stress is therefore the total stress in the web member which is $P = a \times s$ in which $P$ is the total stress in all web members in one section, $a$ is the distance between web members on a line normal to the direction of the maximum stress. With web members inclined at 45°, so most efficiently to resist the diagonal tension, we have $s = a \cos 45° = 0.707 a$ and $P = 0.707 b v a = 0.707 a \frac{v}{b}$. If the same total load necessarily were carried by a web member regardless of the angle of inclination, we should have the formula $P = 0.707 a \frac{v}{c} \sec \phi$ in which $\phi$ is the angle made by the web member and the 45° line. For a vertical member, we would have $P = a \frac{v}{c}$. This is the formula most used for vertical web members. It has not been considered to be anything more than an arbitrary standard for use in design. It is in error due to the fact that when the inclination of the web member varies from 45° it no longer carries the same total load. This may be seen by assuming that the web member is placed in a horizontal direction instead of a vertical direction. Again $P = 0.707 a \frac{v}{c} \sec \phi = a \frac{v}{c} \frac{1}{\sin \phi}$. This gives the same stress for a horizontal as for a vertical web member while manifestly it is impossible for the horizontal member to carry any tensile stress due directly to vertical shear.

These two equations $P = 0.707 a \frac{v}{c}$ and $P = a \frac{v}{c}$ are the ones commonly used for calculating stirrup stresses and the object of this thesis is to find out what relation exists.
between the stress as calculated above and the actual stress as found by direct measurement on the steel.

In beams reinforced with inclined web members it is readily seen that the web members can take diagonal stress for they are nearly in line with the stress and the action can be considered as analogous to that of a Pratt truss with the horizontal steel taking the place of the lower chord, the concrete above the neutral axis the upper chord, the web members the diagonals and the concrete between stirrups the verticals. The action of vertical stirrups is not so readily seen and one of the most reasonable explanations is to be found in Henschel’s “Der Eisenbetonbau” from which the main part of the following explanation was taken. Until the formation of diagonal cracks the web stresses are carried mainly by the concrete alone, but after a diagonal crack has formed, the condition will be somewhat as shown in the sketch. The compression area of the concrete

![Fig. a](image1)

A B is uncracked, the portion of the beam BCD has a crack running across it, while x and y are vertical stirrups and z represents the longitudinal steel. In this section of the beam the vertical shear is carried by three resistances. The unbroken area of concrete carries some shear and the remainder is transferred by cantilever beam action from the portion of the beam
The action as outlined above would throw practically no stress in the stirrups until the diagonal cracks had formed and the test results later indicate this to be a fact.

A consideration of the above analysis of stirrup action indicates the basis for the use of the shear as an approximate measure of the magnitude of the stress in a web member for in the case of inclined stirrups the diagonal tension stress, which according to the assumptions made is equal to the shear, is carried by the stirrup while in the case of vertical stirrups most of the vertical shear is carried directly by the stirrups after the formation of the diagonal cracks.

It should be noted in connection with web stresses that the formulas do not give the true stresses and it should be kept in mind that the actual diagonal tension unit stresses greatly exceed the stresses as calculated under the above assumptions. The diagonal stresses as found by the formula for combined stress for certain ratios of v and s are given below.
When \( s = 0 \) 

\[ t = v \]

When \( s = v \) 

\[ t = 1.52 v \]

When \( s = 2v \) 

\[ t = 2.41 v \]

When \( s = 3v \) 

\[ t = 3.30 v \]

From this it is seen that the diagonal tension per square inch may be as high as 3 times the shear, but this high diagonal stress is near the bottom of the beam, and when \( s \) is equal to or greater than \( t \), its line of action is very nearly horizontal so that the horizontal reinforcement is largely effective in resisting it. At the neutral axis the value of the diagonal tension equals that of the shear. Previous tests have shown that in the formulas for total stress in an inclined web member, if \( 2/3v \) is used in place of \( v \), a value is obtained which agrees closely with the stresses as found by experiment.

(b) Beams Reinforced with Bent-up Bars: Where part of the bars are bent up at an angle from the bottom of the beam as reinforcement for diagonal tension, a new problem arises which by its complexity defies exact analysis. The bent-up bar runs from the point of maximum tension to the neutral axis where there is longitudinal stress with only a comparatively short length in which the required bond stress may be developed. This results in a very high bond stress along the bent-up bar and if the concrete does not crack due to this bond stress acting in connection with the existing diagonal tension, local slipping of the bent-up bar is very likely to occur, this slipping is progressive and ultimately the bar will have slipped in the concrete along its full length unless some means of anchoring the
holt the upper end is used. The web action resulting in a failure of the beam by bond along the bent-up bar, although the secondary failure may be by diagonal tension since if the bar slips, it cannot resist the diagonal stresses. If the bond stress acting in conjunction with the diagonal tension is sufficient to crack the concrete before the bond failure takes place, the whole diagonal stress is then thrown into the bent-up bar and this stress must be taken off by bond in the upper portion of the beam necessitating either a large embedment or a firm anchorage of the bent-up bar.

Bond stress is also likely to be troublesome in the case of the straight bars. Due to the fact that part of the bars are bent-up, the stress in the remaining bars stays nearly constant till near the end of the beam and again we have a large stress which must be taken off in a short distance necessitating if failure is to be prevented, a firm anchorage or a large embedment of the straight bars.

(c) Beams Without Web Reinforcement:—In beams without web reinforcement, all diagonal tension is carried by the concrete alone. This diagonal stress is found directly by the formula given in Merriman's "Mechanics of Materials" and when the diagonal stress exceeds the tensile strength of the concrete, failure soon occurs. Failures of this kind are likely to occur very suddenly and without warning.
II. MATERIALS, TEST PIECES, APPARATUS, and METHODS OF TESTING.
5. Materials.

The materials used in making the test pieces were of a grade much used in this section of the country and in this regard the quality of the concrete may be considered as representative of that to be expected in construction work. The materials with the exception of the steel were of the same lots as those from which the 1911-12 test pieces were made.

The Sand:—The sand used was a good quality of well graded Wabash River sand from Attica, Indiana, and was bought in the open market. The following table gives a mechanical analysis of the sand.

**TABLE I.**

**MECHANICAL ANALYSIS OF SAND.**

<table>
<thead>
<tr>
<th>Sieve Number</th>
<th>Per Cent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>100</td>
</tr>
<tr>
<td>5</td>
<td>90.9</td>
</tr>
<tr>
<td>10</td>
<td>69.1</td>
</tr>
<tr>
<td>12</td>
<td>63.8</td>
</tr>
<tr>
<td>16</td>
<td>58.5</td>
</tr>
<tr>
<td>18</td>
<td>48.4</td>
</tr>
<tr>
<td>30</td>
<td>31.1</td>
</tr>
<tr>
<td>40</td>
<td>19.5</td>
</tr>
<tr>
<td>50</td>
<td>6.5</td>
</tr>
<tr>
<td>74</td>
<td>2.9</td>
</tr>
<tr>
<td>150</td>
<td>0.9</td>
</tr>
</tbody>
</table>
The Stone: - The analysis given in Table II represents the average results obtained from five samples taken at intervals during the season of 1911-12. The stone used was a good quality of hard lime stone from Kankakee.

Table II.
MECHANICAL ANALYSIS OF STONE.

<table>
<thead>
<tr>
<th>Sieve Number</th>
<th>Per cent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/4</td>
<td>95.5</td>
</tr>
<tr>
<td>1/2</td>
<td>86.7</td>
</tr>
<tr>
<td>3/8</td>
<td>46.3</td>
</tr>
<tr>
<td>5</td>
<td>35.0</td>
</tr>
<tr>
<td>8</td>
<td>8.1</td>
</tr>
<tr>
<td>10</td>
<td>3.4</td>
</tr>
</tbody>
</table>

The Cement: - The cement used in making the beams was Universal Portland Cement, and was of the same lot as was used in making the test pieces of the 1911-12 tests. Table III gives the average of fifteen tests made in the cement laboratory of the University of Illinois on samples of this cement taken at intervals during the season of 1912-13.

Table III.
TENSILE STRENGTH OF CEMENT.

<table>
<thead>
<tr>
<th>Age when tested</th>
<th>7 Days</th>
<th>28 Days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixture</td>
<td>Heat</td>
<td>1:2</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>595</td>
<td>207</td>
</tr>
</tbody>
</table>

The Steel: - Medium steel was used in all the beams.
All of the beams in this series, fifteen in number, were made from 10 feet 6 inches long, 17 inches deep and 8 inches wide, the other beam No. 7 was 1 foot 6 inches long, 18 inches deep and 6 inches wide. The details of the beams are shown in Figures 1 to 4.

All of the large beams had longitudinal reinforcing consisting of four 3/4 inch bars and all longitudinal bars which extend the full length of the beam were hooked at the ends, being bent through 180° on a radius of 4 inches. Beams
No. 301.1-2 had reinforcing bars which extended the full length of the beam; beams No. 302.1-2 had two of the longitudinal bars bent up as shown in Figure 1, for web reinforcement; beam No. 303.1 had in addition to the longitudinal rods, eight 1/4 in. plain round rods in each end of the beam arranged as shown in Figure 5. Beams No. 304.1-2 were similar to beam No. 302.1 except that there were 16 rods in each end of the beam (see Figure 3); beams No. 305.1-2 had sixteen 3/8 in. plain round rods in each end of the beam (see Figure 3); beams No. 306.1-2 were reinforced with 16, 3/8 in. plain round rods in each end of the beam, seven diagonal rods being attached to each outside longitudinal rod as shown in Figure 2; beams No. 307.1-2 were similar to beams No. 305.1-2 and beams No. 306.1-2, except that the diagonal rods were of 3/4 in. diameter. Beams No. 308.1-2 had vertical web members, the details of which are shown in Figure 4. Beams 309.1 and an odd one and were reinforced with a Gabriel unit frame having five deformed bars of elliptical section as longitudinal reinforcement and 3/4 in. bars were looped around the bars on 1/2 in. spacing as web reinforcement. In all beams with the exception of No. 301.1 in which the Gabriel unit frame was used, the connection between the longitudinal rods and the web members was made with an Oxy-Acetylene weld so that there was no possibility of the stirrup slipping along the longitudinal rod.

7. Making of Specimens.

The beams were made in "knock-down" forms on the
Close to the face of the laboratory, being protection from the floor to the floor by a strip of building paper. In placing the steel in the forms, care was taken to locate it as nearly as possible in the position shown in the detail. The longitudinal steel was set in blocks of wood, bringing it to the proper distance above the bottom of the beam, and the web reinforcement was wired and otherwise braced so as to be kept in position during the pouring of the concrete. For a short 1/4 in. in diameter were tacked to the face of the place. Where gage points were to be located ed in to save much of the chipping necessary to reach the steel. Unfortunately these were not a little too small so that nearly all holes had to be chipped to admit the leg of the extensometer. After the steel and gage had been placed, the concrete was poured. In placing the concrete, care was taken to get it around all the steel and it was well puddled to prevent the formation of pockets of any kind. Two batches of concrete were made for each beam and from this, three cubes, one cylinder, and one control beam were made and numbered the same as the corresponding beam.


The concrete was allowed to set seven days before the removal of the forms. After the removal of the forms, the beams were allowed to stand in the place in which they were cast until tested. The temperature of the room in which the beams were cast did not fall below freezing at any time. The control beams were stored in air under the same conditions as the beams but the cubes and cylinders were stored in damp sand.

The most important feature of making a beam ready for testing lies in the preparation of page holes. The gage line is the length between a pair of gage holes and is the length over which the deformations are measured. The page holes were small holes .055 inch in diameter drilled into the steel reinforcing. In order to expose the steel so that gage holes might be drilled, corks were attached to the forms in the proper places. On the removal of the forms, these corks stayed in the concrete and were withdrawn at the desired time, leaving a hole in the concrete which exposed the steel.

After the drilling of the gage holes, the specimens were whitewashed before being placed in the testing machine, in order that the cracks might be much easier found and marked. The beams were placed in the machine, carefully centered, and at all load points and points over supports, iron plates set in plaster of paris were used to distribute the bearing stresses. In setting the plates, care was taken to see that all plates were firmly bedded and also that the beam stood plumb in the machine.

The beams were tested upside down in order that the gage lines on the longitudinal reinforcement could be more easily reached. As some of the beams had gage lines on the bottom, it was necessary to apply the load through large cast-iron rocker bearings as can be seen in Figure 5, which gives a general view of the method used in applying load. All beams were tested with third-point loading, the increments of
Sketch Showing Method of Applying Load.

18" I Beam

Specimen

Rocker Bearing

100° 3:4

Fig. 5
Load being varied so that five sets of readings could be taken before failure. At each increment of load, deformation readings were taken on all gage lines, each recorded reading being the average of five readings on the gage line. Before applying any load, two sets of zero readings were taken for all gage lines and the average of these was used as the zero reading. At each increment of load, all cracks were carefully marked on the beam.

For a more general description of the Berry type of extensometer and its use, reference is made to a paper by Willis A. Slater and Herbert F. McCre, written for the 1915 meeting of the American Society for Testing Materials on "The Use of the Strain Gage in the Testing of Materials" and to Bulletin No. 64, University of Illinois. The methods outlined in these papers were followed in these tests.

In the numbering of gage lines, a uniform system was used. Beginning at the south end on the east side of the beam set in the machine, all gage lines on the longitudinal steel were numbered from 1 continuously to the north end of the beam, while all gage lines on the stirrups were numbered from 100 up. On the west side of the beam, beginning at the south end, the gage lines on longitudinal steel were numbered from 30 up, while the gage line numbers on the stirrups started with 200.

All beams were tested in the 200,000 pound Olsen beam testing machine. The method of applying the load is shown in Figure 5.
III. DATA AND DISCUSSION.
10. **Description of Curves, Tables, Figures, etc.**

Figures 1 to 4 are drawings showing all details required for the construction of the beams and the insertion of gage lines. Figure No. 5, which is self-explanatory, is a sketch showing the method of applying lines to the specimen.

Figures No. 6 to 11 are photographs taken of the beams after they had been tested and removed from the machine. In order to get photographs which would show the cracks and other features of the beam, all cracks were gone over with black paint and, therefore, showed in much greater prominence than they did in the unpainted beam. These photographs show the nature of the cracks and figures at certain points along their length show the load in thousand of pounds at which the crack had reached the point indicated. Such information can be gained from these photographs as to when and how cracks form. Photographs were taken of the beams tested by Schmitz in 1913 and are included here as a matter of record.

Table I is a general table giving the data on all beams. The first column gives the number of the beam followed by the amount of web reinforcement in each end of the beam, the third column gives the age of the beam in days at the time of the test; then come a series of columns giving the maximum measured stress, the calculated stress and the location of the steel for the various loadings. The calculated stress in the longitudinal steel is the calculated stress for the point of maximum moment while the measured stress is the maximum of all the stresses measured on the longitudinal reinforcement. The maximum stresses measured
In working up the data of these tests, two complete sets of curves were plotted for each beam. The curves of the first set were load-stress curves for each gage line; that is, the total load on the beam was plotted as ordinates, while the resulting stress in the steel at each gage line was plotted as abscissas. These curves are especially valuable in determining the rate at which the steel takes stress with increase of load. A second set of curves was plotted with the stress in the steel as ordinates and the location of the gage line on the beam determining the abscissas. These curves show graphically the relation of the stress to the location of the gage line. On the same sheet as the second set of curves, are sketches showing the location of the cracks which developed together with the load at which they made their appearance, much valuable information on the reasons for high local stresses can be gained by a study of the location of the cracks with reference to the location of the points of high stress.

One sheet of curves gives the load-deformation curves for the test cylinders and gives an idea as to the quality of the concrete. The average value of the Modulus of Elasticity is 2,700,000 and in the calculations for stresses, a value of n of 11 was used.
11. Comparison of Results.

It is unfortunate that these beams did not contain more longitudinal steel, as eleven of them failed by tension in the steel without developing the full strength of the web reinforcement. But some interesting conclusions can be drawn from a study of these results.

As might be expected, the beams without web reinforcement failed by diagonal tension at a calculated vertical shear of 116 pounds per square inch. This value agrees very well with previous tests and is an indication of what can be expected of concrete alone. The beams with bent-up bars carried considerable more load than did those with straight bars only, and developed an average vertical shear of 167 pounds per square inch. According to Schmitz, these beams failed by bond along the bent-up bars and he states that if the bent-bars had been anchored, the beams might have carried much more load. This is a reasonable conclusion and emphasizes again the great importance of properly anchoring all bars when bent-up rods are used as web reinforcement because of the high bond stresses developed with this type of web reinforcement.

Of the eight beams reinforced with inclined stirrups, all but one failed by tension in the steel at a load of approximately 40,000 pounds. Beam No. 203.1 failed by diagonal tension and developed the full strength of the stirrups before failure. The steel in the stirrups passed beyond the yield point. The results obtained from these beams are discussed later, but it is well to note here that loads at failure were fairly uniform with the exception of Beam No. 203.1
<table>
<thead>
<tr>
<th>Beam No.</th>
<th>Reinforcement</th>
<th>Age da.</th>
<th>Maximum Measured Steel Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Location of Steel</td>
</tr>
<tr>
<td>301.1</td>
<td></td>
<td>225</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>301.2</td>
<td></td>
<td>215</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>302.1</td>
<td>2.34 Bent-up Bars</td>
<td>73</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>302.2</td>
<td></td>
<td>84</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>303.1</td>
<td>8/4 Inclined Stirrups 10° c. to c.</td>
<td>238</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>304.1</td>
<td>16/4 Inclined Stirrups 10° c. to c.</td>
<td>225</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>304.2</td>
<td></td>
<td>225</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>305.1</td>
<td>8/4 Inclined Stirrups 10° c. to c.</td>
<td>120</td>
<td>Long. Stir.</td>
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<td>305.2</td>
<td></td>
<td>90</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>306.1</td>
<td>14/4 Inclined Stirrups 5° c. to c.</td>
<td>235</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>306.2</td>
<td></td>
<td>234</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>307.1</td>
<td>16/2 Inclined Stirrups 10° c. to c.</td>
<td>300</td>
<td>Long. Broken in handling</td>
</tr>
<tr>
<td>307.2</td>
<td></td>
<td>227</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>308.1</td>
<td>6/8 Vertical Stirrups 6° 3/4 c. to c.</td>
<td>80</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>308.2</td>
<td></td>
<td>226</td>
<td>Long. Stir.</td>
</tr>
<tr>
<td>309.1</td>
<td>5/8 Continuous (see sketch)</td>
<td>300</td>
<td>Long. Stir.</td>
</tr>
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</table>

Note:
Small figures in upper lefthand corner give load in thousands of pounds if increments used were different from heading.
<table>
<thead>
<tr>
<th>Load Calc.</th>
<th>Meas. Calc.</th>
<th>Maximum Load</th>
<th>Cubes</th>
<th>Cylinders</th>
<th>Control Beams</th>
<th>Tested by</th>
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<tr>
<td>1000</td>
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<td>D.T.</td>
<td>2910</td>
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<td>40,000</td>
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<td>33.800</td>
<td>169</td>
<td>Bond</td>
<td>2010</td>
<td>E.A.S.</td>
<td></td>
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<td>165</td>
<td>Bond</td>
<td>2620</td>
<td>E.A.S.</td>
<td></td>
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</table>

**Note:**
In the column of calculated stirrup stresses, figures above the line are based on \( V \) and those below the line on \( 2/3V \).
which failed in tension at a load of 30,700 pounds. The explanation of this failure is apparent, that being an unusual number of small cracks at the point of failure which may indicate a region of rather poor concrete and, therefore, a lack of the usual assistance to the resisting moment given by the tension in the concrete. The average shearing unit stress developed by these beams was 194 pounds per square inch.

Beams No. 301-3 were reinforced with vertical web members and carried an average load of 4,300 pounds more than did the beams reinforced with inclined stirrups. The average shearing unit stress developed by the beams was 215 pounds per square inch.


As would be expected, the beams reinforced with longitudinal steel only, failed at a comparatively low load by diagonal tension in the concrete. While the beams reinforced for web stresses with (or) bars bent up carried considerably more load than those without such reinforcement, they carried very much less load than those beams well reinforced for web stresses, and failed by bond along the bent up bars. The other beams were all reinforced for web stresses by means of either inclined or vertical web members and all except No. 300.1 and No. 300.2 failed by tension in the steel at a load of approximately 40,000 pounds. Beam No. 300.1 was very lightly reinforced in the web and as would be expected, failed by diagonal tension across the web members. All failures were in line with deduction made from the results of previous tests.
Fig. 6

Beams No. 8, (306.1)

"  " IT, (307.2)

W.  I4, (301.1)
Fig. 7

Beam No. 13, (304...)  
" " 15, (304.L)  
" " 16, (301.L)
Fig. 8

Beam No. 7, (309.1)
" " 18, (307.2)
" " 9, (303.1)
Fig. 8

Beam No. 4, (302.2
" " 7, (300.10
" " 1, (302.1)
Fig. 10

Beam No. 5, (CON. B)

" " 2, (CON. I)
Fig. II

Showing failure of beam without web reinforcement
and no unusual or unexpected features were present.

The following are notes under each beam. The location of cracks, type, line, and in which, may be found by reference to the curve sheets.

**Beam No. 203.1.** The first cracks to show up were short tension cracks in the middle portion of the beam. These cracks extended higher with each succeeding increment of load and at failure were short of the neutral axis of the beam.

**Beam No. 203.2.** showed no evidence of diagonal tension cracks until a load of about 20,000 pounds was reached. At 22,000 pounds, a small diagonal would have occurred at the north end of the beam and with the load of 24,900 pounds was reached, this crack rapidly extended in both directions from near the central axis of the beam failed by diagonal tension at a maximum load of 24,650 pounds.

**Beam No. 231.1.** showed at the north end, evidence of diagonal tension cracks at the low beam, toward the load point with each added increment of load, and the receiving a great deal of attention from the observers since the beam suddenly failed by diagonal tension at the south end of a place where there had been no evidence of diagonal cracks before the failure. Maximum load carried 24,200 pounds.

**Beam No. 203.1.** This beam gave the only diagonal tension failure among all the beams with web reinforcement. The diagonal tension cracks opened up directly above the web members which failed in tension, the steel being stressed beyond the elastic limit. Very few cracks showed up until a
A load of 10,000 pounds was reached, at which load tension cracks opened only a large portion of the flange of the beam and at the south end diagonal tension cracks had opened between two web members. At a load of 24,000 pounds a very small diagonal crack was noted at the north end of the beam, while at the south end a fine diagonal crack cutting across two stirrups had opened up. At 32,000 pounds the flexure at the north end had extended and the extensometer readings show that some of the web steel in the south end had been stressed beyond the elastic limit. Final failure occurred at 38,700 pounds by diagonal tension.

Beam No. 304.1-2. Both of these beams failed by tension in the steel, No. 304 near north load point, and No. 304.2 near south load point. The results of the tests on these similar beams are very consistent, there being a difference of only 700 pounds in the ultimate loads.

No. 304.1 carried a maximum load of 59,500 pounds, while No. 304.2 carried 40,200 pounds. In both beams, the flexure of any deflection until a load of 24,000 pounds was reached although at this load, No. 304.2 showed considerably greater tendency to crack across the web members than did No. 304.1. These cracks had extended a great deal at a load of 38,700 pounds and in both beams maximum stresses in web members of 35,000 to 40,000 pounds per square inch were noted. Before failure could occur by diagonal tension, the longitudinal steel failed giving in both beams tension failures at the above mentioned various loads.
Beam No. 306.1-2. Up to a load of 16,000 pounds, no cracks were noticeable. At 16,000 pounds load, tension cracks were quite prevalent throughout the central portion of both beams and these cracks extended higher with each increment of load. Beam No. 306.1 showed a few small diagonal tension cracks at 14,000 pounds load and at 28,000 and 40,000 pounds load; and a number of diagonal cracks at each end of the beam. These diagonal cracks did not open and the stresses in the web steel did not yet above 27,000 pounds per square inch. Failure occurred by tension in the longitudinal steel at a load of 40,000 pounds. Beam No. 306.2 failed unexpectedly by tension in the longitudinal steel at a load of 36,050 pounds, the steel under the south load, but on the east side of the beam being stressed beyond the elastic limit. The only apparent explanation for the low load of failure lies in the fact that on both sides of the point of failure, little or no cracking of the concrete was visible and it seems possible that the failure occurred at a point of concentration of deformation due to a patch of poor concrete.

Beam No. 307.2. Beam No. 307.2 failed by tension in the longitudinal steel at the center of the beam under a load of 40,780 pounds. A very few small cracks opened across the web members in which the stress did not yet above 10,000 pounds per square inch. This beam was apparently over-reinforced against diagonal tension.

Beam No. 308.2. Beam No. 308.2 was apparently in very poor shape for testing when placed in the machine, having
been broken across the middle by concrete and nearly one-half the concrete below the steel knocked off in chipping away leaves; nevertheless this beam carried a heavier load than any other one in the series. Several small cracks opened up across the web members, but they were very small and the strains in these members did not get above 15,000 pounds per square inch. Failure occurred apparently in the longitudinal steel at the center of the beam under a load of 44,600 pounds.

Beam No. 3091. This beam was very highly reinforced and carried the highest shearing unit stress of any in the series. Very few diagonal cracks opened up and they failed apparently by compression in the concrete at the center of the beam. It is doubtful if this was a case of a pure compression failure as the longitudinal steel passed the yield point at about the same time the concrete did, but we load 37,500 pounds.

10. Action of Web and Members.

As was indicated in the analysis of web stresses, there is no very reliable method of computing stresses in web members, nor is there any general agreement as to how such members take stress. It is hoped that a study of the results will help to or at least alleviate uncertainty.

The curves have been run on the basis of stress varying greatly from the web members, being greatest at the bottom of the beam and least at the top. This will true except where the webs are extremely narrow. This fact
and the load of 10,000 pounds in tension, the load stress curves for the web members show the initial axis (lower two gage lines) would be parallel to the approximately parallel with the middle of the beam. The calculated vertical load at this point is 150 pounds per square inch and the unit of the ultimate diagonal tension, according to the theory of stress distribution, in case of homogeneous material, would be as the maximum stress the concrete per square inch. If this stress exceeds the strength of the web members, the web members would resist the diagonal tension and that the load stress curves would be approximately straight lines and parallel. This is exactly the case as shown in these tests and leads to the conclusion that the diagonal web members resist diagonal tension directly.
If the web members are designed to carry the stresses thus thrown into them, it would appear that the diagonal cracks would be small, well distributed, and easily extend above the neutral axis of the beam. If, on the other hand, the stresses in web members become sufficiently large, deformation results. It would seem probable that large cracks would open, diagonal stresses from a large area would be concentrated in one place, and the stresses in web members of the cracked region, would be high.

From the above, it would seem to result that in order to get the best efficient service from web members, they should be well distributed in the beam so as to prevent as far as possible, the formation of large diagonal cracks. Then that diagonal web members resist only stresses directly and after the diagonal tension exceeds the strength of the plain concrete, the portion of the member below the neutral axis takes stress in direct proportion to the load and carries practically all the web stress. If slipping does not take place, the portion of the stirs which lies in the compression area of the beam receives but little stress and forms an efficient anchorage for the lower portion of the stirrup.

Beam No. 303.1 was reinforced with eight 1/4 in. round web members. This beam failed in almost the same manner as would a beam without web reinforcement. There was not enough steel to prevent the formation of one large diagonal crack across the region of greatest stress and failure resulted by diagonal tension while Beam No. 304.1-2 which was reinforced with sixteen 1/4 in. web members in each end failed by tension in the.
longitudinal steel. The beams were reinforced with sixteen 3/6 in. web members in each end. Beams No. 305.1-2 were reinforced with fourteen, 3/6 in. web members in each end. These two beams are compared in Table 5, where the advantages to be gained by distributing the steel over the entire cross section of the beam are shown.

While both beams contained enough total steel to develop the full flexural strength of the beam, the stresses in the web members in Beam No. 306.1-2 were considerably higher than those found in Beam No. 305.1-2 and in all cases were the stresses calculated by the formula. While No. 305.1-2 not only had lower stresses than No. 306.1-2 corresponding, but in all cases the stresses were lower than the calculated ones.

Beam No. 307.2 was probably over-reinforced against web stress considering the amount of longitudinal steel. The stresses in the web members were small, not exceeding 15,000 pounds per square inch, and very few cracks opened up. These were small and did not extend very far. It is evident also that the maximum measured stress in all cases exceeded somewhat the calculated stress.

Several other points brought out in these tests are worthy of note. The web members near the support and those near the load point are not as highly stressed as those midway between. This fact is partly explained, at least, by
the fact that in cases when the web members are not in vertical stress, cracks are formed and the reduction of strength occurs, is supported by the experimental action of the beams over the support and over the load. Both of these sections being subject to very small, or no, direct stress.

A point of great interest to engineers is the question as to whether the stresses are actually calculated as high as those found in the test. Although in the opinion of the writer the formulas as derived are given under "Analysis" are but rough approximations and in all cases of the sections which actually take place, any other law than an over calculated stresses which correspond very closely with those found by experiment and in the full vertical beam used in calculation, the stresses as calculated will be on the safe side, and in the maximum stresses as assumed over cracks did not in general exceed the calculated stresses. For working stresses, the formulas give values higher than were found in the test. But the case of web member.


In view of the fact that only two beams having vertical web members were tested, the results as obtained can hardly be considered as conclusive for stresses. In conclusion, in inclined web members, which develop stress directly, vertical members do not take such stress with the load in tension, which corresponds closely to the maximum in web beams without web reinforcement. In most of these beams, a number of diagonal cracks are formed and parallel crack failure was
The measured stresses are consistently below the calculated stresses, indicating that the actual values of the measured stresses will be considerably below the calculated.


Although the fact that the non-welded members in these tests developed their full calculated strength and eluded immediate failure indicates that such members will always act thus. It must be borne in mind that in all cases, these were firmly secured to the longitudinal steel and were well anchored in the heart of the beam at the ends and all the conditions were favorable for the best action. Vertical web members are used generally in practice, and are seldom firmly attached to the longitudinal steel. It would appear advisable if the full strength and efficiency of the stirrups is desired, to design them so that they will be firmly attached to the longitudinal steel and will
The average of the stresses in the longitudinal steel over the central portion of the beam according to the theory, should have been constant, but were generally slightly less at the center.

Schmitz found that the maximum steel stresses were about the calculated values. This fact is apparently confirmed by these tests, and the reason for this probably lies more in the values of calculation and in the method of selecting the measured stresses than in any inadequacy...
in error in the formula. The formulas used with the calculations were based upon a straight line load stress curve, whereas the actual load stress curve could not be a straight line because the load increases the smallest and is lowest, thereby decreasing the moment arm of the resisting moment, which results in a higher steel stress than would be found by using the straight line formula and a constant value for \( K \).

The measured stresses selected for comparison with calculated stresses were the largest found on anygage line. In most cases averaging this with the stress on the opposite side of the beam would have given a value agreeing reasonably well with that given by the formula.

17. Non-Uniformity of Stress.

One striking feature of the results of these tests was the frequent wide variation in the measured stress on the two sides of the beam. For instance in Beam No. 301.1 on the west side of the beam, the stress ran consistently from 3,000 to 5,000 pounds per square inch higher than the stresses measured at corresponding gage lines on the east side of the beam. In Beam No. 301.2 there is no such uniformity. In Beam No. 304.1 the stress in the longitudinal steel are fairly uniform on both sides of the beam with the east side stress a little higher. Instances can be found in every beam in which the stress in the longitudinal steel varies considerably on the two sides of the beam. This same difference is noticeable in
The stresses in web members caused by differences of 5000 to 10,000 pounds in the members are not at all uncommon.

This difference in the longitudinal steel when it is consistent (that is, there are more in one case than the other) cannot probably be entirely accounted for at least, by the fact that in setting and such work is done in centering and setting the beam up in the machine, there is always some movement of loading or bending to occur, but this does not account for a difference which is not consistent, and is in favor of one side of the beam. It seems possible that this variable difference is accounted for by the fact that the concrete is not perfectly homogeneous. That is, there is a variation in the strength of the concrete through the beam and this variation in the concrete leads to movement in the transferring of stress and also to irregularities in the way cracks are formed. These variations tend to make a variation in the stresses which come to the steel. This emphasizes the fact that whenever concrete is used, the nature of the material must be taken into consideration, and consideration always given to the fact that reasonable variations in the material must be allowed for.

18. Inclined vs. Vertical web members.

Although only two beams in this series were reinforced with vertical web members, nevertheless some tentative comparisons between inclined and vertical web members can be made from the results of those tests. Inclined web members take stress much earlier than do vertical web members, provided the same
preventing the opening of diagonal cracks, but this does not lead directly to the conclusion that the inclined ones, inclined web members are better than vertical ones. In fact in this respect of loads, the beams reinforced with vertical web members carried the highest loads by over 1,000 pounds. It would seem reasonable, in fact, that if the beams were designed one having inclined and one vertical web members, and both sets of members capable of preventing web failure, that the beams with vertical stirrups would carry the highest loads if failure were to occur in both sets by tension in the steel, as the inclined web members would be exerting a direct pull on the lower inclined steel. This extra stress would be the horizontal component of the vertical tensile stress in the web member due to the bending force and only the horizontal component of the total stress in the web member which is made up of diagonal web and flexural stresses combined. This is partly borne out by the results of these tests and generally supports the higher loads carried by the beams reinforced with vertical web members.

Referring to Table 1, it can be seen that for corresponding loads, the maximum measured stress in the horizontal tension steel for beam No. 307.0 is equal to 1.58 times the maximum measured stress in beam No. 308.1 for No. 308.1 is not considered, as it was in very bad shape when tested, having broken through in handling. These two beams carried members with the same amount of web reinforcement, i.e., with a 11 of web members. 
The maximum measured stresses in Beam 1 show that the stress difference is greatest in a region of large strains and the amount of web reinforcement.

10. Influence of Cracks in the Concrete.

The very marked connection between cracks and local high steel strains is well or hinted at by the former sheets during elevations of the beam with the loads plotted on them. For instance, in Beam No. 208.1 under the north load point, there is a difference of 16,000 pounds for the left in the stresses in the web of the beam and region of the steel on both sides of the point of crack (Fig. 7). This difference can be accounted for by the fact that the point of maximum stress in a cracked area while those of low stress are in large, large areas of uncracked concrete.

Similar instances of such a point of high stress can be found in almost any sheet of curves of strained in web members. Just for example, take an 808.1 beam with the large increase in stress at a beam of 20,000 pounds in the bottom gage line of the stirrup, immediately to the left of the left load point, a place at the elevation curve shows that at 20,000 pounds, a crack forms which runs across this stirrup, which accounts for the high stress. Also note the large increase in stress in the gage line second from the bottom of the beam and second from the left end for a load of 20,000 pounds. An explanation is at hand. Here it is seen that at this load the large diagonal crack opened up across this web line at 20,000 pounds load.
This beam is given a separate consideration, as it varies so widely from the others that no reasonable generalization can be made. This beam was reinforced with 1.57 per cent of corrugated longitudinal steel and had 1/4"/wire looped and twisted around the bars in shape of the Morse chart. The web reinforcement was practically equivalent to some theoretical area to ten 1/4" rebar in each 1/2" length, making a very highly reinforced beam. The extremely high vertical shear carried by this beam is noteworthy and illustrates the results which can be obtained by highly reinforcing the web of comparatively short, shallow beams. It seems more as an illustration of what is possible with reinforced concrete than as a method of practical construction or of conditions which will be met in practice. The vertical shearing unit stress of 572 pounds per square inch, developed by this beam is the highest value ever known in similar tests on rectangular beams so far as the writer's knowledge goes.


The foregoing discussion may be summed up in the following conclusions:

I. Properly designed web members are effective agents for the resisting of web stresses.

II. Inclined web members take stress at earlier loads than do vertical members, but they have not been shown to be more effective in preventing slip to web failures. Further tests are required to settle this question.
III. Inclined web members are more effective than vertical members in preventing the opening of concrete cracks.

IV. Vertical web members do not act appreciably until the concrete has failed in tension or shear.

V. The usual method based on the full value of the vertical shear of calculating stresses in web members gives approximately correct values for inclined members, but the measured stresses for vertical members are only about one-half the calculated stresses.

VI. Welding, or otherwise rigidly attaching the web members to the longitudinal steel greatly increases their effectiveness.

VII. Large variations in stress across a section of a beam are likely to occur.

VIII. The unusually high shearing unit stress developed by beam No. 309.1 is an indication of the strength which may be obtained by a high percentage of well anchored web reinforcement.

IX. The methods of testing used were very satisfactory and the results obtained prove conclusively the value of this method of investigation for web stresses.

X. Further tests along this line are desirable in order that more may be learned of the action of vertical web members. Tests with beams using a higher percentage of longitudinal steel so as to develop the full strength of the web reinforcement would be expected to yield interesting results.
Gage Line Numbers

East Side

West Side

S

303.1

307.2

S

N

N

West Side
Gage Line Numbers

301.1

East Side

West Side

301.2

East Side

West Side
Beam No. 301.2

Load in 10,000 lbs.

20,000

10,000

Load in 10,000 lbs.

20,000

10,000

Unit Stress 1 in 20,000 lbs.
Beam No. 301.2

Unit Stress $1^2 \times 20,000$ lbs.
Beam No. 303.1

- Top row of gage lines
- 20 ft
- 30 ft
- Bottom

30,000 lb
20,000 lb
10,000 lb
5,000 lb
0 lb

32,000 lb
26,000 lb
16,000 lb
8,000 lb
0 lb

Stirrups
Beam No. 3031

Load vs. Unit Stress

Load: 10,000 lbs, 20,000 lbs, 30,000 lbs

Unit Stress: 1 lb/2,000 lbs

Tests 24, 25, 26, 27, 28, 29, 30, 31, 32, 33
Beam No. 304.2

Applied Load in lps

10,000  20,000  30,000  40,000

39  40  44  45  48  49  50  57

Unit Stress - 1 to 20,000 lbs
Beam No. 304.2

Applied Load in lbs.

10000 124 125 127 128 129 130

Unit Stress - 1'20,000 lbs.
Beam No. 3061

- Top row of gage lines
- 2nd
- 3rd
- Bottom

4000 lb

36000 lb

24000 lb

16000 lb

8000 lb

Stirrups
Beam No. 3062

--- Top row of gage lines
--- 2nd
--- 3rd
--- Bottom

20000
10000
0

32000 lb
24000 lb
16000 lb
8000 lb

Stirrups
Beam No. 306.2

Unit Stress - lb/sq.in.

Longitudinal Steel
Beam No. 306.2

Applied Load in lbs

10,000

20,000

30,000

112 113 114 115 116 117 118 119 120 121 122 123

30,000

20,000

10,000

Unit Stress - \( f = 20,000 \) lbs.
Beam No. 3072

Applied Load in lbs

Unit Stress - 1" = 20,000 lbs.
Beam No. 307.2

Applied Load in lbs.

10,000

20,000

30,000

40,000

49 50 51 52 53 54

Unit Stress - \( \tau = 20,000 \text{ lbs.} \)
Beam No. 307.2

Applied Load in lbs.

10,000
20,000
30,000
40,000

100 101 102 103 104 105 106 107 108 109 110

Unit Stress - 1\*2,000 lbs.
Beam No. 308.2

- Top row of gage lines
- Middle
- Bottom

Stirrups
Beam No. 308.2

Applied Load in lbs

10,000 132 133 134 137

40,000 201 202 203 204 205 206 207 208 209 210 211 212 213 214

Unit Stress = \tau = 20,000 \text{ lb/}

in
Beam No. 308.2

Applied Load in lbs.

Unit Stress = 1" 20,000 lb.
Beam No. 7 (3091)

Applied Load in lbs.

10,000
20,000
30,000
40,000

Unit Stress - 1" = 20,000 lbs.
Cylinders
Load-Deformation Curves

Unit Stress - \( \frac{\text{lb}}{\text{in.}^2} \)

Unit Deformation - in.
**BEAM NO. 301.1**

**STRESSES IN THOUSANDS OF LBS. PER SQ. IN.**

(Compression marked minus.
Tension not marked)

<table>
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<th>Gage Line</th>
<th>Load in thousands of pounds</th>
<th>Gage Line</th>
<th>Load in thousands of pounds</th>
<th>Gage Line</th>
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## BEAM NO. 301.2

**STRESSES IN THOUSANDS OF LBS. PER SQ. IN.**

(Compression marked minus
Tension not marked)

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**Notes:**
- Compression marked minus.
- Table shows stresses in thousands of lbs. per sq. in., and load in thousands of pounds.
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### BEAM NO. 303.1

**STRESSES IN THOUSANDS OF LBS. PER SQ. IN.**

(Compression marked minus.
Tension not marked)

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BEAM NO. 304.1

STRESSES IN THOUSANDS OF LBS. PER SQ. IN.
(Compression marked minus.
Tension not marked)

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### BEAM NO. 304.1

**STRESSES IN THOUSANDS OF LBS. PER SQ. IN.**

**Compression marked minus.**

**Tension not marked**

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<th>West Side Load in thousands of pounds</th>
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### Beam No. 304.2

**Stresses in Thousands of Lbs. Per Sq. In.**

 *(Compression marked minus.

Tension not marked)*

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**BEAM NO. 304.2**

**STRESSES IN THOUSANDS OF LBS. PER SQ. IN.**

(Compression marked minus. Tension not marked)
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(Compression marked minus.
Tension not marked)

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# Beam No. 306.1

**Stresses in Thousands of lbs. per sq. in.**

(Compression marked minus.
Tension not marked)

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## BEAM NO. 306.2

**STRESSES IN THOUSANDS OF LBS. PER SQ. IN.**

(Compressions marked minus. Tension not marked)

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### Table: Stresses in Thousands of Lbs. per Sq. In.

#### Beam No. 306.2

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<th>West Side</th>
<th>Average</th>
<th>East Side</th>
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#### Load in thousands of pounds

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(Compression marked minus, Tension not marked)
**BEAM NO. 307.2**

**STRESSES IN THOUSANDS OF LBS. PER SQ. IN.**

*(Compression marked minus. Tension not marked)*

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</tbody>
</table>
**BEAM NO. 307.2**

**STRESSES IN THOUSANDS OF LBS. PER SQ. IN.**  
(Compression marked minus.  
Tension not marked)

<table>
<thead>
<tr>
<th>East Side</th>
<th>West Side</th>
<th>Average</th>
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<td>8 16 24 32 37</td>
<td>8 16 24 32 37</td>
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### BEAM NO. 307.2

**STRESSES IN THOUSANDS OF LBS. PER SQ. IN.**  
(Compression marked minus. Tension not marked)

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<tr>
<th>Gage Line</th>
<th>East Side Load in thousands of pounds</th>
<th>West Side Load in thousands of pounds</th>
<th>Average Load in thousands of pounds</th>
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BEAM NO. 308.2

STRESSES IN THOUSANDS OF LBS. PER SQ. IN.
(Compression marked minus.
Tension not marked)

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<th>Gage Line</th>
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<th>Average Load in thousands of pounds</th>
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<td>East Side</td>
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<td>Gage 16</td>
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**Average Stress in Thousands of Lbs. per Sq. Ft.**

(Tension not marked minus compression.)

**BEAM NO. 306-2**

**STRESSES IN THOUSANDS OF LBS. PER SQ. FT.**

*Compression not marked minus tension.*
<table>
<thead>
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**Note:** Compression not marked.
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<tr>
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*Compression marked rounds.*