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AN INVESTIGATION OF THE EFFECTIVENESS OF STIFFENERS

ON SHEAR-RESISTANT PLATE-GIRDER WEBS

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SUMMARY

The results of 60 different tests on 2 aluminum
alloy 17S-T plate girders are presented to show the in-
fluence of size and spacing of stiffeners upon the buck-
ling characteristics of shear-resistant webs within the
elastic range. It is demonstrated that stiffeners in-
crease the stability of a web by retarding the formation
of buckles and by providing partial edge restraint to the
subdivided panels. An empirical method of proportioning
stiffeners is proposed which recognizes both of these
stiffener functions, and comparisons are made with design
procedures based upon theoretical considerations of the
buckling problem. Also, some experimental data are pro-
vided to show the effect of stiffener size and spacing
upon ultimate web strengths.

INTRODUCTION

Although stiffeners have been used for many years to
prevent shear buckling in plate girders of structural
steel, apparently little progress has been made in plac-
ing the design of stiffeners for this class of structure
upon a rational basis. The specifications for steel rail-
way bridges adopted by the American Railway Engineering
Association in 1910 required that the width of outstand-
ing leg on intermediate stiffeners should be not less than
one-thirtieth of the depth of the girder plus 2 inches,
and this same requirement is incorporated in the 1938
specifications. In plate girders with a uniform depth,
no provision is made for varying the size of stiffener
as stiffener spacings are varied; this procedure is ob-
viously essential for a balanced design.

The increasing emphasis being placed upon the use of
more accurate methods in the design of light-weight

structures, particularly those for aircraft, requires some
consideration of the stiffener problem. In reference 1
(p. 418) Timoshenko gives some data pertaining to the
flexural rigidity of stiffeners required to stiffen panels
of different proportions. (See also reference 2.) Although
theoretically the stiffener size increases with the number
of stiffeners used on any given web, only cases involving
one and two stiffeners have been considered. In the ap-
plication of these results to practical design, Timoshenko
assumes a required stiffness no more than double that in-
dicated by the theory for one stiffener.

The empirical formula for stiffener size given in
reference 3 is based upon a proposal by E. Chwalla found
in reference 4. (See also reference 5.) This solution
appears to be somewhat more suited for design than the
analysis of Timoshenko because it covers any number of
stiffeners.

The value of any solution on the basis of design de-
pends upon how closely it predicts actual behavior. Any
attempt to correlate tests results and the theory for
shear buckling in stiffened plate-girder webs, of course,
involves a number of complicating factors. Probably of
foremost importance is the fact that definite critical
buckling loads usually cannot be experimentally deter-
mined, either for the individual web panels between stif-
feners or for the stiffened panels as a whole. Because
of eccentricities of loading, lateral deflections may oc-
cur in both stiffeners and web from the early stages of
a test and no point that might be called critical, or
might serve as a basis for judging the effectiveness of
a given stiffener, will be observed. Complete failure of
a web as a shear-resistant member usually cannot occur
because of the redistribution of stress that accompanies
large deflections; hence the significance of a critical
buckling load in shear, even if it could be definitely
determined, is somewhat questionable.

The objects of this investigation were: (1) to de-
termine experimentally the influence of size and spacing
of stiffeners upon the buckling characteristics of shear-
resistant plate-girder webs within the elastic range;
(2) to evaluate, as far as possible from the test results,
certain methods of stiffener design that have been pro-
posed; and (3) to obtain some information on the influence
of stiffener size and spacing upon ultimate web strengths.

DESCRIPTIONS OF SPECIMENS

All the stiffener tests were made on two plate gird-
ers, designated specimens A and B, which were fabricated
from 17S-T aluminum-alloy plates, angles, and rivets.
Figures 1 and 2 show the structural details of the two
girders and give the principal stress and deflection fac-
tors for the type of loading used.

Table I summarizes the results of mechanical prop-
erty tests on the plate and angle materials. The tension
tests were made according to the method of reference 6;
the compression tests were made by the single-thickness
method described in reference 7. All strength values are
considerably above the guaranteed minimums (see reference
8) for 17S-T, although they are not outside the range of
properties frequently obtained on sheet and extruded forms
of this alloy. From the values of yield strength obtained
for the webs in both tension and compression, the yield
strength in shear, which is of particular interest for the
purpose of these tests, was estimated to be in the vicin-
ity of 24,000 pounds per square inch.

Although the choice of web proportions was quite
arbitrary, an attempt was made to provide specimens in
which different stiffener spacings would give a wide range
of buckling resistances within the elastic strength of the
web material. The flanges were proportioned to provide
comparatively low ratios of maximum bending to shear stress
in order to minimize the effect of bending upon the buck-
ling of the webs. This feature of the design is empha-
sized by the fact that the ratios of shear to bending de-
flection at the center of the spans under central concen-
trated loads were computed to be approximately 2:1 for
both girders.

Figure 3 shows the different stiffener spacings in-
vestigated and gives the theoretical buckling loads and
corresponding average shear stresses for the subdivided
panels, assuming simply supported edges. (See reference,
9, p. 60.) Panel widths were assumed to be equal to the
distances center to center of intermediate stiffeners.

Eight different sizes of intermediate stiffener an-
gles of 17S-T, ranging from 1/2 by 1/2 by 1/16 inch to
$1\frac{3}{4}$ by $1\frac{1}{4}$ inches by 1/4 inch, were provided, although all
sizes were not used for each spacing indicated in figure

3.   Figure 4 shows the details of these stiffener connec-
tions.

.PROCEDURE .

Figure 5 shows a typical test set-up in the 300,000-
pound-capacity Amsler testing machine. Central concen-
trated loads were applied on both girders, the end reac-
tions being carried through aluminum-alloy plate and shelf-
angle supports bolted to the end stiffeners. Roller-
bearing supports were used as indicated to permit free
movement at the ends of the span, resulting from lengthen-
ing of the bottom or tension flanges.

Measurements of lateral deflection, which were used
to indicate the buckling characteristics of the webs and
the stiffeners, were made by means of the apparatus shown
in figure 6. The use of a dial indicator, graduated in
thousandths of an inch, between the webs of the girders
and a reference bar held against the top and bottom flanges
made possible the rapid determination of deflections within
0.001 or 0.002 inch. Readings were taken at seven differ-
ent stations over the clear depth of each web on sections
spaced 2 to 4 inches along the length of the girders.

In order to determine experimentally the effect of a
number of different sizes and spacings of stiffeners upon
the behavior of a single web, it was necessary to produce
fairly definite buckle patterns for each case without
exceeding the elastic strength of the material. For cases
involving relatively few stiffeners this requirement was
easily met although, as the number of stiffeners increased,
it became increasingly difficult to obtain the desired
buckle patterns without producing permanent sets. The
theoretical buckling loads for an assumed condition of
simply supported edges (see fig. 3) were used as a guide
in the selection of safe loads, although in no cases were
the average shear stresses allowed to exceed 20,000 .
pounds per square inch, or a value slightly below the
shear yield strength estimated for the web material.used.
Loads were applied in increments up to the maximum value
selected for each case, after which permanent-set measure-
ments were made.

Figure 3 indicates the order in which the different
stiffener spacings were investigated on each girder. The

first tests were made on the largest two panels without stiffeners, labeled "First Test; later tests involved 1, 2, 3, 5, or 7 stiffeners, labeled "Series I, Series II," and so forth. Table II indicates the sizes of stiffeners included in each series. The smallest stiffeners for each spacing were selected as far as possible from theoretical requirements (reference 1, p. 418), while the largest sizes had a stiffness many times the theoretical values. The order of tests was generally according to stiffener size, starting first with the smallest single angle to be investigated for a given case and proceeding through a series of 4 to 10 different tests to a pair of the larger angles. The tests for any particular spacing were stopped when a pair of stiffeners was obtained that showed relatively little lateral deflection as compared with the deflection found for the web panels, provided such a condition could be obtained with the stiffeners available and without exceeding the imposed limit of 20,000 pounds per square inch for average shear stress.

Lateral-deflection measurements in each test were limited to the half of the span where the stiffener sizes were varied (series I, II, III, etc.), which, as is indicated in figure 3, alternated from side to side with each change of stiffener spacing. The sizes and spacings of stiffeners used on the opposite half of the girders for each series (fig. 3) generally produced a more stable web condition than that to be investigated; hence deflection readings throughout the length of the span in each test were not deemed necessary.

The method used in determining the flexural rigidity of single-angle stiffeners differs from the methods that have been proposed by other investigators. Instead of using the moment of inertia for an angle alone, about the face of the web to which it was attached, an effective width of web equal to 25 percent of the clear depth was assumed to act with each stiffener. The justification for such a procedure regarding effective widths is based upon observations made in a previous investigation. (See reference 10.) The use of an axis in the face of the web, which recognizes the stiffening influence of the web, seems somewhat inconsistent in that it implies a different effective width for each size of stiffener. For a 1/2- by 1/2- by 1/16-inch angle on a 1/8-inch web, for example, an effective width of web of 1 inch is sufficient to shift the neutral axis for the combined section to the face of the web. For a 3/4- by 3/4- by 3/16-inch angle, however, approximately 8 inches of effective width are required for

a corresponding change in the position of the neutral axis.

Table III shows a comparison of moments of inertia
for all sizes of angle determined by the two methods. For
the small single angles the values obtained when an effec-
tive width equal to 25 percent of the depth was assumed
were larger than those computed for the angles alone about
an axis in the face of the web; for the larger angles this
relative position was reversed. Although the differences
between moments of inertia computed by the two methods are
in most cases not significant, the effective-width method
seems to provide a more logical basis for the interpreta-
tion of test results. Effective widths of web were ne-
glected in computing moments of inertia for the double-
angle stiffeners, where the neutral axis from symmetry was
in the middle plane of the web.

At the conclusion of the tests to determine the ef-
fectiveness of different sizes and spacings of stiffeners
within the elastic range, both girders were tested to
failure. (Figs. 9 and 10, to be discussed later, show the
conditions investigated.) In these final loadings, the
lateral-deflection measurements were supplemented by 2-
inch Berry strain-gage readings on the flanges and stiff-
eners. (Figs. 24 to 29 show the location of the gage
lines used.) Vertical deflections at the center of the
spans were also determined, using mirrored scales attached
to the webs, midway between flanges, and fine wires
stretched between the ends of the spans.

DISCUSSION OF RESULTS

Analysis of Lateral Deflections

An analysis of the buckling phenomena observed in
this investigation involves a study of load-lateral de-
flection data obtained from 60 different tests. Although
no attempt has been made to show the results of all meas-
urements, figures 7 to 10 show typical load-deflection
relations and buckle patterns for different sizes and
combinations of stiffeners.

Figures 11 to 18 show average load-lateral deflec-
tion curves for the web panels and stiffeners in all
tests. The web deflections are the average of the maxi-
mum measured values found midway between stiffeners,
which were also the maximum values for each panel in most

cases. The stiffener deflections are the average of the
maximum values measured for each stiffener. Although
considerable variation was found in some cases between
the deflections of supposedly like panels and stiffeners,
average rather than individual maximum values were believed
to provide the most satisfactory basis for a general in-
terpretation of the test results. The influence of dif-
ferent amounts of bending upon the shear—buckling tenden-
cies of a series of like panels was apparently negligible.
Table II, which gives a summary of all but the ultimate
load tests, indicates the maximum range of web and stiff-
ener deflections observed.

     From the nature of the load—deflection curves shown
in figures 11 to 18, it seems quite evident that a definite
value cannot be experimentally determined for the flexural
rigidity of stiffeners required to stiffen panels of given
proportions, such as might be obtained by application of
the buckling theory. The first difficulty encountered is
in the determination of critical loads or the relative
buckling resistances for the different sizes of panel from
which some measure of stiffener effectiveness might be ob-
tained. Although most of the curves in figures 11, 14,
15, and 16 show a fairly pronounced knee, which is believed
to be indicative of some buckling phenomena, a quantitative
comparison of these results is obviously difficult. In
curves of the type shown in figures 12, 13, 17, and 18 the
change in the rate of deflection is so gradual that buck-
ling apparently was not involved. An analysis of these
average load—deflection data by the Southwell method (ref-
erence 1, p. 177) failed, moreover, to provide a generally
satisfactory basis for the selection of critical buckling
loads.

     In spite of the questionable status of the buckling
involved in these tests, the results indicated fairly con-
sistently that the average lateral deflections of the web
panels decreased with increasing sizes of stiffener.
Where such a behavior was observed, it seems reasonable
to assume that the buckling resistance of the web panels
had been increased by increasing the size of stiffener.
This increase may be attributed both to the effect of edge
restraint along the boundaries of the panels and to the
increased effectiveness of the larger stiffeners in con-
fining buckling to the web. The buckling theory previously
referred to assumes that the stiffeners need support the
subdivided web panels only until the critical load for a
condition of simply supported edges is developed, after
which general buckling may occur. It appears from these

tests that, although a given size of stiffener may apparently meet this requirement, a larger size may result in a greater buckling resistance in the web.

From the load-deflection curves for the largest panels tested without stiffeners, there seems little question that the actual buckling loads were considerably above the theoretical values for a condition of simply supported edges. The curves shown in figure 7 for specimen A are believed to be as satisfactory for determining experimental buckling loads as any obtained and indicate a critical value in the vicinity of 40,000 pounds. From the ratio of the theoretical buckling values for this size of panel for fixed and simply supported edges (see reference 11), a load of 40,000 pounds corresponds to an edge fixity of about 70 percent. The estimated buckling load of 20,000 pounds for the unstiffened 24- by 48-inch web panel of specimen B corresponds to a fixity of almost 84 percent. The difference in apparent edge restraint for the two specimens is of the order expected in view of the fact that different sizes of flange angle were used on webs of the same thickness.

Although no attempt was made to estimate buckling loads for the tests involving intermediate stiffeners, it seems reasonable to assume that edge restraint also had a significant bearing upon the deflections observed for these cases. In order to permit some estimate of this effect, theoretical buckling loads for a condition of simply supported edges are indicated on the load-deflection curves in figures 7 to 10 and in figures 11 to 18.

In a few tests involving a close spacing of stiffeners, loads were applied which produced accidental permanent sets sufficient to influence the buckling characteristics of the webs and stiffeners in all subsequent loadings. In the case of specimen A shown in figure 12, for example, the first test was made on an intermediate size of stiffener (test 3). Both larger and smaller sizes were left to be investigated later. Although the loads applied in this first test did not involve an average shear stress greater than 17,500 pounds per square inch, the permanent sets measured in the web were larger than the values found in any previous case. As a result, the load-deflection relations observed for both web and stiffeners in all subsequent tests indicated the effect of some eccentricity of loading.

Table II gives a summary of the maximum permanent

sets measured for the webs and stiffeners in all tests.
In most cases these values do not appear large enough to
indicate any significant departure from the range of elas-
tic action. Permanent sets of 0.015 inch or greater were
found in the web in only three tests and these involved
average shear stresses ranging from 17,300 to 18,700
pounds per square inch, which were undoubtedly above the
elastic range of the web material.

Figures 19 and 20 show the results of an attempt to
reduce all tests to a basis of comparison where some ap-
praisal of the effect of size and spacing of stiffeners
and the effect of edge restraint might be made. Since
definite values of buckling load could not be experimen-
tally determined, test loads corresponding to certain
arbitrary values of lateral deflection were selected from
figures 11 to 18 to indicate relative buckling resistances.
Loads corresponding to average maximum deflections of
0.060 inch in the web and 0.020 inch in the stiffeners
were selected for comparison with the theoretical buck-
ling loads for the web panels assuming simply supported
edges. These load ratios are plotted as ordinates in the
figures. It appears significant that, for some cases at
least, a lateral deflection of 0.060 inch in the web was
within the range of deflections where buckling occurred,
according to analyses of the load-deflection data made by
the Southwell method. Such an arbitrary value of deflec-
tion does not, of course, imply the same degree of buck-
ling for all the different sizes of panels investigated,
which is admittedly an objectionable feature of the method
of comparison used. Eccentricities of loading that may
have had a negligible effect in panels having a low buck-
ling resistance may have accounted for the entire deflec-
tion of 0.060 inch, where high buckling resistances were
involved. An average deflection of 0.020 inch was used
for the stiffeners, both because it was small and because
it was one value within the range of values measured for
most of the sizes investigated.

The abscissas in figures 19 and 20 are ratios of the
flexural rigidity (EI) of one stiffener to that for a web
panel between stiffeners, defined here as the ratio λ.
The moments of inertia used for the stiffeners in comput-
ing these ratios are shown on the load-deflection curves
in figures 11 to 18. As previously indicated, the values
for the single-angle stiffeners include an effective width
of web equal to 25 percent of the clear depth. Although
the deflections of the stiffeners appear reasonably con-

sistent in most cases with the moments of inertia computed,
the relative positions of the load-deflection curves can
hardly be used to demonstrate the correctness of the effec-
tive-width method over that in which the moments of inertia
for single angles are computed about the face of the web in
contact with the stiffeners. The moments of inertia of the
web panels between stiffeners were computed from the rela-
tion:

$$I = \frac{bt^3}{12}$$

where

I   moment of inertia, inch^4

b   stiffener spacing, inch

t   web thickness, inch

Several observations may be made from figure 19, show-
ing the influence of stiffener size upon web deflections,
which appear significant from the standpoint of design.
The ratios of the test loads corresponding to an average
lateral deflection of 0.060 inch in the web panels to the
theoretical buckling values for the case of simply sup-
ported edges are shown to increase with increasing size
of stiffener for any given proportions of panel. Such a
result not only indicates the extent to which stiffener
size may influence the buckling resistance of the webs
but also suggests that in no instance were the tests car-
ried far enough to obtain the maximum possible web effi-
ciencies. For values of λ greater than those shown, the
load ratio should presumably approach a constant value.
As the proportions of the panels were changed, however,
and a closer spacing of stiffeners used, the ratios of the
test to the theoretical buckling loads decreased. For
example, the values obtained for specimen B having only
one stiffener ($b/d = 1$) correspond to an edge condition
ranging from 30 to almost 100 percent fixed. The ratio of
buckling loads for fixed edges to simply supported edges
is assumed equal to 1.68 for all sizes of panel, which is
the theoretical ratio for infinitely long plates. (See
reference 1, p. 362, and reference 11.) For the case of
seven stiffeners ($b/d = 1/4$), the ratios correspond to
test loads less than the theoretical values for panels
with simply supported edges. In other words, the effec-
tiveness of the stiffeners, as measured by a constant
value of web deflection, decreased as the stresses corre-

sponding to the computed buckling loads increased. The
relative position of the load—stiffness ratio diagrams
for specimens A and B, for cases involving the same pro-
portions of panel with one or two stiffeners, corresponds
to the position that would be expected from the different
degrees of edge fixity indicated in the tests of the un-
stiffened panels.

The curves in figure 20, showing the influence of
stiffener size upon stiffener deflection, indicate about
the same relative behavior for panels of different pro-
portions as shown in figure 19. The shape of the curves
is fundamentally different, however, in that the load
ratios approach an infinite rather than a constant value
as the size of stiffener is increased. Stiffener deflec-
tions may approach zero; whereas deflections for the web
cannot be reduced below the deflections accompanying buck-
ling for edges completely fixed.

Proposed Basis for Stiffener Design

In the selection of stiffener sizes suitable for de-
sign from the results of these tests, an attempt was made
to recognize as far as possible the principal character-
istics of behavior noted in the foregoing figures. The
degree of edge fixity obtained for any case is, of course,
not known, and various interpretations may be placed upon
the significance of the load—deflection curves shown in
figures 11 to 18 with respect to this factor. For the
tests in which a fairly definite buckling action was ap-
parent within the elastic range, it is believed that an
assumption of 50—percent edge fixity, which involves loads
approximately 34 percent greater than the theoretical
buckling values for panels with simply supported edges,
may well be made as a basis for selecting relative pro-
portions of webs and stiffeners. For the tests in which
web buckling was not so evident, an average shear stress
of 16,000 pounds per square inch appeared to mark the
approximate limit of elastic action, and loads correspond-
ing to this stress were assumed to be equally significant
from the standpoint of stiffener design. Figures 11 to 18
show the position of the lower or critical value of these
two arbitrary design—load limits with respect to the aver-
age web and stiffener deflections measured for each size
of panel.

Some arbitrary limits on stiffener deflections were
also necessary because none of the stiffeners investigated

remained straight under the design loads selected, and
stiffener effectiveness could be determined only on a
relative basis. It is believed that a stiffener which
shows essentially the same load—deflection characteris-
tics as the web that it supports (and there are numerous
such cases indicated in figs. 11 to 18) is not adequate,
regardless of the loading for which buckling may seem to
occur. Two arbitrary deflection requirements were there-
fore imposed: (1) that the stiffener deflection not ex-
ceed 0.020 inch for the design loading assumed, and (2)
that the stiffener deflection not exceed 25 percent of
the smallest average web deflection observed for this
loading.

The moments of inertia required of stiffeners to
meet the foregoing conditions may be estimated from the
values of moment of inertia indicated on the load—
deflection curves in figures 11 to 18. For each propor-
tion of panel investigated, one value for stiffener mo-
ment of inertia was obtained. Although the buckling
theory indicates that the number of stiffeners used in
providing panels of given proportions has a significant
bearing upon the flexural rigidity required for each
stiffener, the limited scope of these tests did not make
possible a consideration of this factor. In order to
make the results obtained generally applicable to design,
ratios of the flexural rigidity of each selected stiffener
to the flexural rigidity for the corresponding web panels
were determined and plotted against proportions of panel,
as shown in figure 21. The relationship obtained may be
expressed approximately as:

$$\lambda = \frac{14}{\left(\dfrac{b}{d}\right)^3}$$

where

λ ratio of flexural rigidity of one stiffener to flexrual
      rigidity of web panel between adjacent stiffeners

d clear depth of web, inch

Figures 11 to 18 show estimated load—lateral deflec-
tion curves for stiffeners proportioned by means of the
foregoing empirical formula. The relative position of
these curves with respect to those determined from the
tests is, of course, only approximate, since the measured
deflections themselves were not always consistent with the
moments of inertia involved.

Figures 19 and 20 provide a basis for evaluating the
proposed design formula in terms of observed lateral de-
flections and theoretical buckling loads for the web pan-
els. The stiffener sizes computed for every case investi-
gated were sufficient to develop loads from 1 to $1\frac{1}{2}$ times
the theoretical values for a condition of simply supported
edges without exceeding an average web deflection of 0.060
inch, or a value less than one-half the web thickness.
These load ratios correspond to edge-fixity factors rang-
ing from zero for the closest stiffener spacing (b/d = 1/4)
to 73 percent for the widest stiffener spacing (b/d = 1).

Comparison of Stiffener Design Methods

For purposes of comparison, the moments of inertia of
stiffeners computed by the other two methods previously
referred to are also included in figures 11 to 18. In the
first method the moments of inertia were determined from
reference 3, where

$$I = \frac{(0.1 + 0.02N)t^3 d}{\beta^4} (\beta^2 + 0.625)$$

but not to exceed

$$\frac{0.2\ t^3 d}{\beta^4}(\beta^2 + 0.625)$$

where

N   number of stiffeners

d   over-all depth of web, inch

β ratio of stiffener spacing to over-all depth of web
     (Use $\beta = 0.4$ for all ratios less than 0.4.)

In the second method the moments of inertia were de-
termined from the theoretical treatment of the stiffener
problem given in reference 1 (p. 418), where ratios of
flexural rigidity λ, as previously defined, are given
for cases of one or two stiffeners on panels of different
proportions. The moments of inertia selected for design
on the basis of the tests were in most cases considerably
greater than those obtained by either of the other two
methods. No attempt was made to apply the theory to cases
involving more than two stiffeners.

In the comparison of these different methods of com-
puting moments of inertia for stiffeners, it should be
pointed out that the empirical formula proposed from the
tests and the theoretical solution given by Timoshenko in
reference 1 involve ratios of stiffener spacing to clear
depth of web; whereas the formula given in reference 3
involves ratios of stiffener spacing to over-all depth.
The significance of the over-all depth dimension from the
standpoint of web buckling is not obvious unless a con-
stant ratio of clear to over-all depth is assumed. It
appears that the design of the flange for a particular
girder might be varied in such a manner as to influence
the buckling resistance of the web appreciably without
changing the over-all depth and hence the size of stiff-
ener required to prevent such buckling.

Another feature of the formula given in reference 3
to which attention is called is the indication of con-
stant size of stiffener for cases involving five or more
stiffeners, where the ratio of stiffener spacing to over-
all depth is 0.4 or less. Unfortunately, the deflections
shown in figures 17 and 18 for tests that meet these con-
ditions do not permit any conclusion regarding this limit
on maximum stiffener size. From the standpoint of elastic
stability, however, it would seem that for a given depth
and thickness of web the size of stiffener should always
increase as the stiffener spacing decreases; otherwise
the resistance to general buckling would fall below the
resistance for the subdivided panels.

Table IV presents a further comparison of these
stiffener design methods applied to a plate girder having
proportions far outside the range investigated. The ex-
ample of plate-girder design in table IV is taken from
reference 5. As in most of the cases previously consid-
ered, the flexural rigidities required by the empirical
method proposed are the highest. The maximum sizes pro-
posed for the double-angle stiffeners, however, are no
larger than those required by current specifications for
designs in steel. (See reference 12.) It will be noted
that the same size of stiffener is required by the
Moisseiff-Lienhard method of reference 3 for two of the
three stiffener spacings considered; whereas the method
proposed provides a different size for each spacing,
which seems to be a more logical procedure. For the
cases shown, it appears that the method used in computing
moments of inertia for the single bulb-angle stiffeners,
whether based upon the assumption of a definite effective

width of web (I_{4-4}) or upon the assumption that bending
in the stiffener is produced about an axis in the face of
the web (I_{3-3}), may not be important as far as the actual
size of the angle used is concerned. It should be recog-
nized, however, that for certain proportions of web and
stiffener, moments of inertia computed about the face of
the web in contact with the stiffener (I_{3-3}) may be higher
than those obtainable from any reasonable assumption re-
garding effective widths. In the case of the $6\frac{1}{2}-$ by $3-$ by
3/8-inch and 6- by 3- by 5/16-inch bulb angles given in the
table, for example, the values of I_{3-3} correspond to effec-
tive widths over twice the stiffener spacing or the maximum
width available for each stiffener. One of the most sig-
nificant observations to be made from the stiffener ele-
ments given in the table is that the single bulb-angle
stiffeners are much more effective, from the standpoint of
weight-stiffness ratios, than the conventional double-
angle type of stiffener.

                    Ultimate-Load Tests

     Although stiffeners proportioned by the method pro-
posed are seemingly adequate for shear stresses within the
elastic range, their ultimate resistance to buckling is
also important from consideration of design. Ultimate-
load tests on the two girders used throughout the investi-
gation have provided an opportunity to obtain a few data
on this aspect of the stiffener problem. Figures 9 and 10
show the sizes and the spacings of stiffeners used in the
ultimate-load tests. The flexural rigidity of the stiff-
eners on the left half, where the closest spacings were
used, was chosen to agree approximately with the require-
ments of the proposed design formula. The same sizes were
also used for the wider spacings on the right half to pro-
vide an extra margin of stiffener rigidity (46 percent
for specimen A and 86 percent for specimen B) to offset
in some measure the differences in buckling resistance
for the two sizes of web panel.

     Table V gives the results of the ultimate-load tests
with the corresponding computed average shear and maximum
bending stresses. The shear stresses developed in the
webs of both girders were in the vicinity of the shear
yield strength estimated for the web material, which is
generally assumed to be the design limit for shear-
resistant web action. The strengths obtained in these

tests, therefore, were as high as could be reasonably ex-
pected.

Essentially the same type of failure was obtained in
both girders. The severe buckling action produced on the
weak half of the webs eventually broke the machine-screw
connections holding the stiffeners, so that collapse and
fracture of the webs immediately followed. The stiffeners
on specimen A were badly bent before failure of the con-
nections, but those on specimen B apparently were undam-
aged except for a somewhat battered condition at the ends
where they were pinched between the flange angles. Fig-
ures 22 and 23 show the nature of the failures obtained.
In specimen A, the wide diagonal-tension fracture pro-
duced in the web passed through one of the holes for the
stiffener connections, which presumably constituted a
"stress raiser." In specimen B, the concentration of
tensile stress at the upper corner fractured the web and
sheared the end-flange rivet.

Failure in the stiffener connections was not expect-
ed in these tests, although the weakness of such details
must be recognized as a possibility in design. From the
large distortions produced in the stiffeners on the right
half of specimen A before failure occurred, it appears
that about the maximum possible degree of effectiveness
was obtained from these stiffeners, and there is little
reason to question the adequacy of the connections. The
connections used for the stiffeners on specimen B are
admittedly smaller than would have been used if this de-
tail had not been carried over from previous tests in-
volving smaller angles. The use of stronger connections
undoubtedly would have increased the load-carrying capac-
ity of the web; although the method to be used in design-
ing such details, other than maintaining reasonable pro-
portions, is not apparent. Even though the connections
used on specimen B were not adequate to develop the full
flexural rigidity of the stiffeners, their shortcomings
in this particular test are not considered   serious
in view of the high average shear stress developed.

The lateral deflections shown in figures 9 and 10
and the condition of the girders after failure shown in
figures 5 and 22 indicate that the stiffeners used on
the left half of both girders were adequate to develop
the full strength of the webs as shear-resistant members.
It is obviously not possible to say what margin of
strength these stiffeners may have had against ultimate
collapse as tension-field action became more pronounced.

The fact that stresses in the vicinity of the shear yield
strength of the material were developed, however, without
any signs of stiffener weakness appears to be a signifi-
cant observation from the standpoint of design. For loads
within a few percent of the maximum applied, none of the
stiffener deflections on the left half exceeded 0.035
inch. The theoretical buckling loads for the web panels
were also near the shear yield strength; appreciable web
deflections were therefore not produced until stresses in
excess of about 20,000 pounds per square inch stress were
imposed.

     The lateral deflections produced on the right half
of the girders, where web failures ultimately occurred,
were of much greater magnitude than those found on the
left half. The buckle patterns shown in figures 9 and
10 for loads near the ultimate load indicate two quite
different types of action. In specimen A the wave forma-
tion was continuous across the stiffeners and this pat-
tern, as shown in figure 22, was not changed appreciably
by failure involving some degree of tension-field action.
In specimen B the stiffeners were sufficiently rigid to
confine buckling almost entirely to the web panels and
three or more half-waves were produced in each. As soon
as the stiffeners were broken off, however, a typical
tension-field buckle pattern was produced, as shown in
figure 23.

     In view of the fact that the stiffeners used on the
right half of both girders had flexural rigidities some-
what greater than the rigidity required by the proposed
formula, it is only possible to estimate the adequacy of
the formula for these particular cases. There is appar-
ently little question concerning the stiffeners on spec-
imen B because only small lateral deflections were
observed and a maximum shear stress was developed which
was greater than the yield strength of the material and
approximately 90 percent greater than the theoretical
buckling stress for the web panels. A decrease of 54
percent in the flexural rigidity of the stiffeners, in
accordance with the proposed method, would not, it is
believed, seriously impair the strength of the web.

     In specimen A large stiffener deflections were not
observed until loads corresponding to an average shear
stress of about 20,000 pounds per square inch were im-
posed. Under such conditions, plastic yielding of the
web would be expected and the accompanying loss in buck-
ling resistance should result in some deflection of the

stiffeners.   The margin of strength against failure was
obviously not so great as in the case of specimen B,
but the fact that an average shear stress in the vicinity
of the shear yield strength of the material and about 40
percent greater than the theoretical buckling stress for
the panels was developed seems indicative of fairly well-
balanced proportions for shear-resistant web action.
Accordingly, as far as the results of these few ultimate-
strength tests are concerned, there appears to be no
reason to question seriously the adequacy of the proposed
stiffener formula for purposes of design.

In addition to the lateral deflections already dis-
cussed, figures 9 and 10 show the results of vertical-
deflection measurements made at the center of the spans
in the ultimate-load tests.   Unfortunately, the elastic
strength of the girders cannot be estimated from these
data because small amounts of overstrain were produced
unintentionally in some of the earlier tests.   It is of
interest to note, however, the close agreement obtained
between measured and computed deflections within the
elastic range indicated.   In each case approximately two-
thirds of the deflection was computed to be the result
of shearing deformations, the remaining one-third was
computed to be the result of flexure.   Such girder pro-
portions are not generally encountered in design, but
apparently they present no difficulty as far as the esti-
mation of probable deflections is concerned.

Figures 24 and 25 show the results of stress measure-
ments on a number of the intermediate stiffeners of both
girders.   Although there is ample evidence of bending in
the stiffeners, which deflected appreciably with the
webs, no data were obtained to show that the stiffeners
carried part of the shear by column action, as is the case
for stiffeners on webs of the tension-field type.   This
observation is of interest in view of the requirement
given in reference 5 (art. 226) that vertical stiffeners
be designed as columns to resist a portion of the shear
load, the amount depending upon the ratio of stiffener
spacing to depth of web.   According to the method of com-
putation outlined in this specification, the intermediate
stiffeners on the weaker half of specimen A, under a load
of 80,000 pounds, should have been subjected to an average
compressive stress of approximately 31,000 pounds per
square inch.   From the measurements shown in figure 24,
such a stress condition was not produced.   By the same
requirement, the intermediate stiffeners on specimen B

under a load of 160,000 pounds should have been subjected
to an average compression of 15,000 pounds per square
inch, which is also not supported by the stress measure-
ments given.

The design of load-bearing stiffeners on the assump-
tion of column action is perhaps a more logical procedure;
although, as far as the results of these test are concerned,
such a method appears quite conservative. Figures 26 and
27 show that the average measured stresses in the stiff-
eners near the top flange accounted for only about two-
thirds of the applied load, while the stresses measured
at the middle accounted for about one-third of the total.
The ends of the load-bearing stiffeners on both girders
were machined to fit closely between the fillets of the
top and bottom flange angles. It should also be noted
that the top of the web was flush with the face of the
compression flange. This condition caused the web to be
loaded directly in bearing on its extreme fibers rather
than through the compression-flange rivets, as is usually
the case.

The results of stress measurements on the top and
bottom flanges of both girders are shown in figures 28
and 29. A very satisfactory agreement between average
measured and computed bending stresses was obtained for
the compression flange of specimen A, but in all other
cases the measured values were considerably greater than
those computed. Although it is not possible to account
definitely for the discrepancies shown, the effect of
gage length with respect to rivet spacing, the unequal
distribution of load between the flange rivets, the ef-
fect of stress concentrations, and the lack of integral
action are all possible contributing factors. Moments of
inertia based upon net sections rather than gross sections
would have provided a better agreement between measured
and computed stresses in some cases, but there appears
to be no logical reason for the use of net sections when
an attempt is made to compute average stresses over gage
lengths equal to the distance between rivet holes. From
the good agreement between measured and computed vertical
deflections previously shown in figures 9 and 10, it ap-
pears that these irregularities in measured stress were
not reflected in the over-all behavior of the girders.

Table V gives the computed bending stresses corre-
sponding to the maximum loads carried by both girders.
It should be recognized that, since no evidence of flange
failure other than plastic yielding was obtained, the

values of stress given do not represent ultimate strengths.
It may be pointed out, however, that the maximum stress
computed for specimen B corresponds very closely to the
theoretical buckling value, assuming that one edge of the
flange is to be built in and the other edge is to be free.
(See reference 9, tables 11 and 12.) The maximum computed
stress for specimen A is about 20 percent less than the
theoretical buckling value for the same edge conditions,
an indication that a considerably higher value of flange
stress might have been developed if failure in the web had
not occurred.

The average bearing stresses on the flange rivets
corresponding to the maximum applied loads were computed
to be approximately 67,000 pounds per square inch in
specimen A and 72,000 pounds per square inch in specimen
B. After the ultimate-load tests had been completed, a
portion of the top and bottom flange angles and the end
load-bearing stiffeners were removed from the less severe-
ly damaged end of each girder for inspection of the rivet
holes in the webs. From the measurements of hole distor-
tion it appears that, even for the sides where the webs
were still intact, the distribution of load between rivets
ultimately obtained was not uniform. The largest changes
in hole diameter, about 10 percent for specimen B, were
in a direction consistent with the diagonal tension devel-
oped in the webs. The maximum changes in hole diameter
found in specimen A, where a somewhat lower average shear
stress was developed, were only about 2 percent. An ex-
amination of the rivet holes in the webs on the side where
failures occurred was not made because of the severe local
distortions produced and the uncertainty concerning the
magnitude of the bearing stresses involved.

CONCLUSIONS

The results of this investigation are believed to
justify the following conclusions:

1. Definite values for the flexural rigidity of stiff-
eners required to stiffen panels of given proportions,
such as have been obtained by application of the buckling
theory, apparently cannot be experimentally determined.
Measurements of lateral deflection, as made in these tests,
are useful in presenting a relative picture of web and
stiffener behavior, but they do not permit a quantitative

determination of buckling resistance or stiffener effec-
tiveness. Perhaps the most significant observation made,
and the one that is also the most confusing from the
standpoint of analysis, is that the buckling resistance
of a web always may be increased by increasing the size
of stiffener used until a condition of complete edge
fixity is obtained for the subdivided panels.

2. The relative lateral deflections observed for the
different sizes and types of stiffeners, whether of
single- or double-angle type, were reasonably consistent
with the computed stiffener moments of inertia. Effec-
tive widths of web equal to 25 percent of the clear depths
were assumed for the single-angle stiffeners, although
essentially the same results would have been obtained for
most of the sizes considered if moments of inertia had
been computed about the face of the web in contact with
the stiffeners. This procedure is simpler from the stand-
point of design but implies an appreciably different ef-
fective width of web for each size of angle, a condition
that is not believed to be consistent with actual behavior.
For large angles, moments of inertia computed about the
face of the web may correspond to effective widths far
greater than the stiffener spacing or the available web
for each stiffener.

3. A comparison of the flexural rigidities obtainable
from single- and double-angle stiffeners of similar pro-
portions indicates the single-angle stiffeners to be more
effective from the standpoint of stiffness-weight ratios.

4. The selection of stiffener proportions on the
assumption that buckling will occur in the web for the
load computed as critical for a condition of simply sup-
ported edges, as is done in the case of the stiffener
theory, does not appear to be a conservative procedure
as far as stiffener design is concerned in view of the
appreciable edge restraint indicated for the web panels
in many of the tests.

5. The following empirical formula is proposed as a
tentative basis for the design of stiffeners on shear-
resistant webs:

$$\lambda = \frac{14}{\left(\dfrac{b}{d}\right)^3}$$

where

λ ratio of flexural rigidity of one stiffener to flexural
        rigidity of web panel between adjacent stiffeners

b   stiffener spacing, inch

d   clear depth of web, inch

6. For most cases, apparently, the formula given in conclusion 5 provides stiffeners having more flexural rigidity than was indicated as necessary by either of the other two stiffener-design methods considered. Since there is no accepted basis for the determination of the requirements of an adequate stiffener for purposes of design, it is obviously difficult to evaluate different design methods. On the basis of the deflections observed in these tests, it hardly seems likely that the stiffener sizes proposed as adequate for the web panels investigated will be generally classed as too large. The stiffeners proposed are not, in general, so large as the stiffeners that would be required by current specifications for designs in structural steel.

7. As far as could be determined from ultimate-load tests on only two girders, each involving one size of stiffener on two different spacings, the proposed design method provides ample margin of strength against ultimate failure in the stiffeners. In both girders, the average shear stresses corresponding to the maximum applied loads were in the vicinity of the shear yield strength estimated for the web material. These maximum shear stresses also exceeded the theoretical buckling values for the weakest web panels by approximately 40 percent in specimen A and 90 percent in specimen B.

8. Although the strengths developed in the two girders were as high as would normally be considered obtainable in the design of shear-resistant webs of aluminum alloy 17S-T, it is significant that ultimate collapse and fracture did not occur until the connections between webs and stiffeners on the weaker half of the girders were broken. In specimen A, the full flexural rigidity of the stiffeners was apparently developed; in specimen B, the use of stronger stiffener connections would undoubtedly have increased the load-carrying capacity of the web.

9. The stress measurements made on a number of intermediate stiffeners on both girders provided no evidence

that these members should be designed as columns to re-
sist a portion of the shear. The average measured stresses
in the load-bearing stiffeners at the center of the spans,
for sections within $2\frac{3}{8}$ inches of the top flange, accounted
for only about two-thirds of the applied load. The
stresses at the center of these stiffeners accounted for
only one-third of the applied load.

10. The maximum computed bending stresses in the
flanges for the loads producing web failures were 28,100
pounds per square inch in specimen A and 33,700 pounds
per square inch in specimen B. The value for specimen B
corresponds closely to the theoretical buckling stress
for the outstanding flange, assuming one edge built in
and the other edge free. No evidence of primary flange
failure was obtained.

11. Within the apparent elastic range, the measured
vertical deflections at the center of the spans were in
very close agreement with the computed values. Approxi-
mately two-thirds of these deflections were computed to be
the result of shear; the remaining one-third were computed
to be the result of flexure.

12. The average computed bearing stresses between
flange rivets and webs for the maximum applied loads were
approximately 67,000 pounds per square inch in specimen A
and 72,000 pounds per square inch in specimen B. An ex-
amination of some of the rivet holes in the webs for the
half of the girders still intact indicated permanent dis-
tortions in the direction of the diagonal-tensile stresses
ultimately developed. The maximum increases in hole diam-
eter were about 2 percent in specimen A and 10 percent in
specimen B.

RECOMMENDATIONS

It is recognized that the proof of the dependability
of any proposed new method of stiffener design requires
more experimental verification than was obtained in this
investigation. It is proposed, therefore, as an essential
step in the formulation of a satisfactory solution to the
stiffener problem, that an additional series of aluminum-
alloy 17S-T plate girders be fabricated for test purposes.
The principal object of these new tests should be to com-
pare the method of stiffener design proposed in this report
with other methods on girders representing more balanced

proportions of conventional design. The intermediate
stiffeners should extend the full depth between flanges
rather than over only the clear depth of web, and they
should be riveted rather than bolted to the webs. Each
girder should involve only one size and spacing of inter-
mediate stiffeners and should be used for only one test,
and that test should be carried to failure. Such an
investigation not only would provide comparative data on
methods of proportioning intermediate stiffeners but also
would make possible some analysis of the present design
methods of providing a reasonable equality in shear and
flexural strengths.

Aluminum Research Laboratories,
      Aluminum Company of America,
            New Kensington, Penna., December 19, 1941.

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TABLE I.— PROPERTIES OF 17S—T PLATE—GIRDER MATERIAL

Girder	Material[a]	Yield strengths (offset = 0.2 percent)		Tensile strength	Elongation in 2 inches
		Tension[b] (lb/sq in.)	Compression[c] (lb/sq in.)	(lb/sq in.)	(percent)
Specimen A	Web—X	44,800	47,300	63,900	18.0
	Web—W	51,100	40,900	65,800	22.5
	Flange—W	48,700	42,800	68,800	18.0
Specimen B	Web—X	42,800	44,400	65,600	20.5
	Web—W	48,300	39,400	66,200	21.0
	Flange—W	49,300	44,300	71,500	19.0

[a] Web material 1/8 in. thick.
Flanges: 3- by 2- by 5/16—in. extruded angles, Specimen A
         4- by 3- by 3/8—in. extruded angles, Specimen B
X indicates cross—grain specimen.
W indicates with—grain specimen.

[b] Tests made on standard rectangular tension specimens with 2—in. gage lengths. (See fig. 2 of reference 6.)

[c] Tests on web made on 0.125— by 5/8— by $2\frac{5}{8}$—in. specimens by single—thickness method. (See reference 7.) Tests on flanges made on 5/8— by 2—in. specimens of full thickness, tested as columns with flat ends.

TABLE II.- SUMMARY OF MEASURED LATERAL DEFLECTIONS AND PERMANENT SETS FOR WEBS AND STIFFENERS

Number and size of panels	Number of stiffeners	Number and size of angles per stiffener	Moment of inertia per stiffener (in.^{4}) (a)	Theoretical buckling load per panel (lb) (b)	Corresponding average shear stress (lb/sq in.)	Test load (lb) (c)	Maximum web deflections midway between stiffeners (0.001 in.)		Maximum stiffener deflections (0.001 in.)		Maximum permanent sets (0.001 in.)	
							Range	Average all panels	Range	Average all stiffeners	Web	Stiffeners
SPECIMEN A												
1 - 12x24 in.	None	----------	----------	28,300	8,570	35,000	13	----------	----------	----------	---	----------
						45,000	133	----------	----------	----------	5	----------
2 - 12x12 in. (Series I)	1	1 - 1/2x1/2x1/16	0.0040	38,700	9,690	40,000	14-16	16	------	9	---	----------
						50,000	88-111	98	------	117	8	13
		2 - 1/2x1/2x1/16	.0076	38,700	9,690	40,000	14-20	17	------	15	---	----------
						50,000	66-76	71	------	81	7	6
		1 - 3/4x3/4x3/32	.0136	38,700	9,690	40,000	9-14	12	------	4	---	----------
						55,000	108-113	110	------	56	4	2
		2 - 3/4x3/4x3/32	.0306	38,700	9,690	40,000	13-17	15	------	5	---	----------
						60,000	135-139	132	------	33	6	4
		1 - 3/4x3/4x3/16	.0264	38,700	9,690	40,000	29-30	30	------	5	---	----------
						60,000	138-132	130	------	38	3	2
		2 - 3/4x3/4x3/16	.0706	38,700	9,690	40,000	20-24	22	------	4	---	----------
						66,000	147-174	161	------	80	11	3
3 - 8x12 in. (Series II)	2	1 - 3/4x3/4x3/32	0.0136	66,400	16,600	40,000	16-37	26	24-25	25	---	----------
						55,000	42-80	63	65-69	67	6	6
		2 - 3/4x3/4x3/32	.0306	66,400	16,600	50,000	13-49	29	22-23	23	---	----------
						70,000	37-111	76	66-75	70	7	9
		1 - 3/4x3/4x3/16[d]	.0264	66,400	16,600	60,000	6-21	13	3-6	4	---	----------
						70,000	34-109	80	47-51	49	15	8
		2 - 3/4x3/4x3/16	.0706	66,400	16,600	60,000	7-61	38	14-21	17	---	----------
						70,000	24-89	52	27-29	28	2	1
		1 - 1x3/4x1/8	.0400	66,400	16,600	60,000	18-75	45	21-23	22	---	----------
						70,000	36-107	71	33-48	40	8	3
		2 - 1x3/4x1/8	.101	66,400	16,600	60,000	11-85	34	9-12	11	---	----------
						70,000	28-84	51	15-19	17	6	2
4 - 6x12 in. (Series III)	3	1 - 3/4x3/4x3/32	0.0136	105,200	26,300	60,000	7-25	16	8-24	15	---	----------
						70,000	32-82	52	36-80	52	7	7
		2 - 3/4x3/4x3/32	.0306	105,200	26,300	60,000	6-18	12	1-14	7	---	----------
						75,000	14-48	28	10-37	20	10	2
		1 - 3/4x3/4x3/16	.0264	105,200	26,300	60,000	3-5	4	3-5	4	---	----------
						75,000	10-17	13	4-15	11	3	1

[a]Moments of inertia for single-angle stiffeners include effective width of web equal to 25 percent of clear depth between flanges. Web neglected in cases of double-angle stiffeners. See table III.

[b]Based on assumption of simply supported edges. See table 17 of reference 9.

[c]Larger test load was maximum applied in each case. Smaller test load selected to show, by comparison with larger load, change in rate of deflection.

[d]First test made in this series. Permanent sets produced resulted in relatively large deflections for subsequent tests.

[e]Check tests made after series V had been completed.

TABLE II.- SUMMARY OF MEASURED LATERAL DEFLECTIONS AND PERMANENT SETS FOR WEBS AND STIFFENERS

Number and size of panels	Number of stiffeners	Number and size of angles per stiffener	Moment of inertia per stiffener (in.^{4}) (a)	Theoretical buckling load per panel (lb) (b)	Corresponding average shear stress (lb/sq in.)	Test load (lb) (c)	Maximum web deflections midway between stiffeners (0.001 in.) Range	Average all panels	Maximum stiffener deflections (0.001 in.) Range	Average all stiffeners	Maximum permanent sets (0.001 in.) Web	Stiffeners
					SPECIMEN B							
1 - 24x48 in.	None	———	——	12,300	1,640	16,000	25	---	———	———	—	———
						30,000	239	---	———	———	5	———
3 - 24x24 in. (Series I)	1	1 - 1/2x1/2x1/16	0.0046	18,100	2,420	20,000	2-27	14	———	16	—	———
						26,000	111-179	145	———	83	1	3
		2 - 1/2x1/2x1/16	.0076	18,100	2,420	20,000	10-18	14	———	21	—	———
						30,000	115-133	124	———	132	2	3
		1 - 3/4x3/4x3/32	.0150	18,100	2,420	20,000	5-13	9	———	9	—	———
						30,000	64-122	93	———	113	4	3
		2 - 3/4x3/4x3/32	.0306	18,100	2,420	20,000	15-16	16	———	8	—	———
						30,000	66-131	100	———	50	5	2
		2 - 3/4x3/4x3/32[e]	.0306	18,100	2,420	20,000	30-32	31	———	7	—	———
						30,000	119-124	122	———	43	3	3
		1 - 3/4x3/4x3/16	.0304	18,100	2,420	20,000	9-18	14	———	3	—	———
						30,000	97-113	115	———	43	3	2
		2 - 3/4x3/4x3/16	.0706	18,100	2,420	20,000	10-12	11	———	3	—	———
						36,000	155-167	161	———	17	4	3
		1 - 1x3/4x1/8	.0447	18,100	2,420	20,000	20-22	21	———	3	—	———
						30,000	108-128	118	———	36	2	1
		2 - 1x3/4x1/8	.101	18,100	2,420	20,000	11-16	13	———	2	—	———
						36,000	150-158	154	———	16	2	2
		1 - 1-1/4x1x1/8	.0826	18,100	2,420	20,000	14-20	17	———	1	—	———
						36,000	165-167	166	———	30	1	2
		2 - 1-1/4x1x1/8	.192	18,100	2,420	24,000	5-10	8	———	1	—	———
						36,000	149-153	150	———	12	1	1
		2 - 1-1/4x1x1/8[e]	.192	18,100	2,420	20,000	23-33	28	———	2	—	———
						34,000	148-155	152	———	7	4	3
3 - 16x24 in. (Series II)	2	1 - 3/4x3/4x3/16	0.0304	31,100	4,150	25,000	10-35	22	0-7	4	—	———
						35,000	40-103	70	27-39	33	3	4
		2 - 3/4x3/4x3/16	.0706	31,100	4,150	30,000	17-24	20	3-9	6	—	———
						45,000	105-147	123	38-68	63	2	3
		1 - 1x3/4x1/8	.0447	31,100	4,150	30,000	9-14	11	1-9	5	—	———
						40,000	65-112	85	56-88	62	2	2
		2 - 1x3/4x1/8	.101	31,100	4,150	35,000	13-19	18	10-12	11	—	———
						55,000	118-147	134	39-64	52	2	2
		1 - 1-1/4x1x1/8	.0826	31,100	4,150	35,000	18-40	28	2-15	8	—	———
						50,000	104-159	138	36-64	63	3	1
		2 - 1-1/4x1x1/8	.192	31,100	4,150	35,000	6-25	14	5-7	6	—	———
						65,000	168-185	177	24-48	36	3	1
		1 - 1-1/2x1x5/32	.168	31,100	4,150	35,000	8-45	31	2-8	6	—	———
						55,000	139-185	166	38-43	40	7	4
		2 - 1-1/2x1x5/32	.420	31,100	4,150	35,000	20-27	24	5-6	6	—	———
						65,000	151-182	185	20-30	25	2	4

TABLE II.- SUMMARY OF MEASURED LATERAL DEFLECTIONS AND PERMANENT SETS FOR WEBS AND STIFFENERS

Number and size of panels	Number of stiffeners	Number and size of angles per stiffener	Moment of inertia per stiffener (in.⁴) (a)	Theoretical buckling load per panel (lb) (b)	Corresponding average shear stress (lb/sq in.)	Test load (lb) (c)	Maximum web deflections midway between stiffeners (0.001 in.)		Maximum stiffener deflections (0.001 in.)		Maximum permanent sets (0.001 in.)	
							Range	Average all panels	Range	Average all stiffeners	Web	Stiffeners
Specimen B (Continued)												
4 - 12x24 in. (Series III)	3	1 - 1x3/4x1/8	0.0447	49,800	6,570	35,000	10-28	19	6-17	13	—	—
						45,000	42-74	61	42-81	54	4	4
		2 - 1x3/4x1/8	.101	49,200	6,570	45,000	3-34	12	1-14	6	—	—
						55,000	55-99	81	36-53	46	6	4
		1 - 1-1/4x1x1/8	.0826	49,200	6,570	40,000	23-51	32	11-24	17	—	—
						50,000	54-99	75	40-57	49	3	4
		2 - 1-1/4x1x1/8	.192	49,200	6,570	50,000	12-38	22	3-8	4	—	—
						70,000	91-123	111	32-56	47	4	6
		1 - 1-1/2x1x5/32	.168	49,200	6,570	45,000	18-48	30	6-17	10	—	—
						60,000	83-117	96	38-44	40	3	4
		1 - 1-1/2x1x5/32	.420	49,200	6,570	50,000	1-36	9	2-9	5	—	—
						80,000	85-163	134	32-49	40	4	3
		1 - 1-1/2x1x1/4	.231	49,200	6,570	50,000	12-46	29	3-13	5	—	—
						70,000	112-130	119	39-47	44	4	5
		2 - 1-1/2x1x1/4	.632	49,200	6,570	60,000	5-71	39	4-6	5	—	—
						90,000	108-212	159	19-29	23	7	3
		1 - 1-3/4x1-1/4x1/4	.366	49,200	6,570	60,000	58-93	78	3-14	11	—	—
						80,000	129-155	147	16-42	31	5	5
		2 - 1-3/4x1-1/4x1/4	1.03	49,200	6,570	80,000	3-87	48	4-9	6	—	—
						90,000	81-204	148	12-31	17	8	3
6 - 8x24 in. (Series IV)	5	1 - 1-1/4x1x1/8	0.0826	101,200	13,500	80,000	7-26	14	2-17	10	—	—
						80,000	39-93	64	33-59	47	2	7
		2 - 1-1/4x1x1/8	.0192	101,200	13,500	80,000	8-78	31	9-37	16	—	—
						100,000	31-124	78	16-69	44	3	3
		1 - 1-1/2x1x5/32	.168	101,200	13,500	70,000	4-38	20	3-18	12	—	—
						90,000	39-72	59	13-37	28	4	5
		2 - 1-1/2x1x5/32	.420	101,200	13,500	80,000	5-52	23	7-19	11	—	—
						110,000	18-122	75	12-42	25	11	4
		1 - 1-1/2x1x1/4	.231	101,200	13,500	80,000	5-43	20	3-13	7	—	—
						110,000	44-119	70	18-48	34	5	5
		2 - 1-1/2x1x1/4	.632	101,200	13,500	90,000	6-65	22	7-15	11	—	—
						130,000	40-145	96	12-45	23	23	3
		1 - 1-3/4x1-1/4x1/4	.366	101,200	13,500	90,000	9-78	39	8-19	11	—	—
						120,000	40-128	81	14-45	25	4	5
		2 - 1-3/4x1-1/4x1/4	1.03	101,200	13,500	90,000	5-77	33	6-14	10	—	—
						130,000	39-118	73	10-28	17	8	3
8 - 6x24 in. (Series V)	7	1 - 1-1/2x1x5/32	0.168	174,000	23,200	80,000	12-37	22	1-15	10	—	—
						110,000	37-87	56	18-53	34	3	3
		2 - 1-1/2x1x5/32	.420	174,000	23,200	100,000	6-48	18	4-18	8	—	—
						130,000	12-107	51	7-40	18	4	6
		1 - 1-1/2x1x1/4	.231	174,000	23,200	100,000	3-63	22	1-18	8	—	—
						140,000	39-167	82	10-66	50	18	9
		2 - 1-1/2x1x1/4	.632	174,000	23,200	120,000	12-81	28	9-28	18	—	—
						150,000	15-127	49	15-44	23	9	6
		1 - 1-3/4x1-1/4x1/4	.366	174,000	23,200	120,000	6-103	39	6-35	17	—	—
						150,000	32-153	72	13-58	35	9	5

TABLE III.— ELEMENTS OF INTERMEDIATE STIFFENERS

Size of stiffener angle (in.)	Area of one angle (sq in.)	Stiffener on one side of web only		I for angle about face of web (in.⁴)	I for stiffeners on both sides of web[a] (in.⁴)
		I for effective width of web equal to 25 percent of clear depth (in.⁴)			
		Specimen A	Specimen B	Specimens A and B	Specimens A and B
1/2 x 1/2 x 1/16	0.059	0.0040	0.0046	0.0026	0.0076
3/4 x 3/4 x 3/32	.132	.0136	.0150	.0116	.0306
3/4 x 3/4 x 3/16	.246	.0264	.0304	.0267	.0706
1 x 3/4 x 1/8	.202	.0400	.0447	.0413	.101
1¼ x 1 x 1/8	.27	.0737	.0826	.0820	.192
1½ x 1 x 5/32	.37	.147	.168	.185	.420
1½ x 1 x 1/4	.56	.198	.231	.276	.632
1¾ x 1½ x 1/4	.69	.314	.366	.462	1.03

[a]Effective width of web neglected.

TABLE IV.- COMPARISON OF STIFFENER SIZES COMPUTED BY DIFFERENT DESIGN METHODS

[Example of plate-girder design (120- x 5/16-inch web; 112.5-inch clear depth) from reference 5, art. 805]

Stiffener spacing (in.)	Number of stiffeners	Required moment of inertia, I, per stiffener (in.^{4})			Required size for single bulb-angle stiffeners[a]			Required size for ordinary double-angle stiffeners[b]			
		Reference 3	Reference 1[c]	Proposed	Reference 3	Reference 1	Proposed	Reference 3	Reference 1	Proposed	Steel specifications (reference 12)
32.6	8	22.46	------	53.5	5x3x5/16 A = 2.99 I_{3-3}=24.9 I_{4-4}=22.7	------	6-1/2x3x3/8 A = 4.27 I_{3-3}=63.6 I_{4-4}=53.3	5x3x5/16 A = 4.80 I_{1-1}=38.7	------	6x3-1/2x3/8 A = 6.86 I_{1-1}= 57.1	6x3-1/2x3/8 A = 6.86 I_{1-1}=57.1
41.5	6	22.46	19.5	33.1	5x3x5/16 A = 2.99 I_{3-3}=24.9 I_{4-4}=22.7	5x3x5/16 A = 2.99 I_{3-3}=24.9 I_{4-4}=22.7	6x3x5/16 A = 3.31 I_{3-3}=40.1 I_{4-4}=35.5	5x3x5/16 A = 4.80 I_{1-1}=38.7	5x3x5/16 A = 4.80 I_{1-1}=38.7	5x3x3/8 A = 5.72 I_{1-1}=34.4	6x3-1/2x3/8 A = 6.86 I_{1-1} = 57.1
61.2	4	8.62	7.7	15.2	4x3x1/4 A = 2.07 I_{3-3}=10.2 I_{4-4}=10.1	4x3x1/4 A = 2.07 I_{3-3}=10.2 I_{4-4}=10.1	5x3x5/16 A = 2.99 I_{3-3}=24.9 I_{4-4}=22.7	4x3x1/4 A = 3.38 I_{1-1}=13.2	3-1/2x3x1/4 A = 3.12 I_{1-1}=8.3	4x3x5/16 A = 4.18 I_{1-1}=15.2	6x3-1/2x3/8 A = 6.86 I_{1-1}=57.1

[a]Selected from table 21 of reference 5 for aluminum alloy 27S-T structures. A = area of stiffener, sq in.; I_{3-3} = moment of inertia of angle alone, about face of web, in.^{4}; I_{4-4} = moment of inertia for angle plus effective width of web equal to 25 percent of clear depth, in.^{4}.

[b]I_{1-1} = moment of inertia about center line of web.

[c]Values of I are 100 percent greater than theoretical values for case of one stiffener. See reference 1, p. 417.

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TABLE V.- ULTIMATE STRENGTHS OF PLATE GIRDERS UNDER CENTRAL CONCENTRATED LOADS

[See figs. 9 and 10 for sizes and spacings of intermediate stiffeners in ultimate-load tests]

Specimen	Over-all depth (in.)	Span (ft)	Span (in.)	Ultimate load (lb)	Corresponding average shear stress (lb/sq in.)	Corresponding maximum bending stress (lb/sq in.)	Remarks
A	16	4	$8\frac{1}{2}$	93,300	23,300	28,100	Web collapsed and fractured after all stiffener connections on weaker half of girder were broken.
B	30	9	1/2	191,500	25,500	33,700	Web collapsed and fractured after connections for two end stiffeners on weaker half were broken. End rivet in compression flange also sheared off.
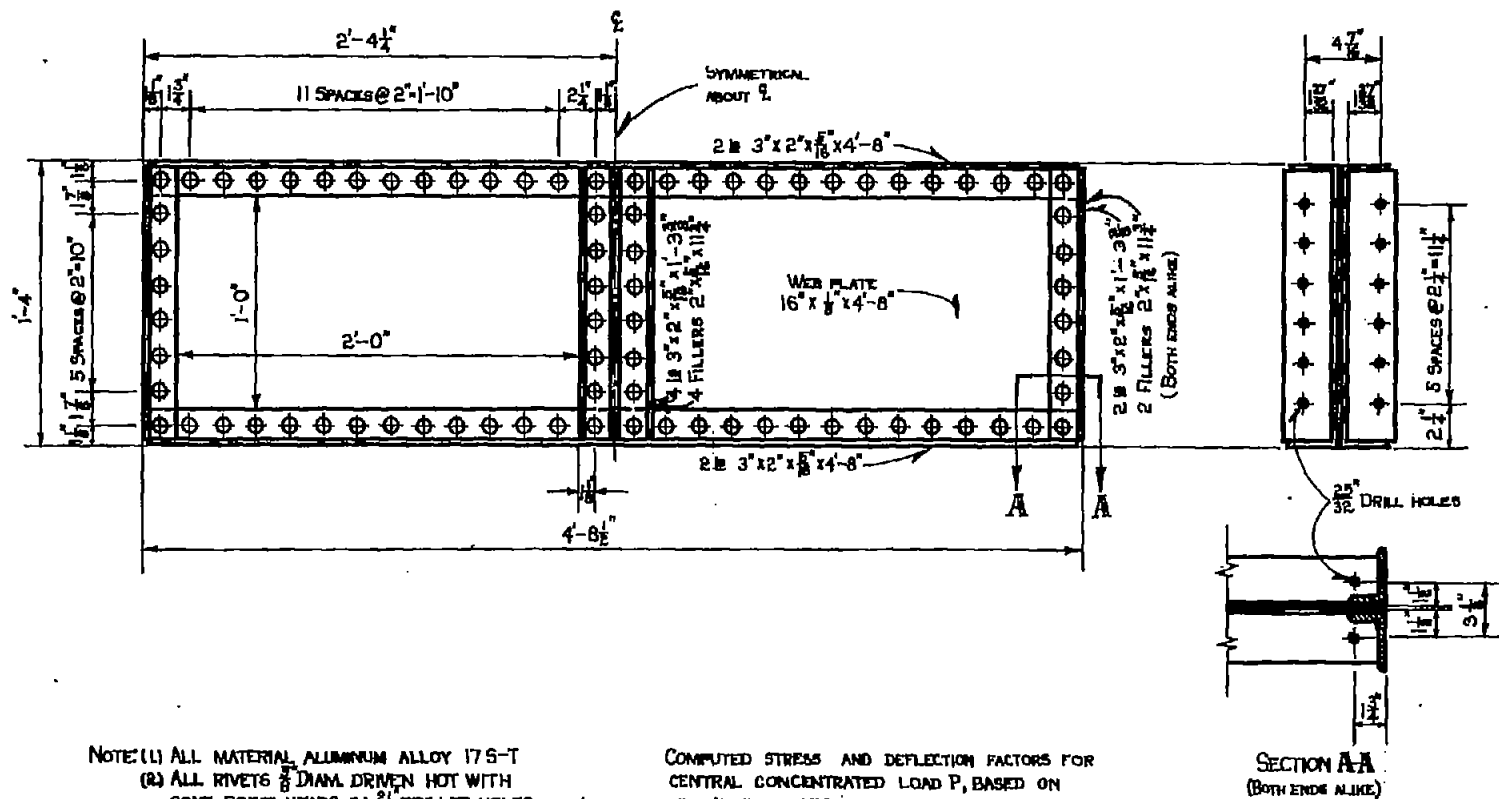
Fig. 1.- PLATE GIRDER FOR STIFFENER TESTS — SPECIMEN A.

NOTE:(1) ALL MATERIAL ALUMINUM ALLOY 17 S-T
(2) ALL RIVETS $\frac{5}{8}$ DIAM. DRIVEN HOT WITH
CONE-POINT HEADS IN $\frac{21}{32}$ DRILLED HOLES
(3) STIFFENERS HAVE CLOSE FIT BETWEEN
FLANGE ANGLES

COMPUTED STRESS AND DEFLECTION FACTORS FOR
CENTRAL CONCENTRATED LOAD P, BASED ON
ELEMENTS OF GROSS SECTION:
AVERAGE SHEAR STRESS = 0.250 P.
MAXIMUM SHEAR STRESS = 0.278 P.
MAXIMUM BENDING STRESS = 0.301 P.
MAXIMUM VERTICAL DEFLECTION = 0.00000880 P. $\begin{cases} 35\% \text{ FLEXURE} \\ 65\% \text{ SHEAR} \end{cases}$

SECTION A-A
(BOTH ENDS ALIKE)

Fig. 1

NOTE: (1) ALL MATERIAL ALUMINUM ALLOY 17S-T
(2) ALL RIVETS $\frac{5}{8}$ DIAM. DRIVEN HOT WITH
CONE-POINT HEADS IN $\frac{11}{32}$ DRILLED HOLES
(3) STIFFENERS HAVE CLOSE FIT BETWEEN
FLANGE ANGLES.
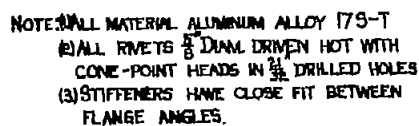
COMPUTED STRESS AND DEFLECTION FACTORS FOR
CENTRAL CONCENTRATED LOAD P, BASED ON
ELEMENTS OF GROSS SECTION:
AVERAGE SHEAR STRESS = 0.133 P
MAXIMUM SHEAR STRESS = 0.147 P
MAXIMUM BENDING STRESS = 0.176 P
MAXIMUM VERTICAL DEFLECTION = 0.00000299 P { 37% FLEXURE 63% SHEAR

SECTION A-A
(BOTH ENDS ALIKE)

Fig. 2.- PLATE GIRDER FOR STIFFENER TESTS - SPECIMEN B.

Fig. 2

SPECIMEN A

CLEAR DEPTH BETWEEN FLANGES

P

1-$\frac{3}{4}$" x $\frac{3}{4}$" x $\frac{3}{16}$" L

FIRST TEST
1-12"x24" PANEL

24" 24"

CLEAR DISTANCE BETWEEN MAIN STIFFENERS

SHEAR BUCK-LING LOAD P FOR TEST PANELS* LB.	CORRESPOND-ING AVERAGE SHEAR STRESS PSI.
26,300	6,570
38,700	9,690
66,400	16,600
106,200	26,300

SERIES I 1-$\frac{3}{4}$" x $\frac{3}{4}$" x $\frac{3}{16}$" L
2-12"x12" PANELS

2-$\frac{3}{4}$" x $\frac{3}{4}$" x $\frac{3}{16}$" LS SERIES II
3-8"x12" PANELS

SERIES III 2-1"x$\frac{3}{4}$"x$\frac{1}{8}$" LS.
4-6"x12" PANELS
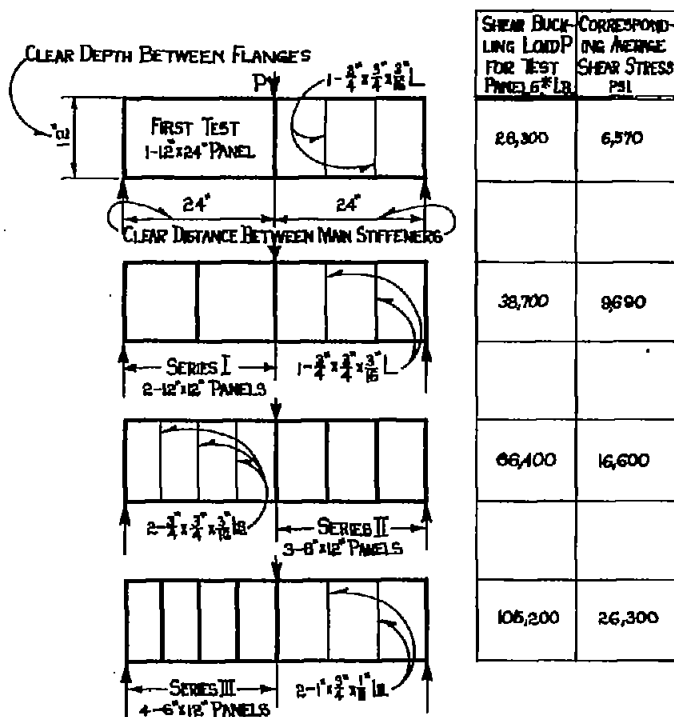
* THEORETICAL VALUE FOR PANELS WITH SIMPLY SUPPORTED EDGES

FIG. 3.- SCHEDULE OF STIFFENER TESTS
SEE TABLE II FOR SIZES IN EACH SERIES

SPECIMEN B

CLEAR DEPTH BETWEEN FLANGES

P

1-1" x $\frac{3}{4}$" x $\frac{1}{8}$" L

FIRST TEST
1-24"x48" PANEL

48" 48"

CLEAR DISTANCE BETWEEN MAIN STIFFENERS

SHEAR BUCK-LING LOAD P FOR TEST PANELS* LB.	CORRESPOND-ING AVERAGE SHEAR STRESS PSI.
12,300	1,640
18,100	2,420
31,100	4,150
49,200	6,570
101,200	13,500
174,000	23,200

SERIES I 1-1" x $\frac{3}{4}$" x $\frac{1}{8}$" L
2-24"x24" PANELS

2-1$\frac{1}{4}$" x 1" x $\frac{1}{8}$" LS SERIES II
3-16"x24" PANELS

SERIES III 2-1$\frac{1}{2}$" x 1" x $\frac{5}{16}$" LS.
4-12"x24" PANELS

2-1$\frac{1}{4}$"x1$\frac{1}{4}$"x$\frac{1}{4}$" LS SERIES IV
6-8"x24" PANELS

SERIES V 2-1$\frac{3}{4}$"x1$\frac{1}{4}$"x$\frac{1}{4}$" LS.
8-6"x24" PANELS

Fig. 3

BODY-BOUND MACHINE SCREW CONNECTION

DOUBLE-ANGLE STIFFENER   SINGLE-ANGLE STIFFENER

Size of Stiffener Angle	Width of Leg Attached to Web	Gage Distance b
$\frac{1}{2}" \times \frac{1}{2}" \times \frac{1}{16}"$	$\frac{1}{2}"$	$\frac{5}{16}"$
$\frac{3}{4}" \times \frac{3}{4}" \times \frac{3}{32}"$	$\frac{3}{4}"$	$\frac{7}{16}"$
$\frac{3}{4}" \times \frac{3}{4}" \times \frac{3}{16}"$	$\frac{3}{4}"$	$\frac{7}{16}"$
$1" \times \frac{3}{4}" \times \frac{1}{8}"$	$\frac{3}{4}"$	$\frac{7}{16}"$
$1\frac{1}{4}" \times 1" \times \frac{1}{8}"$	$1"$	$1\frac{1}{2}"$
$1\frac{1}{2}" \times 1" \times \frac{5}{32}"$	$1"$	$1\frac{1}{2}"$
$1\frac{1}{2}" \times 1" \times \frac{1}{4}"$	$1"$	$\frac{5}{8}"$
$1\frac{3}{4}" \times 1\frac{1}{4}" \times \frac{1}{4}"$	$1\frac{1}{4}"$	$\frac{5}{8}"$

No. 6-32 N.C. (0.138"Dia.) Aluminum Alloy 17 S-T Screws for $\frac{1}{2}" \times \frac{1}{2}" \times \frac{1}{16}"$ Ls
No. 10-32 N.F. (0.190"Dia.) Aluminum Alloy 17 S-T Screws for All Other Sizes
All Holes Drilled & Reamed From Templates To Body Size of Screws

Fig. 4.- Intermediate Stiffeners for Plate Girder Specimens A & B

Fig. 4
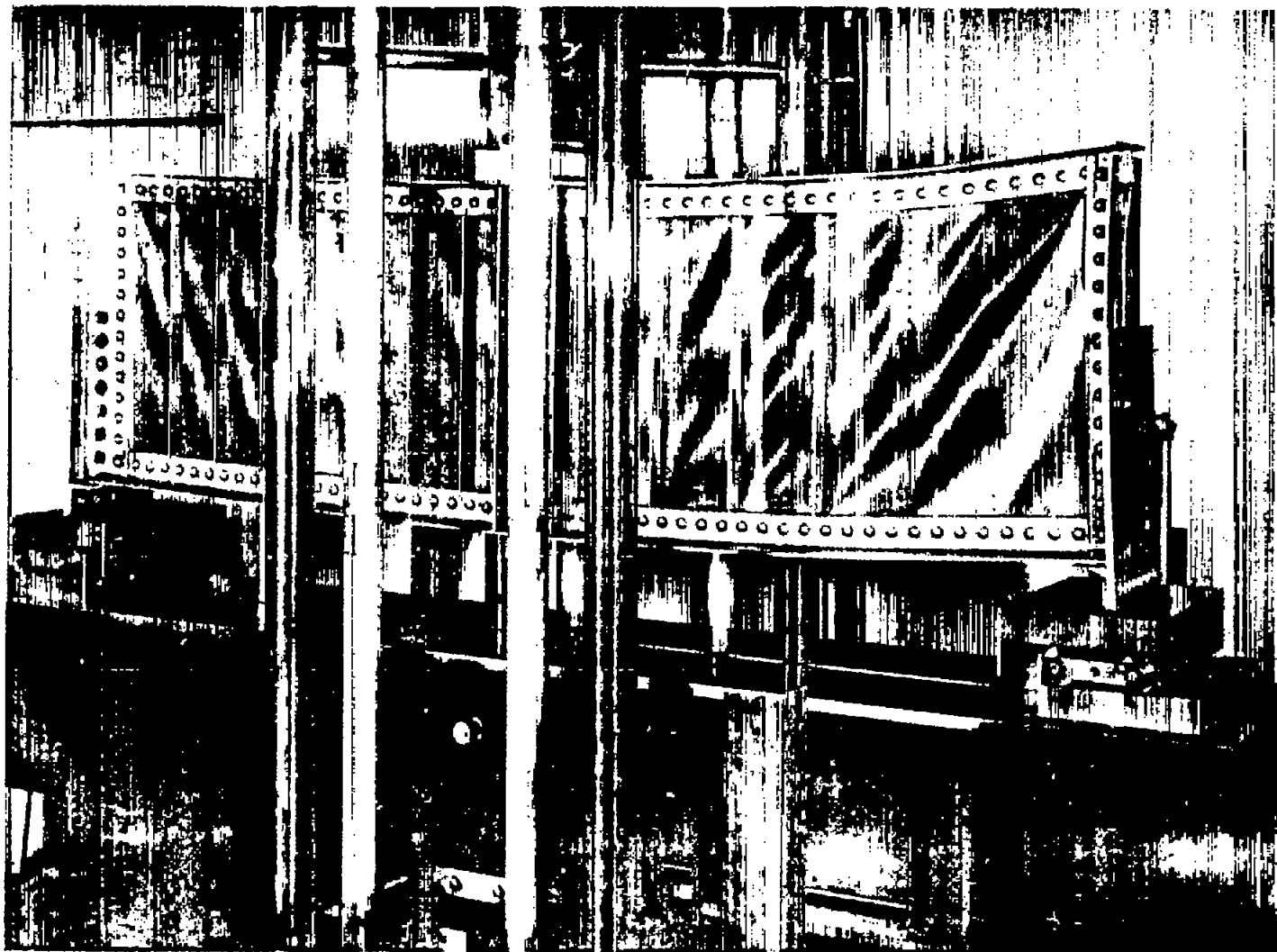
Figure 5.- Girder tests in 300,000-pound-capacity Amsler machine
(specimen B after failure).

Fig. 5

Fig. 6
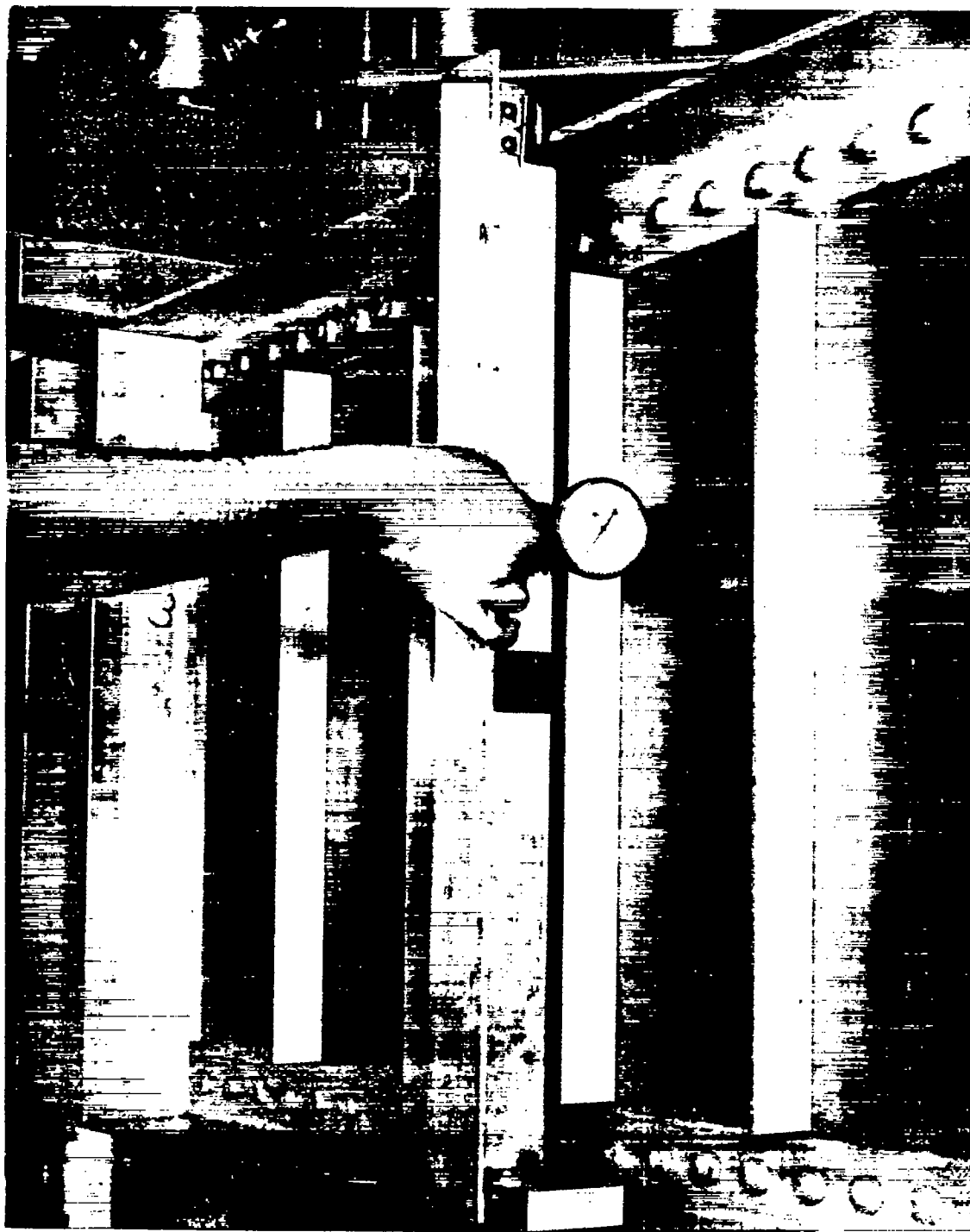
Figure 6.- Apparatus for measurement of lateral deflections
(reference angle held against flanges by tension
springs hooked over opposite edges).
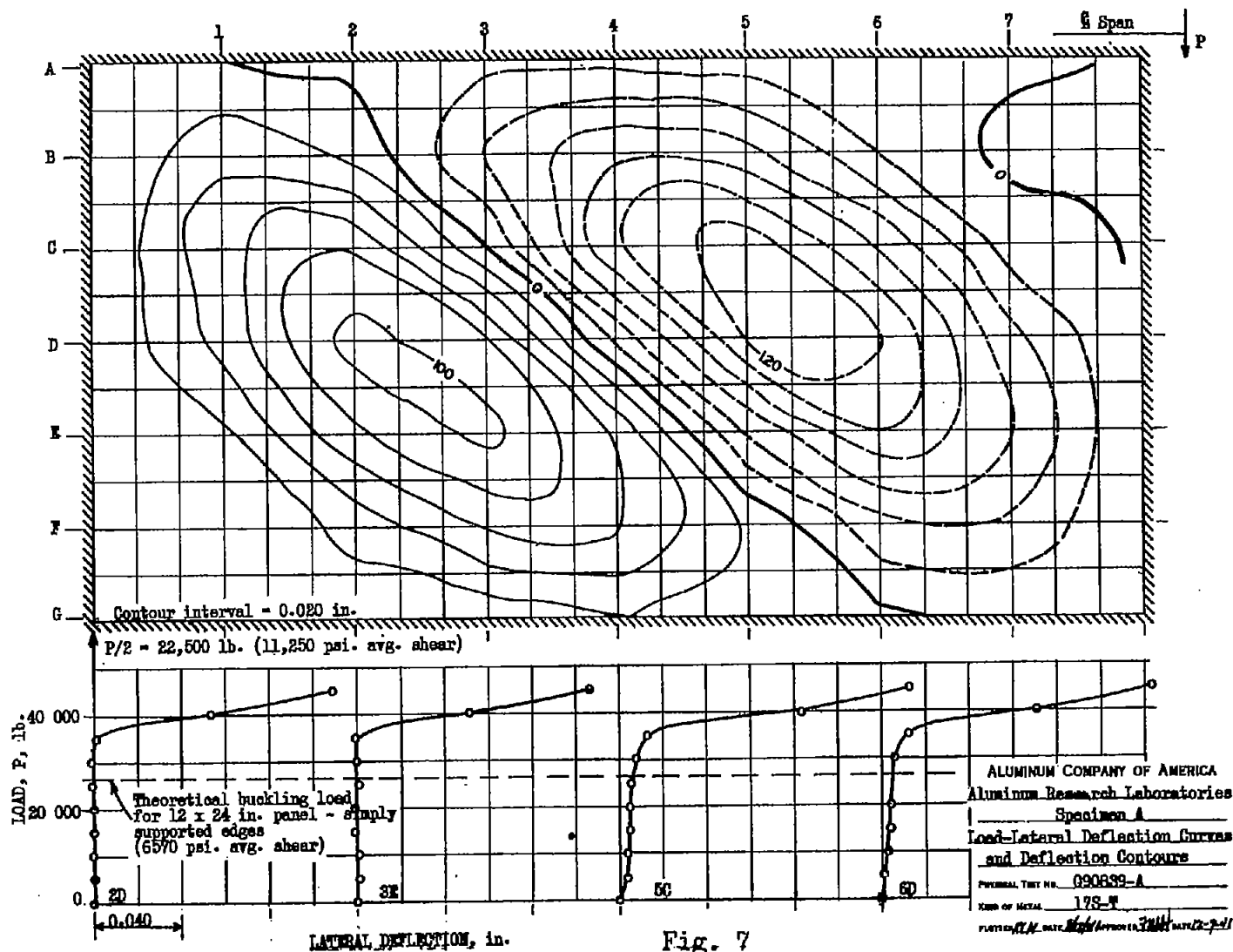
Fig. 7

Load-Lateral Deflection Curves and Deflection Contours

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
Specimen A
Load-Lateral Deflection Curves
and Deflection Contours

Fig. 7

Contour interval = 0.020 in.

P/2 = 25,000 lb. (12,500 psi. avg. shear)

Theoretical buckling load for
12 x 12 in. panels - simply
supported edges
(9690 psi. avg. shear)

LOAD, P, lb.

40 000

20 000

0

0.040

LATERAL DEFLECTIONS, in.   Fig. 8

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
Specimen A
Load-Lateral Deflection Curves
and Deflection Contours
PHYSICAL TEST NO.   090839-A
KIND OF METAL   17S-T

Fig. 8
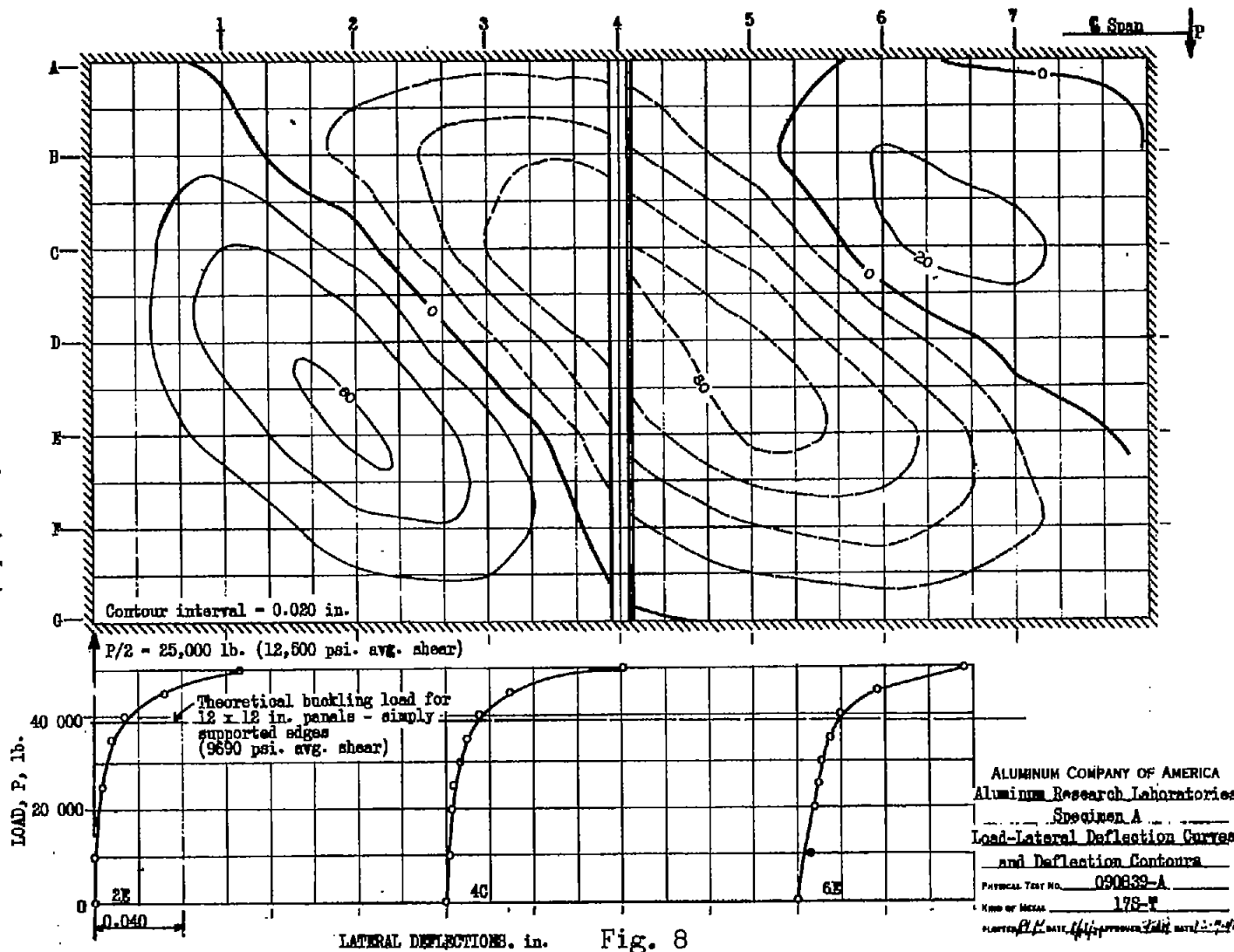
$A_v = 2 - .202 = .404$
$dt = .8 = \frac{1}{8} = 1.00$
$A_v/dt = .404$
$h/t \sim 120$

P = 90 000 lb.

All intermediate stiffeners: 2 - 1 x 3/4 x 1/8 in. angles
4 - 6 x 12 in. panels

3 - 8 x 12 in. panels

1 2 3 4 5 6 7   8 9 10 11 12 13 14 15 16

A
B
C
D
E
F
G

Contour interval = 0.050 in.

Contour interval = 0.050 in.

P/2

Theoretical buckling load for 6 x 12 in. panels
with simply supported edges = 105 200 lb

Theoretical buckling load for 8 x 12 in. panels -
simply supported edges (16 600 psi avg. shear)
Load for 15 000
psi avg. shear

P/2

80 000

Computed

60 000

LOAD, P, lb

Vertical deflection at
center of span

40 000

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
New Kensington

Specimen A   Load-Deflection
Curves and Deflection
Contours

20 000

MATERIAL TEST No. 090839-A
KIND OF METAL 17S-T
PLOTTED BY

2D 3C 5B 6D 8D 10D 12C 13E 14D 16C

0

←0.1→   LATERAL AND VERTICAL DEFLECTIONS, in.   Fig. 9   ←0.1→

Fig. 9

$A_v = 2 \cdot .69 = 1.38$

$dt = 8 \times \frac{1}{8} = 1.00$

$A_v/dt = \underline{1.38}$

$1./t \cdot 2 = \underline{2.00}$

P = 160 000 lb

All intermediate stiffeners: 2 - 1-3/4 x 1-1/4 x 1/4 in. angles
8 - 6 x 24 in. panels

6 - 8 x 24 in. panels

Contour interval = 0.020 in.   Contour interval = 0.020 in.

P/2

Theoretical buckling load for 6 x 24 in. panels -
simply supported edges (23 200 psi avg shear)

Theoretical buckling load for 8 x 24 in. panels -
simply supported edges (13 500 psi avg shear)

Load for 16 000
psi avg shear

Computed

Vertical deflection at
center of span

160 000

120 000

80 000

40 000

LOAD, P, lb

13B   14C   15B   16B   17C   18B   22C   24D

0.1   0.2

LATERAL AND VERTICAL DEFLECTIONS, in.

Fig. 10

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
New Kensington
Specimen B. Load-Deflection
Curves and Deflection
Contours
PHYSICAL TEST No. 090839-A
KIND OF METAL   17S-T
PLOTTED BY   DATE   APPROVED   DATE

Fig. 10

Fig. 11

Load-Lateral Deflection Curves from Tests of Stiffened Plate-Girder Web

Theoretical buckling
load for 8 x 12 in.
panels - simply supported
edges (16,600 psi. avg.
shear)

Load for 16,000 psi. avg.
shear

17S-T Plate Girder - Spec. A
Web thickness = 1/8"

12" x 24" test panel bounded
by 3" x 2" x 5/16" angles

LOAD P IN POUNDS

Test No.	Description of Intermediate Stiffener Angles*
1	3/4 x 3/4 x 3/32 - one side
2	3/4 x 3/4 x 3/32 - both sides
3	3/4 x 3/4 x 3/16 - one side
4	3/4 x 3/4 x 3/16 - both sides
5	1 x 3/4 x 1/8 - one side
6	1 x 3/4 x 1/8 - both sides

* Short legs attached to web.

I = moment of inertia, in.^{4}

P

2'-4-1/4"

3 panels @ 8" = 24"   P/2

Average of Maximum Web Deflections
Midway between Stiffeners

Average of Maximum Stiffener
Deflections

0.020

0.020

LATERAL DEFLECTIONS IN INCHES

Fig. 12

LATERAL DEFLECTIONS IN INCHES

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
Load-Lateral Deflection Curves
from Tests of Stiffened
Plate-Girder Web

Physical Test No. 090639-A

Kind of Metal 17S-T

Fig. 12

Fig. 14

LATERAL DEFLECTIONS IN INCHES

LOAD P IN POUNDS

Theoretical buckling load for
24 x 24 in. panels - edges 50% fixed
(3240 psi. avg. shear)

17S-T Plate Girder - Spec. B
Web thickness - 1/8"
24" x 48" test panel bounded
by 4" x 3" x 3/8" angles

Theoretical buckling load for
24 x 24 in. panels - simply
supported edges
(2420 psi. avg. shear)

I - moment of inertia, in.⁴

I = moment of inertia, in.^{4}

Average of Maximum Web Deflections
Midway between Stiffeners

Average of Maximum Stiffener
Deflections

Test No.	Description of Intermediate Stiffener Angles*
①	1/2 x 1/2 x 1/16 - one side
②	1/2 x 1/2 x 1/16 - both sides
③	3/4 x 3/4 x 3/32 - one side
④	3/4 x 3/4 x 3/32 - both sides
④A	3/4 x 3/4 x 3/32 - both sides (ck.test)
⑤	3/4 x 3/4 x 3/16 - one side
⑥	3/4 x 3/4 x 3/16 - both sides
⑦	1 x 3/4 x 1/8 - one side
⑧	1 x 3/4 x 1/8 - both sides
⑨	1-1/4 x 1 x 1/8 - one side
⑩	1-1/4 x 1 x 1/8 - both sides
⑩A	1-1/4 x 1 x 1/8 - both sides (ck.test)

* Short legs attached to web.

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Load-Lateral Deflection Curves
from Tests of Stiffened
Plate-Girder Web

PHYSICAL TEST No. 090839-A
KIND OF METAL 17S-T
PLOTTER BY ___ DATE ___ APPROVED ___ DATE ___

LATERAL DEFLECTIONS IN INCHES

Fig. 14

Fig. 15

Theoretical buckling load for
16 x 24 in. panels - edges 50% fixed
(5560 psi. avg. shear)

17S-T Plate Girder - Spec. B
Web thickness = 1/8"
24" x 48" test panel bounded
by 4" x 3" x 3/8" angles

Theoretical buckling load
for 16 x 24 in. panels -
simply supported edges
(4150 psi. avg. shear)

Average of Maximum Web Deflections
Midway between Stiffeners

Average of Maximum Stiffener
Deflections

Test No.	Description of Intermediate Stiffener Angles*
①	3/4 x 3/4 x 3/16 - one side
②	3/4 x 3/4 x 3/16 - both sides
③	1 x 3/4 x 1/8 - one side
④	1 x 3/4 x 1/8 - both sides
⑤	1-1/4 x 1 x 1/8 - one side
⑥	1-1/4 x 1 x 1/8 - both sides
⑦	1-1/2 x 1 x 5/32 - one side
⑧	1-1/2 x 1 x 5/32 - both sides

* Short legs attached to web.
I = moment of inertia, in.^{4}

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Load-Lateral Deflection Curves
from Tests of Stiffened
Plate-Girder Web

Physical Test No. 090839-A
Kind of Metal 17S-T

LATERAL DEFLECTIONS IN INCHES

LATERAL DEFLECTIONS IN INCHES

Fig. 16

Theoretical buckling load for 12" x 24" panels - edges 50% fixed (8800 psi. avg. shear)

Theoretical buckling load for 12" x 24" panels - simply supported edges (6570 psi. avg. shear)

178-T Plate Girder - Spec. B
Web thickness - 1/8"
24" x 48" test panel bounded by 4" x 3" x 3/8" angles

LOAD P IN POUNDS

Average of Maximum Web Deflections Midway between Stiffeners

Average of Maximum Stiffener Deflections

LATERAL DEFLECTIONS IN INCHES

LATERAL DEFLECTIONS IN INCHES

Test No.	Description of Intermediate Stiffener Angles*
①	1 x 3/4 x 1/8 - one side
②	1 x 3/4 x 1/8 - both sides
③	1-1/4 x 1 x 1/8 - one side
④	1-1/4 x 1 x 1/8 - both sides
⑤	1-1/2 x 1 x 5/32 - one side
⑥	1-1/2 x 1 x 5/32 - both sides
⑦	1-1/2 x 1 x 1/4 - one side
⑧	1-1/2 x 1 x 1/4 - both sides
⑨	1-3/4 x 1-1/4 x 1/4 - one side
⑩	1-3/4 x 1-1/4 x 1/4 - both sides

* Short legs attached to web.
I - moment of inertia, in.^{4}

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Load-Lateral Deflection Curves
from Tests of Stiffened
Plate-Girder Web.

PHYSICAL TEST No. 090839-A
KIND OF METAL 178-T

FIG. 16

Fig. 17

Fig. 17

Fig. 18

Load for 16,000 psi. avg. shear

17S-T Plate Girder- Spec. B
Web thickness = 1/8"
24" x 48" test panel bounded
by 4" x 3" x 3/8" angles

Theoretical Buckling Load for
6" x 24" panels - simply supported edges
(23,300 psi. shear)

Test No.	Description of Intermediate Stiffener Angles*
①	1-1/2 x 1 x 5/32 - one side
②	1-1/2 x 1 x 5/32 - both sides
③	1-1/2 x 1 x 1/4 - one side
④	1-1/2 x 1 x 1/4 - both sides
⑤	1-3/4 x 1-1/4 x 1/4 - one side
⑥	1-3/4 x 1-1/4 x 1/4 - both sides

* Short legs attached to web.
I = moment of inertia, in.[4]

Average of Maximum Web Deflections Midway
between Stiffeners

Average of Maximum Stiffener
Deflections

LATERAL DEFLECTIONS IN INCHES

LOAD P IN POUNDS

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories

Load-Lateral Deflection Curves
from Tests of Stiffened
Plate-Girder Web

PHYSICAL TEST No. 090839-A
KIND OF METAL 17S-T
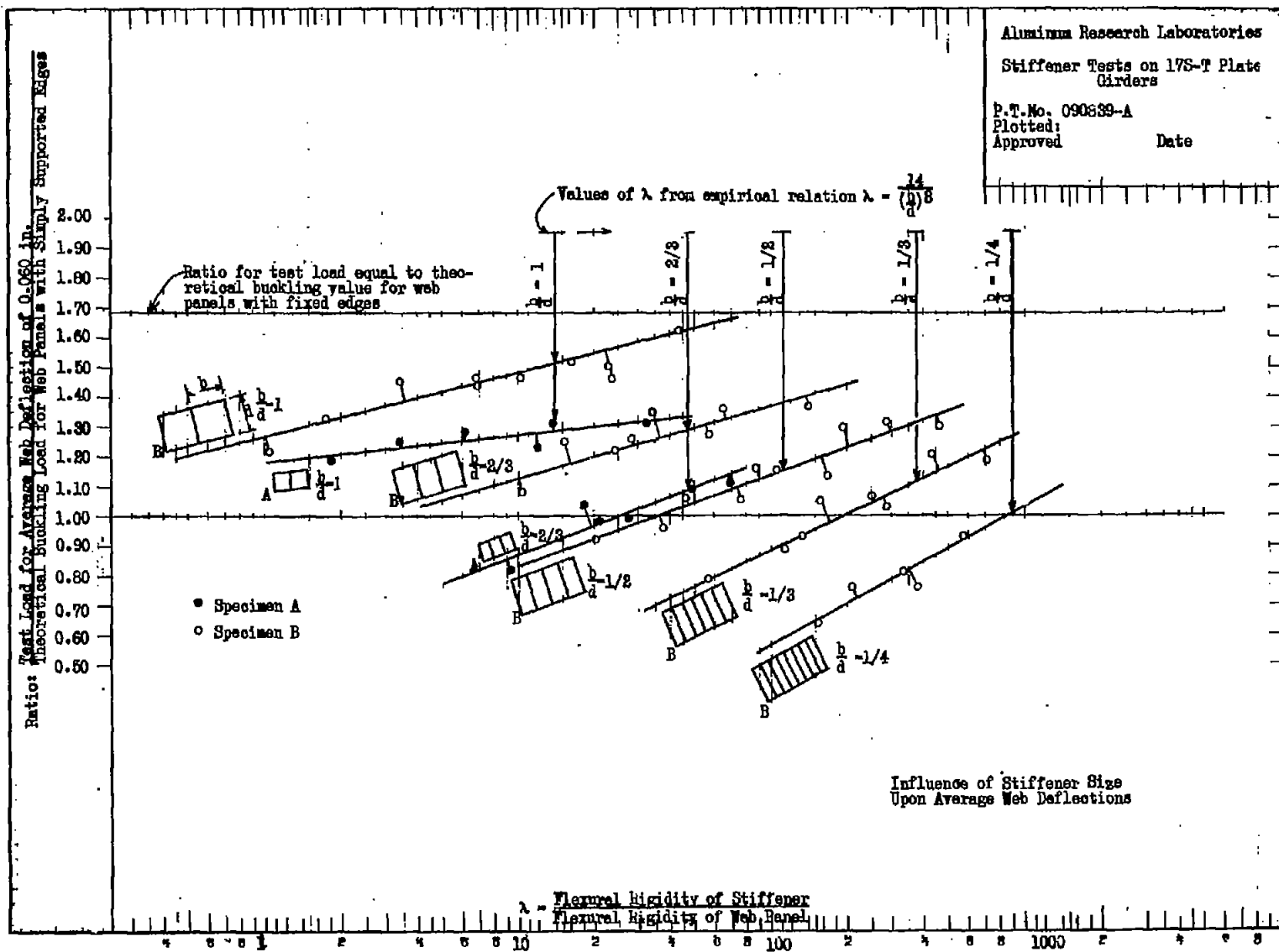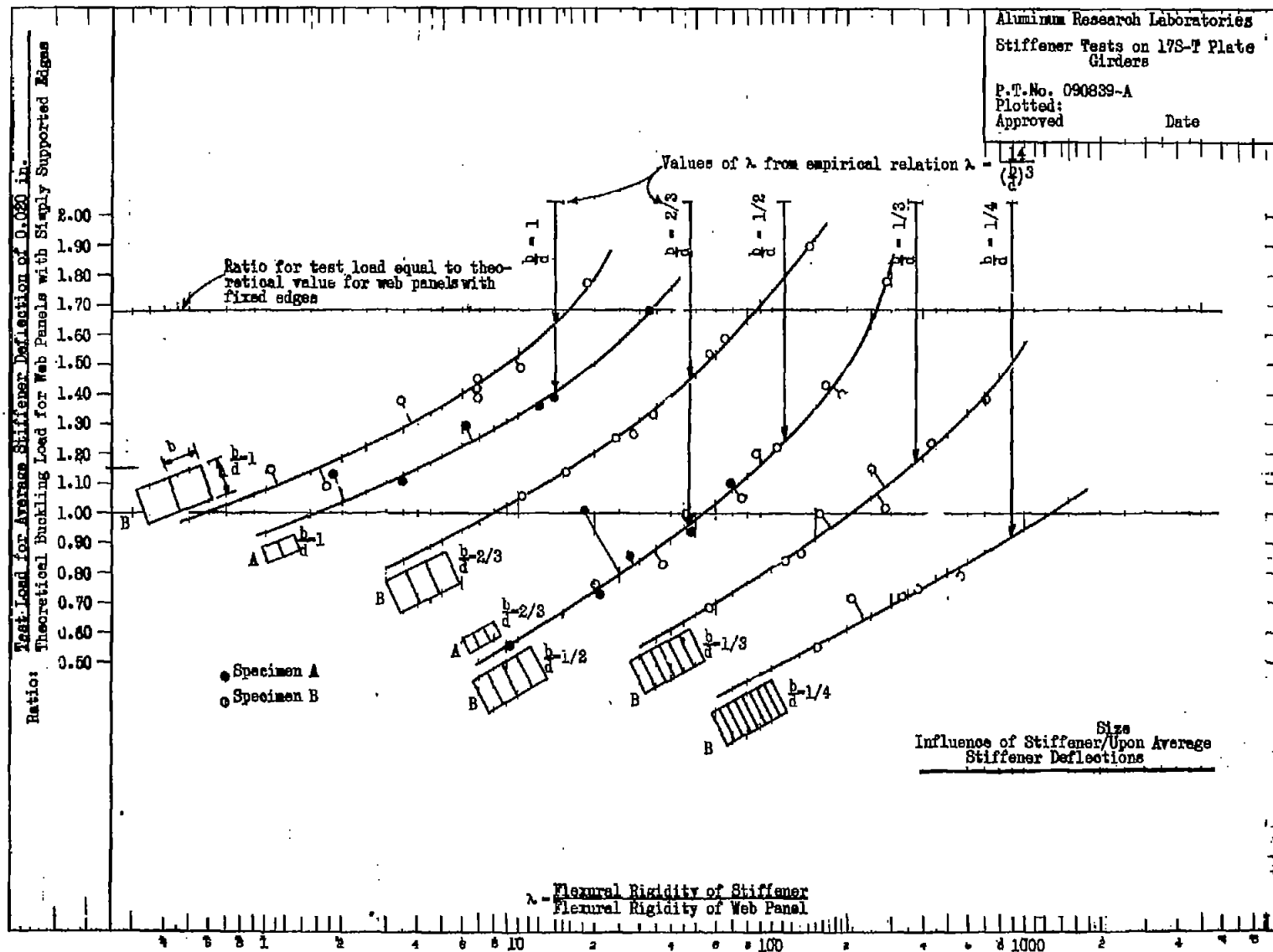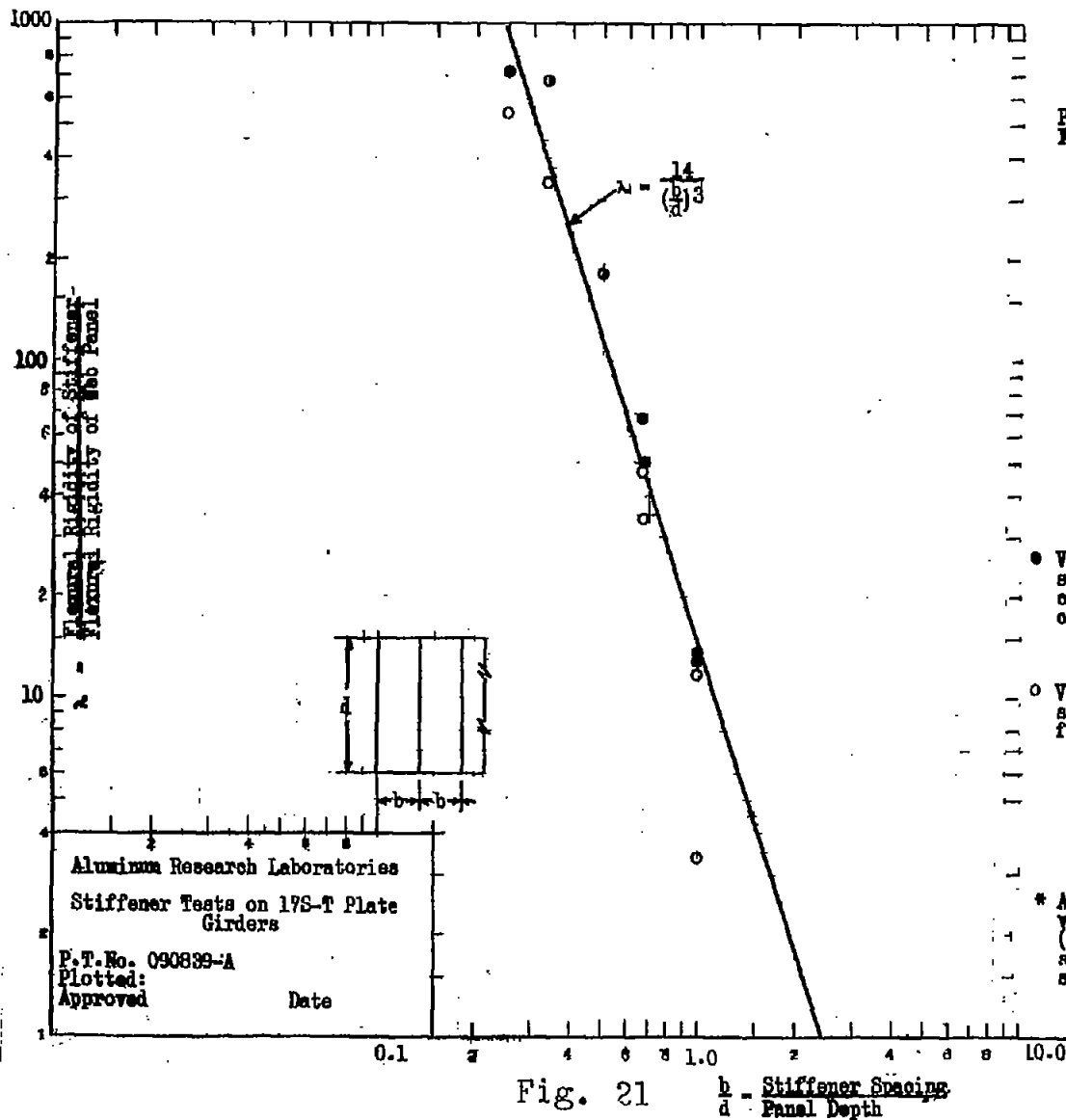
Fig. 19

Fig. 19

Fig. 20

Fig. 20

Proposed Empirical Relation Between Ratios of
Flexural Rigidity λ and Proportions of Web Panel

$$\lambda = \frac{12EI(1-\mu^2)}{Ebt^3}$$

where I = moment of inertia of stiffener, in.[4]
       E = modulus of elasticity, psi.,
       b = stiffener spacing, in.,
       t = web thickness, in.
and μ = Poisson's ratio = 1/3

● Values of λ obtained from tests in which average
  stiffener deflections were approximately 25 per
  cent of average web deflections for assumed
  critical loads.*

○ Values of λ obtained from tests in which average
  stiffener deflections were approximately 0.020 in.
  for assumed critical loads.*

* Assumed critical loads equal to theoretical buckling
  values for web panels with edges 50 per cent fixed,
  (34 per cent greater than buckling loads for simply
  supported edges) but not exceeding average shear
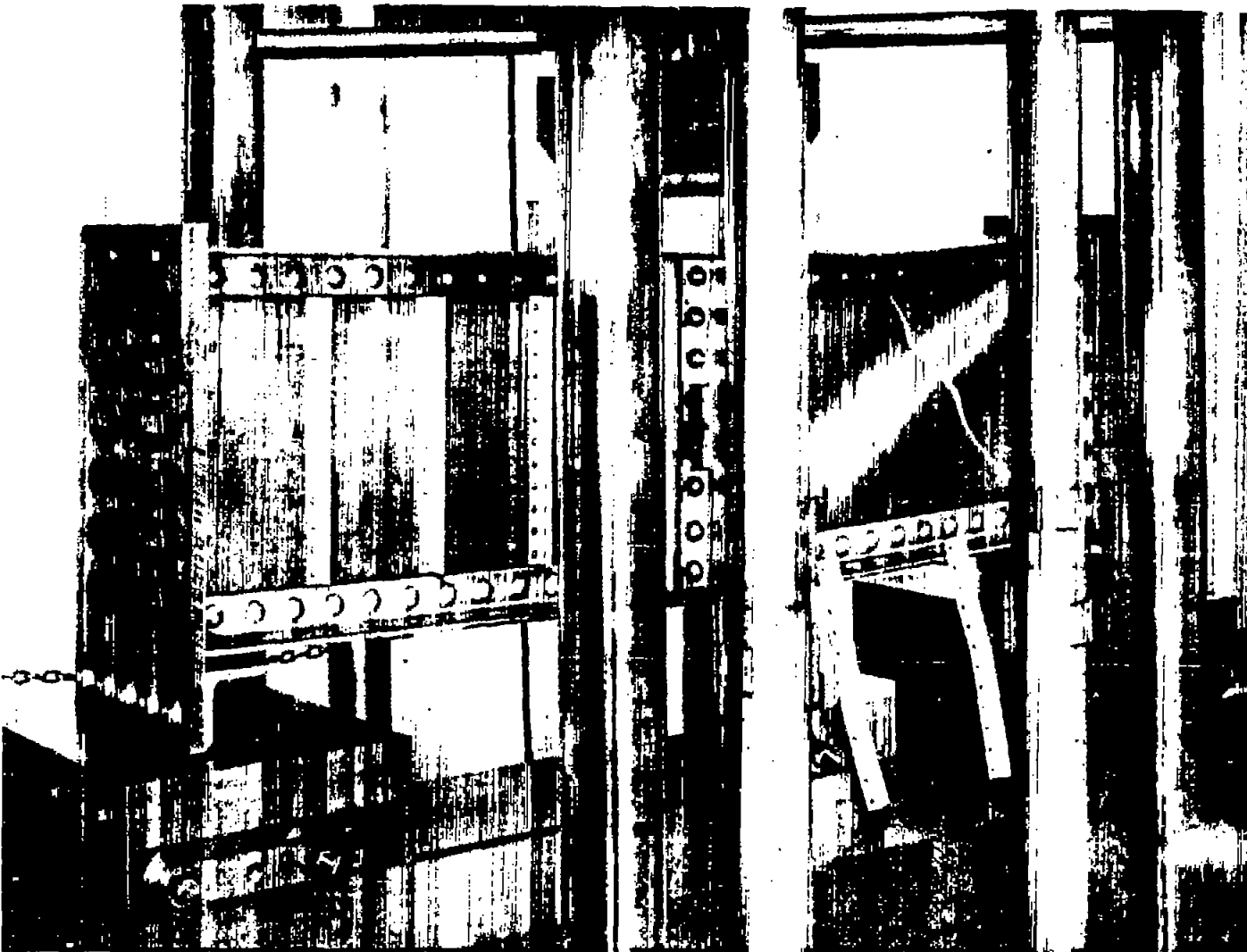 stress of 16,000 psi.

$\lambda = \frac{14}{(\frac{b}{d})^3}$

Flexural Rigidity of Stiffener
Flexural Rigidity of Web Panel

Aluminum Research Laboratories

Stiffener Tests on 17S-T Plate
Girders

P.T.No. 090839-A
Plotted:
Approved   Date

Fig. 21 $\frac{b}{d} = \frac{\text{Stiffener Spacing}}{\text{Panel Depth}}$

Fig. 21

Figure 22.- Specimen A after failure of stiffener connections and fracture of web under load of 93,300 pounds.
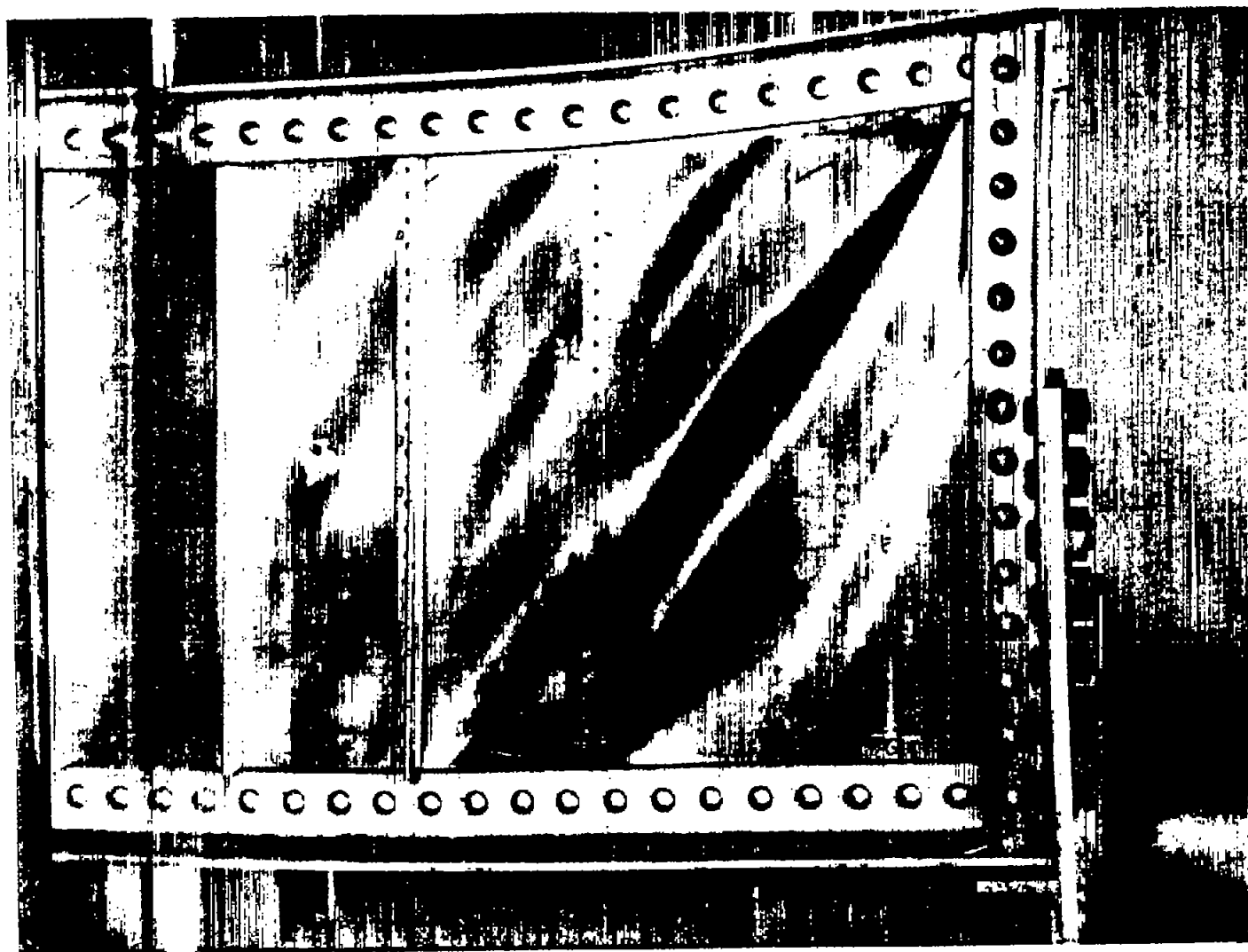
Fig. 22

Figure 23.— Specimen B after failure of stiffener connections and collapse of web under load of 191,500 pounds.
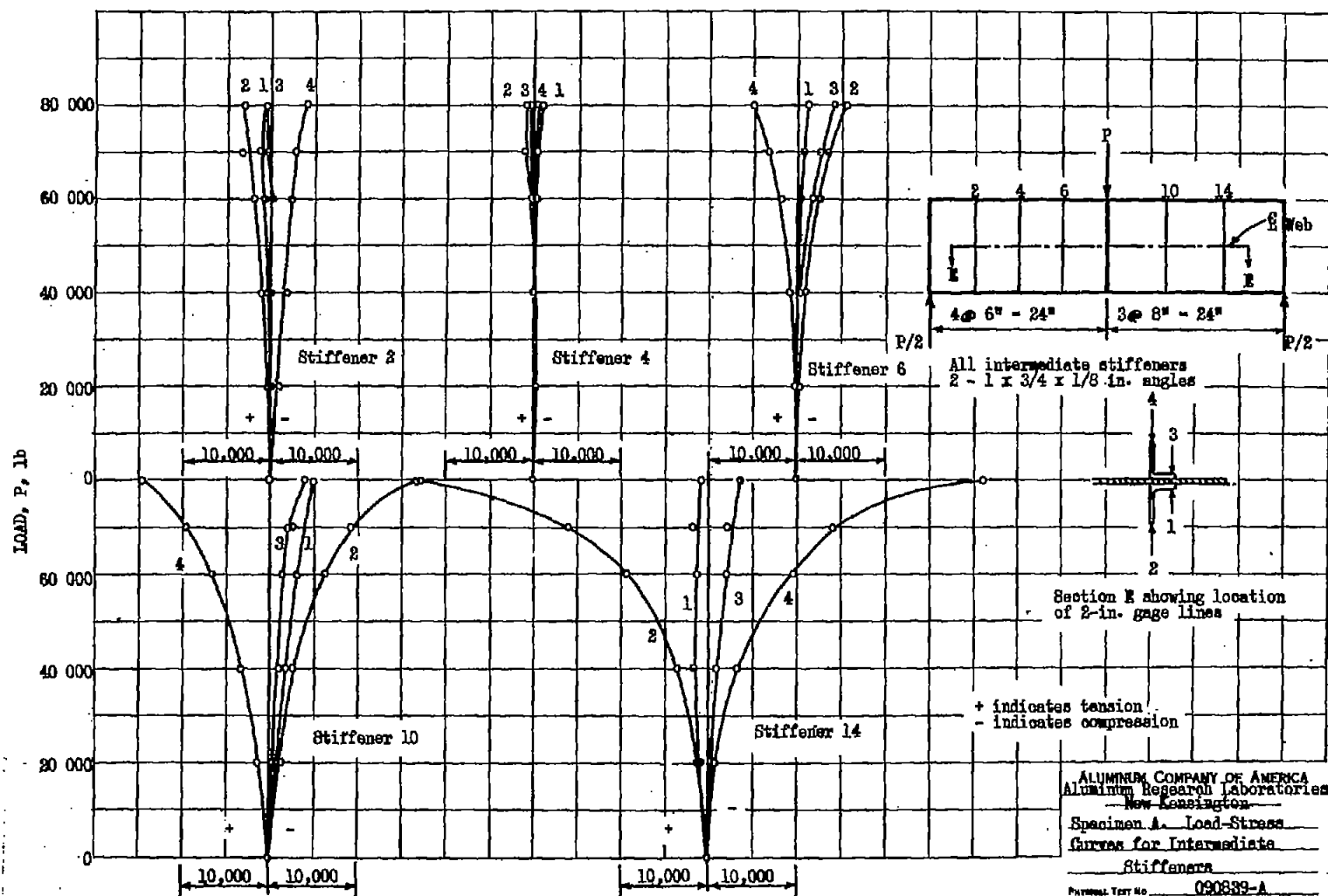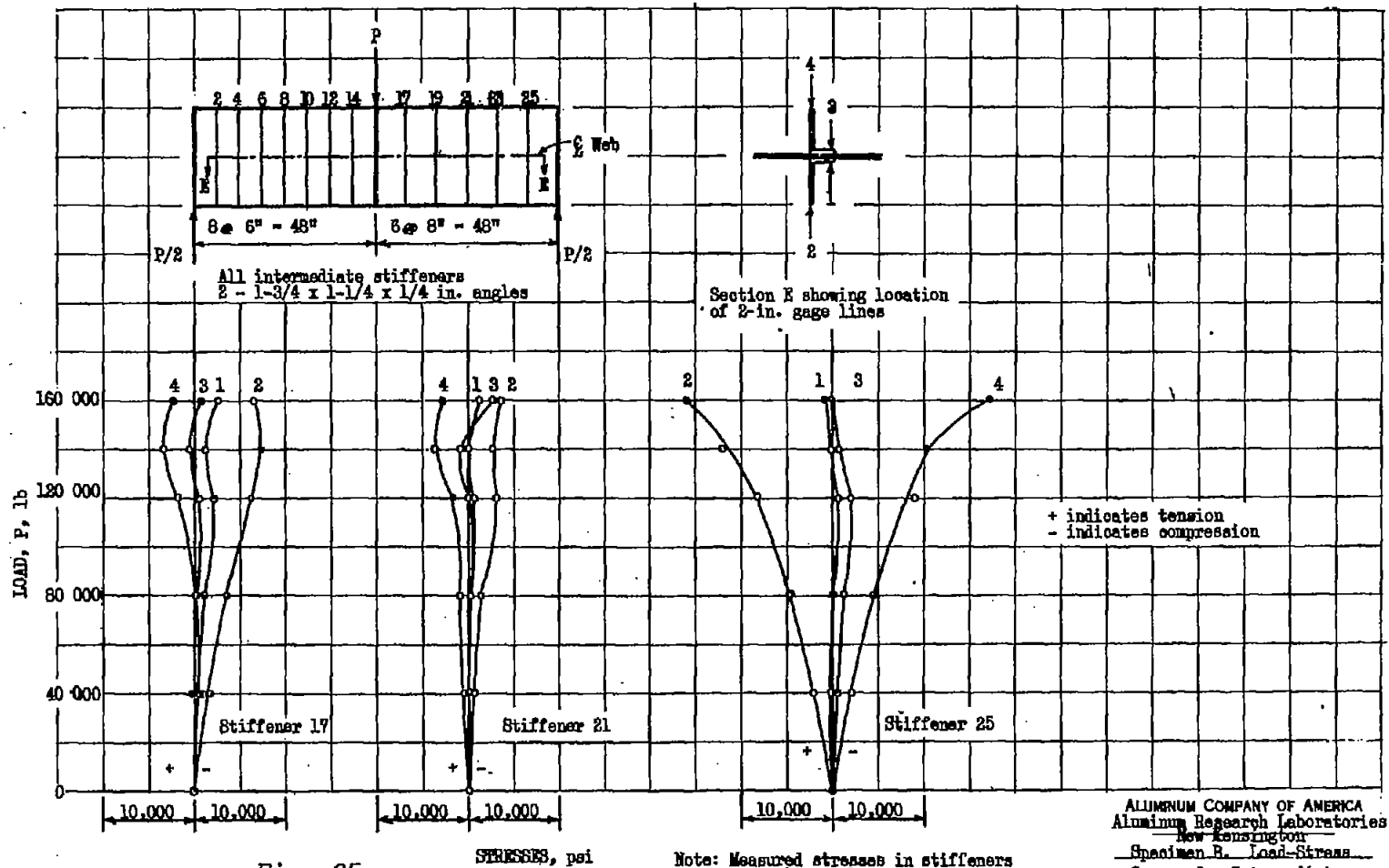
Fig. 23

Fig. 24   STRESSES, psi

Fig. 24

Fig. 25

STRESSES, psi

Note: Measured stresses in stiffeners
2, 6, 10 and 14 of negligible magnitude

+ indicates tension
- indicates compression

Section E showing location
of 2-in. gage lines

All intermediate stiffeners
2 - 1-3/4 x 1-1/4 x 1/4 in. angles

Stiffener 17   Stiffener 21   Stiffener 25

LOAD, P, lb

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
New Kensington
Specimen B. Load-Stress
Curves for Intermediate
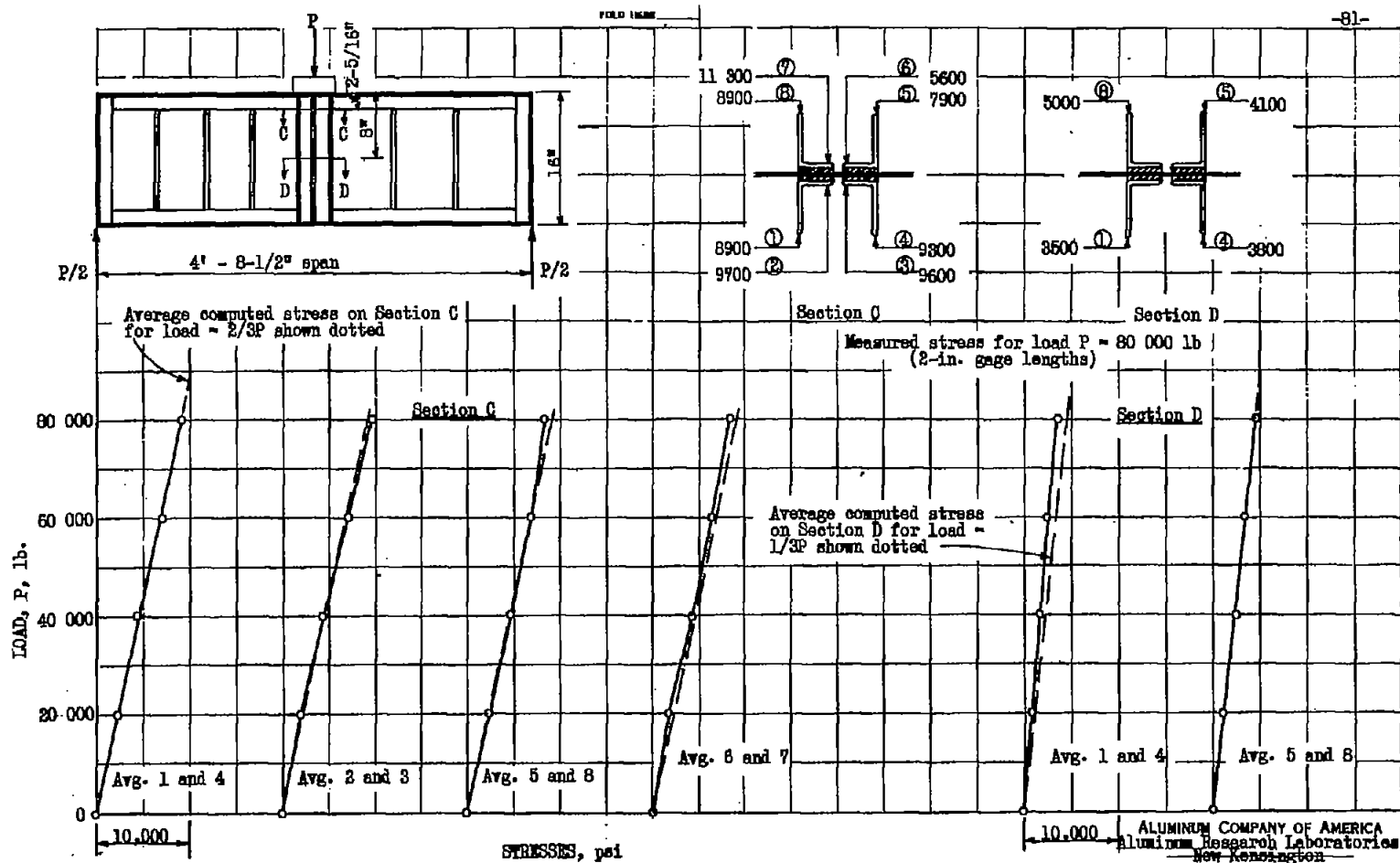Stiffeners

PHYSICAL TEST NO.   090839-A
KIND OF METAL   17S-T

Fig. 25

Fig. 26

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
New Kensington
Spec. A Load-Stress Curves
and Stress Distribution for
Load-Bearing Stiffeners

PHYSICAL TEST No. 090839-A
KIND OF METAL 17S-T

Fig. 26

Measured Stresses for Load P = 60 000 lb
(2-in. gage lengths)

Section A

Section B

LOAD, P, lb

Computed Stresses
based on Gross I
Shown Dotted

Section A

Section B

Avg. 1 and 7   Avg. 2,3,5,6   4

Avg. 1,2,4,5   3
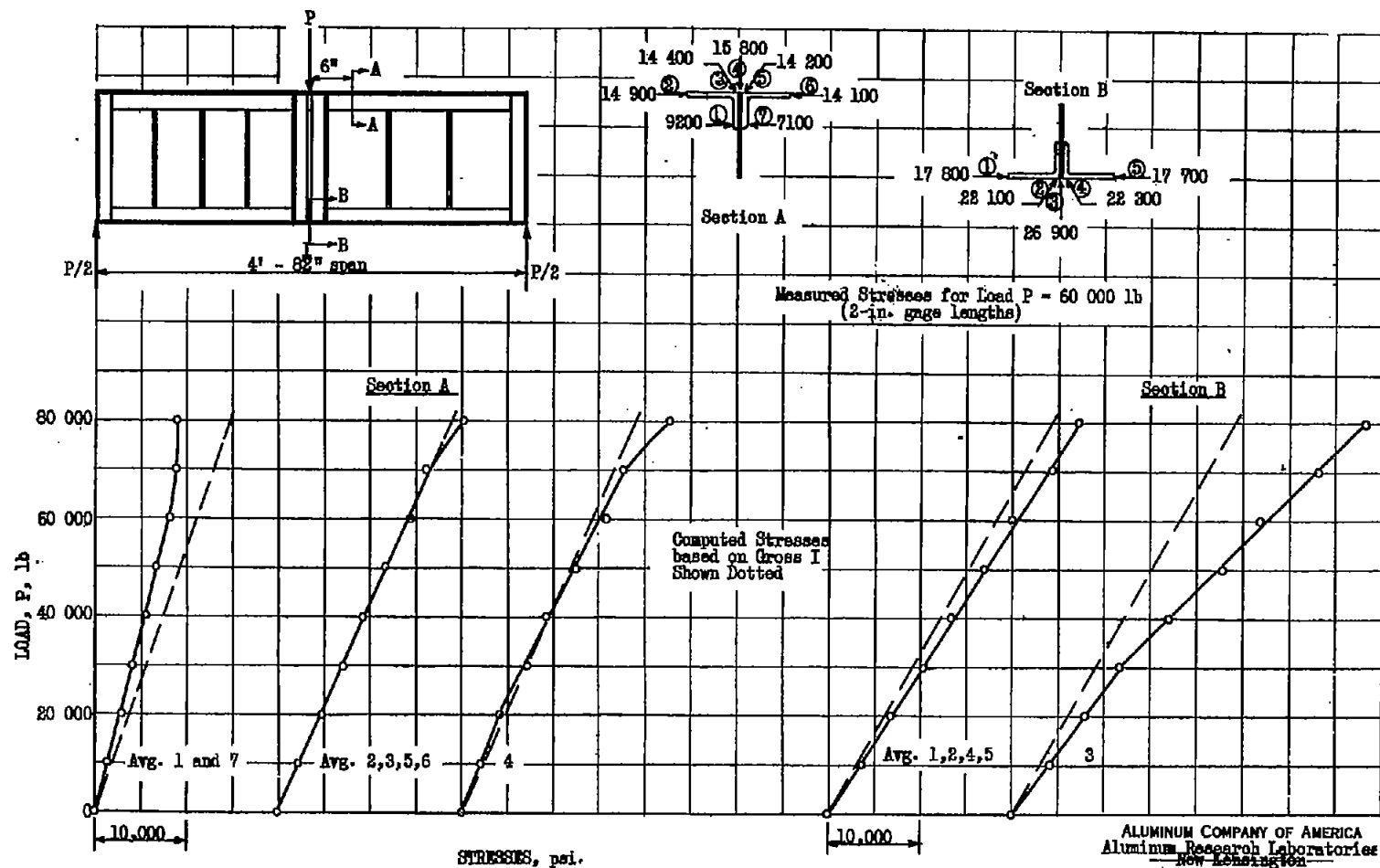
10,000

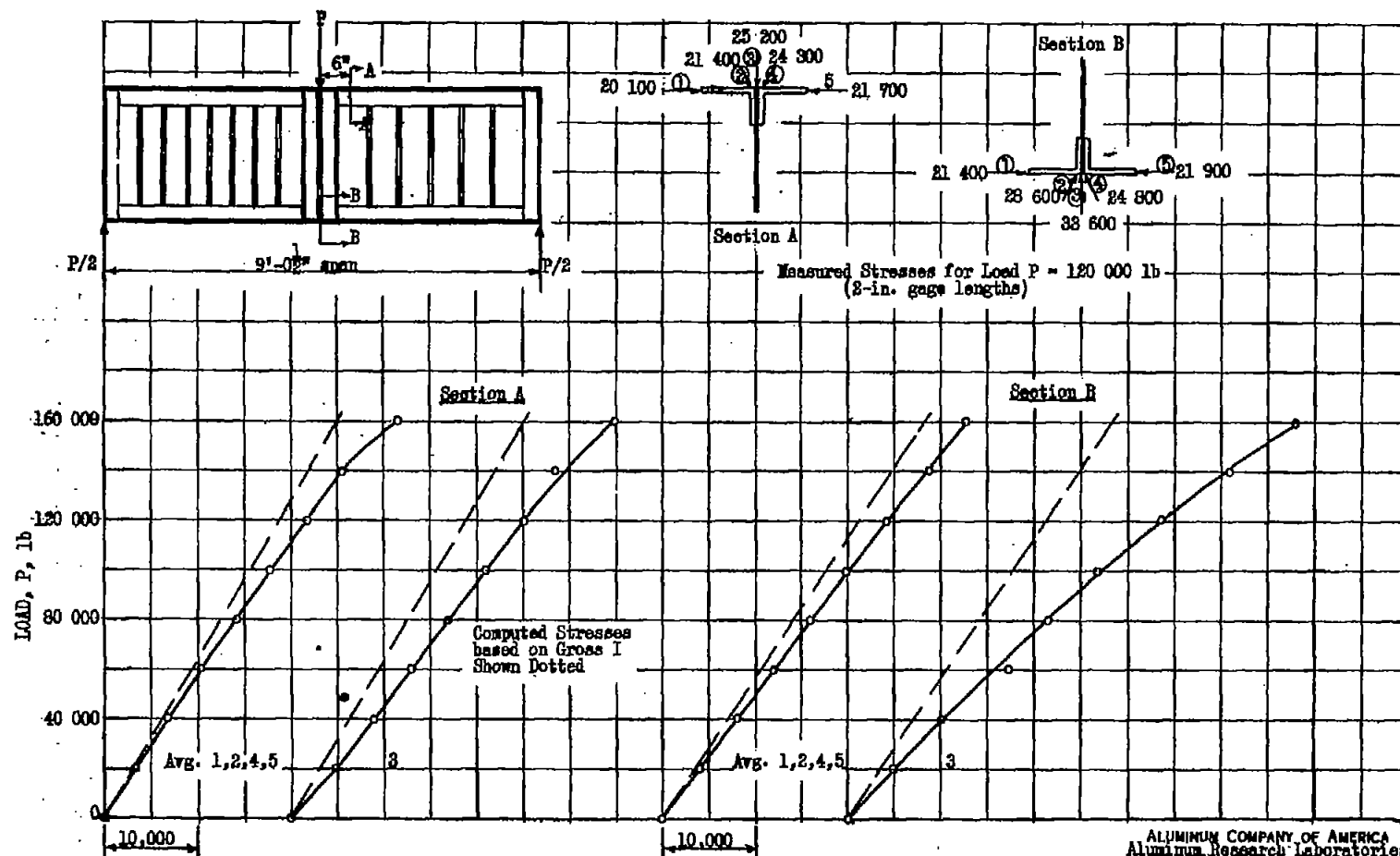10,000

STRESSES, psi.

Fig. 28

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
New Kensington
Specimen A. Load-Stress
Curves and Stress Distri-
bution for Girder Flanges

Physical Test No. 090839-A

Kind of Metal 17S-T

Fig. 28

Measured Stresses for Load P = 120 000 lb
(2-in. gage lengths)

Fig. 29

STRESSES, psi

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
New Kensington
Specimen B, Load-Stress
Curves and Stress Distri-
bution for Girder Flanges
Physical Test No. 090839-A
Kind of Metal 17S-T

Fig. 29