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OVERALL INTRODUCTION AND
PART 1: THE TEST GIRDER

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BUNG-TSENG YEN
JOHN A. MUELLER
BRUNO THÜRLIMANN
WEB BUCKLING TESTS ON WELDED PLATE GIRDERS

including

Foreword, Acknowledgement, Table of Contents, Nomenclature, Literature Survey and References, Part 1: The Test Girders

by

Basler, K., Yen, B.T., Mueller, J.A., and Thürlimann, B.

Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania
May 1960
FOREWORD

An extensive experimental and theoretical investigation was carried out at Lehigh University with the purpose of determining the carrying capacity of plate girders whose web slenderness ratios were beyond the limits stipulated by present specifications. While the theoretical study will soon appear in the Proceedings of the ASCE, a complete report on the experimental work will be published in this Journal in four parts:

Part 1: "The Test Girders"
Part 2: "Tests on Plate Girders Subjected to Bending"
Part 3: "Tests on Plate Girders Subjected to Shear"
Part 4: "Tests on Plate Girders Subjected to Combined Bending and Shear."

The objective of this investigation was to determine the postbuckling strength of thin web plate girders. The design of transversely stiffened plate girders presently is limited to girders whose web depth to web thickness ratios do not exceed the value of 170. This limit was derived from the web buckling theory. But in discussing the application of the theoretical buckling formulae, Ref. 253, p. 415, Timoshenko suggests using a factor of safety of only 1.5
against web buckling, since this occurrence does not cause immediate failure of the girder. Also, similar considerations are advanced by many foreign plate girder specifications to justify their factors of safety against web buckling. For instance, the German specifications, Ref. 52, require no more than 1.35 or 1.25 as a factor of safety under principal loading, and principal and secondary loading respectively. The corresponding Swiss specifications recommend values of 1.3, 1.5, and 1.8 for plate girders used in buildings, highway bridges, and railroad bridges respectively. In Belgium, Massonnet suggests a factor of safety of 1.35 against buckling due to shear and 1.15 against buckling due to bending, Ref. 162, p. 81. Thus, not only the safety factors differ but they also seem to depend on loading conditions and other factors. In order to clarify this uncertainty, this Plate Girder Project was started.

Sponsored jointly by the American Institute of Steel Construction, the U.S. Department of Commerce Bureau of Public Roads, the Pennsylvania Department of Highways, and the Welding Research Council, the research project at Lehigh University was guided by its "Welded Plate Girder Committee" whose members were:
E. L. Erickson, U.S. Bureau of Public Roads, Chairman
A. Amirikian, Bureau of Yards and Docks, U.S. Navy
L. S. Beedle, Lehigh University
Karl de Vries, Bethlehem Steel Company
F. H. Dill, American Bridge Div., U.S. Steel Corp.
Neil van Eenam, U.S. Bureau of Public Roads
E. R. Estes, American Institute of Steel Construction
LaMotte Grover, Air Reduction Sales Company
T. R. Higgins, American Institute of Steel Construction
W. H. Jameson, Bethlehem Steel Company
C. D. Jensen, Pennsylvania Department of Highways
Knut Jensen, Pennsylvania Department of Highways
Bruce G. Johnston, University of Michigan
K. H. Koopman, Welding Research Council, Secretary
George W. Lamb, Consulting Bridge and Struct. Engr. (deceased)
W. B. McLean, Dravo Corporation
N. W. Morgan, U.S. Bureau of Public Roads
W. H. Munse, University of Illinois
E. J. Ruble, Association of American Railroads
J. E. South, Pennsylvania Railroad Company
R. M. Stuchell, Pittsburgh-Des Moines Steel Company
Bruno Thürlimann, Federal Inst. of Technology, Switzerland
George Winter, Cornell University
W. Spraragen, Welding Research Council
ACKNOWLEDGEMENTS

This investigation has been carried out at Fritz Engineering Laboratory of Lehigh University, Bethlehem, Pennsylvania. Wm. J. Eney is Director of the Laboratory and Head of the Civil Engineering Department. The chairman of the Structural Metals Division is Lynn S. Beedle. Thanks are due to both for the support which they have given to this plate girder investigation.

The project is jointly sponsored by the American Institute of Steel Construction, the Pennsylvania Department of Highways, the U.S. Department of Commerce Bureau of Public Roads, and the Welding Research Council. It is supervised by the Welded Plate Girder Project Committee. The financial support of the Sponsors and the continued interest and guidance which the members of the Committee have given to the project is gratefully acknowledged.

Sincere appreciation is expressed to the Engineering and Weldment Department of the Bethlehem Steel Company and in particular to Mr. K. de Vries for supervision and fabrication of the test girders. At Fritz Engineering Laboratory, Ken Harpel and his staff of technicians built
the test rig and gave constant cooperation. Special thanks are due to Üner Taysi and Jin Toh for their assistance in testing, data reduction, and in the preparation of the figures; also to Pete Cooper for proof reading the report.
Table of Contents

WEB BUCKLING TESTS ON WELDED PLATE GIRDERS

Foreword
Acknowledgement
Table of Contents
Nomenclature
Literature Survey and References

Part 1: THE TEST GIRDERS

1.1 Introduction
1.2 Girder Dimensions
1.3 Steel Properties
1.4 Cross Sectional Constants
1.5 Reference Moments and Loads
1.6 Web Buckling Stresses
1.7 Deflections

Part 2: TESTS ON PLATE GIRDERS SUBJECTED TO BENDING

2.1 Introduction
2.2 Design of Girders and Test Setup
2.3 Basic Test Observations
2.4 Ultimate Loads
2.5 Failure Modes
2.6 Discussion
Part 3: TESTS ON PLATE GIRDERS SUBJECTED TO SHEAR

3.1 Introduction
3.2 Design of Girders and Test Setup
3.3 Ultimate Loads and Web Deflections
3.4 SR-4 Strain Gage Measurements
3.5 Additional Strain Measurements
3.6 Discussion

Part 4: TESTS ON PLATE GIRDERS SUBJECTED TO COMBINED BENDING AND SHEAR

4.1 Introduction
4.2 Test Setup
4.3 Test Results
4.4 Discussion
1. **Capital Letters** - preferably used for quantities which do not have linear dimensions

- **A**: Area of cross section
- **E**: Modulus of elasticity, 30,000 ksi
- **G**: Girder, used with a number, for example, G2 refers to girder No. 2; also shear modulus, 11,530 ksi
- **I**: Moment of inertia
- **M**: Bending moment
- **NA**: Neutral axis
- **P**: Applied load
- **Q**: Statical moment of area
- **S**: Section modulus
- **T**: Test, used with a number, for example, T1 refers to the first test on a girder
- **V**: Shear force
- **X, Y, Z**: Cartesian coordinates (in inches) having their origin in the middle of the girder
2. **Small Letters** - preferably used for linear dimensions

- **a**: Panel length
- **b**: Web depth, \( b = 50'' \) for all girders
- **c**: One-half the flange width
- **d**: Flange thickness
- **e**: Distance from NA to the extreme fiber of the flange
- **h**: Distance between the centroids of the flanges
- **k**: Buckling constant
- **l**: Buckling length of a column
- **r**: Radius of gyration
- **t**: Web thickness
- **u, v, w**: Displacements in the X, Y, Z directions
- **x**: Longitudinal coordinate with origins at either end of a girder's span

3. **Greek Letters** - used for nondimensional parameters and stresses

- \( \alpha = a/b \): Aspect ratio, panel length to web depth
- \( \beta = b/t \): Web slenderness ratio, web depth to web thickness
- \( \varepsilon \): Strain
- \( \nu \): Poisson's ratio (\( = 0.3 \))
- \( \sigma \): Normal stress
- \( \tau \): Shear stress
### 4. Subscripts

<table>
<thead>
<tr>
<th>Subscript</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>a : Above</td>
<td>$S_a$ : Section modulus above NA</td>
</tr>
<tr>
<td>b : Below</td>
<td>$e_b$ : Distance from NA to extreme fiber of bottom flange</td>
</tr>
<tr>
<td>cr : Critical</td>
<td>$\sigma_{cr}$ : Critical normal stress</td>
</tr>
<tr>
<td>$\phi$ : Centerline</td>
<td>$v_{\phi}$ : Centerline deflection</td>
</tr>
<tr>
<td>e : End</td>
<td>$I_e$ : Moment of inertia of end sections</td>
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<tr>
<td>f : Flange</td>
<td>$M_f$ : Moment contributed by flanges</td>
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<tr>
<td>i : Ideal</td>
<td>$\tau_{cri}$ : Ideal critical shearing stress before inelastic reductions</td>
</tr>
<tr>
<td>m : Middle</td>
<td>$I_m$ : Moment of inertia of middle or test section</td>
</tr>
<tr>
<td>n : Neutral axis</td>
<td>$I_n$ : Moment of inertia about NA</td>
</tr>
<tr>
<td>p : Plastic</td>
<td>$P_p$ : Load causing the plastic moment</td>
</tr>
<tr>
<td>u : Ultimate</td>
<td>$P_u$ : Ultimate load</td>
</tr>
<tr>
<td>v : Combined</td>
<td>$\sigma_{vcr}$ : Critical stress under combined loading</td>
</tr>
<tr>
<td>w : Web</td>
<td>$A_w$ : Area of the web</td>
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<tr>
<td>y : Yield</td>
<td>$\sigma_{yw}$ : Yield stress of web</td>
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LITERATURE SURVEY AND REFERENCES

An extensive study of the pertinent literature preceded the investigation. This study led to a survey of literature on plate stability of plate girders. Although this study was conducted in 1957, any additional references appearing since then in the technical literature have been inserted at their proper location. Figure 1 graphically summarizes this survey, indicating place, time and nature of the various papers.

This section contains first the abbreviations, as they are found in the English, French, and German literature. Then follows the literature survey, with a translation of the title if in a foreign language. The references are listed alphabetically by authors. Finally, some additional references, Ref. 269 to 276, complete this section. These are papers to which reference is made in this report but do not necessarily belong in the literature survey which is concerned with plate stability only.
<table>
<thead>
<tr>
<th>Publications by:</th>
<th>Type of Their Publications:</th>
<th>Journals</th>
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<tbody>
<tr>
<td>Abbreviation</td>
<td>Abbreviation</td>
<td>Meaning of Abbreviation</td>
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<tr>
<td>I.V.B.H.</td>
<td>Internationale Vereinigung für Brückenbau und Hochbau</td>
<td>Vorb.</td>
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<td></td>
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<td>Schl.Ber.</td>
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<td>Abh.</td>
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<td>A.I.P.C.</td>
<td>Association Internationale des Ponts et Charpentes</td>
<td>Publ.prév.</td>
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<td>Rap. f. Mem.</td>
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<tr>
<td>N.A.C.A.</td>
<td>National Advisory Committee on Aeronautics</td>
<td>T.N.</td>
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<td>T.M.</td>
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<td>W.R.</td>
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<td>A.S.C.E.</td>
<td>American Society of Civil Engineers</td>
<td>Proc.</td>
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<td>sta t i k an der Eidi-</td>
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<td>genössischen Techni-</td>
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<td>schen Hochschule Zürich</td>
<td>schen Hochschule Zürich</td>
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<td>Buing.</td>
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<td>Stahlbau</td>
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<td>Ing. Archiv</td>
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<td>Weld. &amp; Met. Fabrn.</td>
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<tr>
<td>Luftf. Forsch.</td>
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<tr>
<td>Z.Flugtechn.u.Motorluftsch.</td>
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</tbody>
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<table>
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<th>Country</th>
<th>Buckling of Rectangular Plates</th>
<th>Bending, Compression, or Shear</th>
<th>Containing Test Results</th>
<th>Inelastic Buckling</th>
<th>Longitudinal Stiffeners</th>
<th>Transverse Stiffeners</th>
<th>Post Buckling Behavior</th>
<th>Books on Plate Buckling</th>
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<td>1910</td>
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<td>1940</td>
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<tr>
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<td>1849</td>
<td>1890</td>
<td>1894</td>
<td></td>
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</table>

Fig 1

Graphical Summary of References
1.1 Introduction

The purpose of part 1 is to describe the test girders and the physical properties of materials used. Then, based on this information, standard reference values such as "yield loads", "plastic loads", "critical loads", and computed deflections will be established. The organization of the test program must first be described.

A girder section can be subjected to bending, shear, or a combination of both these loadings. In this research project all three conditions were investigated. Consequently, three different test setups were used as shown in Figs. 1.1, 1.2, and 1.3. This classifies the thirteen plate girders into the following three groups:

<table>
<thead>
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<th>Group</th>
<th>shown in</th>
<th>subjected to</th>
<th>Girders</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fig. 1.1</td>
<td>bending</td>
<td>G1, G2, G3, G4, G5</td>
</tr>
<tr>
<td>2</td>
<td>Fig. 1.2</td>
<td>shear</td>
<td>G6, G7</td>
</tr>
<tr>
<td>3</td>
<td>Fig. 1.3</td>
<td>combined</td>
<td>E1, E2, E4, E5, G8, G9</td>
</tr>
</tbody>
</table>

Throughout the entire project the girders are termed as given in the last column of this table.
The cross section of some girders changed within their lengths. The reason for this design was to confine failure to a certain region whose loading conditions were well defined. This region was the test section proper as indicated in Figs.1.1 and 1.2. The end sections, flanking the test section, only differed in web thickness for the first group, whereas in the second group cover plates were also added over a portion of their length. In the third group the cross section did not change and the entire girder was the test section proper. The four girders termed E1, E2, E4, and E5 were fabricated by splicing the undamaged end sections of the correspondingly numbered girders G1, G2, G4 and G5, and reinforcing them with cover plates.

Each girder was subjected to at least two ultimate load tests. After causing failure in a particular panel, the load was removed and the panel reinforced. All bending girder failures occurred in the compression flange and thus reinforcement consisted of welding small steel plates to this flange. Diagonal and transverse stiffeners were used to strengthen the girders of the other two groups. Since no major deformations were caused in panels adjacent to the one which failed in the first test, referred to as T1, a second test, T2, could be conducted. In some cases this process was repeated and additional tests, such as T3 and T4, were carried out.
Of all the possible parameters influencing the carrying capacity of plate girders, the investigation was restricted to the following four:

1. Loading condition: \( \zeta = \frac{\tau}{\sigma} = \frac{\text{shear stress}}{\text{normal stress}} \)

2. Type of cross section: various shapes of compr. flanges

3. Web slenderness: \( \beta = \frac{b}{t} = \frac{\text{web depth}}{\text{web thickness}} \)

4. Stiffener spacing: \( a = \frac{a}{b} = \frac{\text{panel length}}{\text{web depth}} \)

The first parameter was taken care of by choosing the three test setups previously mentioned. The second parameter was of special importance to the bending girders, wherefore three different shapes of the compression flanges were chosen as illustrated with cross sections I, II, and III shown in Fig. 1.4. Denoting with "a" the stiffener spacing, "b" the web depth, and "t" the web thickness, the third and fourth parameters completely defined the shape of a web panel. In the test program, the third parameter was varied by building pairs of girders which only differed in the web slenderness, such as G2 and G4, or G3 and G5. Finally, the fourth parameter was accounted for by subdividing the test section into panels with different lengths. After failure occurred in a longer panel, it was reinforced and thus failure could be forced to occur in a shorter panel.
The parametric values of all the girders' test sections are listed in Table 1.1. The added sketches indicate where each girder failed, how the obtained tests were designated, and where reinforcement plates were welded to the flanges or webs. Taking as an example girder E₄, the sketches give the following information: The first test of this girder, T₁, caused failure in the left-hand end panel whose stiffener spacing was 1.5 times the web depth. The girder was then reinforced by subdividing each of the two larger panels with new transverse stiffeners. Thus failure was forced to occur in the right-half of the girder where the spacing of the stiffeners was 0.75 of the depth. This happened in the panel adjacent to the loading stiffener and furnished the second test, T₂. Welding a reinforcing stiffener across this damaged panel allowed for a third test T₃ in a panel whose aspect ratio was α = 0.5.

In the following sections the evaluation of girder plate dimensions is given first, followed by the determination of steel properties, and finally, the detailed computation of specific reference values.
1.2 Girder Dimensions

After the general survey of the girders given in Sec. 1.1, it is the purpose of this section to establish the accurate dimensions of each girder. The overall dimensions, as ordered, are given in Figs. 1.1, 1.2, and 1.3; for all practical purposes they can also be considered to be the actual ones. However, this situation can not be expected to apply to the size of the component plates and measurements must be taken to determine their true dimensions.

In illustrating the procedure used to obtain the dimensions and the differences between ordered and actual dimensions, the test section of girder Gl is used. The top flange, web, and bottom flange of this girder are shown in Fig. 1.5. Here, it is seen that at the ends of each plate, a piece was cut off and used for coupons. These end pieces were also used to obtain the width and thickness of the corresponding plates. The dots in these portions are points at which thicknesses were measured and the results are recorded beside them. The averages of all the measurements recorded were considered to be the true dimensions. Measured at 20 different locations, the web presented an interesting finding. As readings were taken from the upper edge to the lower, the thickness was found to increase, exceeding the ordered quarter inch thickness anywhere from 1 to 10%. 
This variation is due to the fact that the web was originally cut from a plate whose width was 100 inches and the lower edge of the web was located at the midspan of the rolls during the rolling operation. The slight flexibility of the rolls gave this increase of two hundredths of an inch.

Using the same procedure and layout of observation points, the other girders' dimensions were determined and are presented in Table 1.2. All subsequent computations are based on these values and other data given in Figs. 1.1, 1.2 and 1.3.
1.3 Steel Properties

A great amount of time and effort was spent in evaluating the properties of the girders' component steel plates. Because the property of paramount importance to this research program was the yield stress, the major part of this section is devoted to its definition and determination. It will be seen that, although mild steel was specified for all girder components and care was taken to obtain a uniform yield level, a considerable scatter of results is unavoidable.

Tests on tension coupons made from the material under consideration were conducted to determine the yield level. At least one coupon was cut from each flange plate, unless two or more flanges came from the same slab. In this case, a single coupon was considered sufficient for the entire slab. This same principle applied to the web plates and, in addition, a limited number of coupons were cut transversely to the plate's longitudinal direction. The relative location of both flange and web coupons in their respective plates is shown in Fig. 1.5.

In Table 1.3 all coupons tested for the plates comprising girders G1 through G7 are listed. The additional coupons, needed to complete the yield stress evaluation in the group of girders under combined bending and shear, are recorded in Table 1.4. The first columns of these tables
describe the location of the plates from which the corresponding coupons were cut. Although the exact dimensions of these plates already appear in Table 1.2, for the convenience, the nominal plate thicknesses are tabulated again. Each coupon is assigned a number as shown in the third column. Besides its number, a coupon is designated further by listing its steel quality according to ASTM specifications and its heat and slab numbers provided by the steel manufacturer. It may be observed that, if two or more coupons have the same slab number, they originate from one and the same rolled piece. A common heat number indicates that the steel of these coupons were taken from the same furnace charge, therefore they must have the same chemical composition.

Further listed in Tables 1.3 and 1.4 is the chemical analysis procured from the mill, showing the carbon, manganese, phosphorus, and sulfur content of the steel. In the following columns of the tables are the yield stresses, ultimate stresses and elongations, all determined by the mill according to standard practice. Finally, in the last five columns are tabulated the results of the coupon tests conducted at Fritz Engineering Laboratory. The three characteristic values of yield stress, ultimate stress and elongation are listed here, together with the area reduction at the fractured section and the rupture stress \( \sigma_r \) occurring over the reduced area. To compare the laboratory results
with those obtained by the mill, the former results need further explanation.

Each coupon was machined to the dimensions given in Fig. 1.6. These coupons conform with ASTM requirements for plates over three-sixteenths of an inch in thickness. (For plates below this value, specifications call for a smaller coupon with a two inch gage length rather than the eight inch length used.) Figure 1.6 is a typical data sheet illustrating in detail the evaluation of pertinent data of the coupon. Figure 1.7 is the corresponding load-strain curve for this coupon. An extensometer was used to obtain the strain for this figure and an electronic recorder automatically plotted the resulting curve. The abscissa is the average strain in inches per inch gage length, while the ordinate is the tension force applied to the coupon. By converting load to stress, this diagram could be considered as a stress-strain curve extending to about thirty times the yield strain, only about one-eighth of the complete diagram up to the rupture point. Characteristic of this diagram would be the straight-lined elastic part, the yield level and the inception of strain hardening. Also included in this graph are: the upper yield point; immediately following the lower yield point; the dynamic yield level about which the load fluctuates during yielding; and, finally, the static yield level which shall be discussed further.
The static yield level is the yield stress obtained under a zero strain rate. This strain rate could easily be imposed by the 120,000 pound Tinius Olsen machine used; a screw-powered type machine which allowed complete control of the speed of the movable crosshead. When pronounced yielding was apparent, the movement of this crosshead was stopped after which the load settled to the static yield level, Fig. 1.7. After about five minutes, the speed of the crosshead was again set at its former value of 0.10 inches per minute. As seen, the resulting dynamic yield level coincided with its previous value. This procedure was repeated a second time in the yield zone where another typical V-notch in the recorded load-strain curve occurred.

It is generally known that the yield stress level does depend on the speed used to test a coupon and increases with higher testing speeds. However, the significant research work carried out in Fritz Engineering Laboratory, Ref. 274, correlates the dynamic yield levels obtained at various strain rates and points out that this static yield level is a material constant which can be obtained more accurately than the fluctuating dynamic yield level. Since it is impossible to maintain any constant strain rate on a steel element such as a plate girder, it is obvious that the static yield level must be adopted as the significant level in the testing of structural members. Only with zero strain rate can complete correlation between both structure
and coupon be attained. Therefore, the yield stresses $\sigma_y$ henceforth mentioned in these reports will always be the static yield stresses. As a consequence, the ultimate load must be defined as the highest load which the structure can statically maintain.

The results of these carefully conducted coupon tests can certainly be used as added data to be collated for statistical purposes. Of the many conclusions which may be drawn from the two summary tables, only the following are mentioned. From Table 1.3 the seven three-quarter inch coupons show the scatter of yield level which must be expected when plates of equal dimensions are rolled, even though they originated from the same ingot. The one-half inch and three-eighth inch web material also came from a common ingot but differed in thickness and, thus, in the extent of rolling. This resulted in a marked difference in the static yield levels. Furthermore, the static yield levels of the three-quarter inch plates were about 10% lower than the yield stresses determined by the mill. However, this percentage changes considerably with the plate thickness and the chemical composition. For plate thicknesses greater than three-quarter of an inch, this reduction may well be as much as 25%, as seen from Table 1.4.

Contrary to the aforementioned, it was found that for the coupons cut from the one-eighth inch plates, CP 23 and
CP 47, the relation was reversed. Static yield levels as much as 15% higher than the dynamic ones furnished by the mill were observed. This was then believed to be a mistake and additional coupons adjacent to the previous ones were cut from the plate specimen and tested by both the fabricator and the investigator. It can be seen from Table 1.3 that the results of these duplicate coupons, termed CP 23B and CP 47B, were just about the same as previously obtained. As stated before, the gage length of the mill and laboratory coupons were two and eight inches respectively; the mill coupons conformed to ASTM standards. It is interesting to speculate as to whether the size causes such effects.

In summary, it must be emphasized again that the important material property called "yield stress", as determined by standard practice in the mills, is not adequate for strength predictions in research work where structures are subjected to static loads.

For the tubular compression flanges of girders G3 and G5, a compression test was conducted on a short section of the pipe rather than tension coupon test. Reference should be made to Fig. 1.8 where the size of this stub column, the load-deformation record, and the computed stress-strain diagram are all shown. Also the wide scattering of wall thickness in the tested pipe can be seen. The yield stress was evaluated from the evident yield level.
Finally, the yield stresses of all component plates are summarized in Table 1.5, grouped according to the respective girders. The computation of all girder reference values is based on the data tabulated here.
1.4 Cross Sectional Constants

In this section will be presented the moments of inertia for all girders with their corresponding section moduli. To compute these values, it is necessary to know the cross sectional shapes and dimensions. The former can be found in Fig. 1.4, while the latter are summarized in Table 1.2.

A typical computation of the cross sectional constants is carried out below. The procedure was first to find the moment of inertia $I_z$ about the Z-axis which was located at the mid-depth of the web. Then, after determining the actual centroid of the section, the moment of inertia about the neutral axis was found by means of the parallel axis theorem. Finally, dividing this value by the distance to the extreme fibers $e_a$ and $e_b$, the section moduli $S_a$ and $S_b$ were obtained. The indices "a" and "b" distinguish between quantities above and below the neutral axis respectively.
Computation of Section Moduli of Gl-T1, Test Section

\[
\Delta Y = \frac{Q_z}{A} = \frac{-15.0}{31.59} = -0.47 \text{ in}
\]

\[
I_m = I_z - (\Delta Y)^2 A = 14,390 - 0.47^2 \times 31.59 = 14,380 \text{ in}^4
\]

\[
e_a = 25 + 0.47 + 0.43 = 25.90 \text{ in}
\]

\[
e_b = 25 - 0.47 + 0.76 = 25.29 \text{ in}
\]

\[
S_a = \frac{I_n}{e_a} = \frac{14,380}{25.90} = 555 \text{ in}^3
\]

\[
S_b = \frac{I_n}{e_b} = \frac{14,380}{25.29} = 568 \text{ in}^3
\]

Following the above procedure, all necessary cross sectional constants were computed for the girders and are presented in Table 1.6. In the first three columns of this table are given the properties of the test section, namely, the moment of inertia \( I_m \) and the corresponding section moduli \( S_a \) and \( S_b \). Next, the moments of inertia of the bending and shear girders' end sections, \( I_e \), are added. Finally, some special moments of inertia are given in the last column which will now be explained for each girder:
- G1. After the first test on this girder was completed, its top flange width was reduced by flame cutting to 13.56 inches. Thus, for computations involving G1-T2 (second test of girder G1) this new width must be used, resulting in \( I = 12,210 \text{ in}^4 \) and a neutral axis at \( y = -3.158 \text{ in} \).

- G2, G3, G4, G5. After completion of the first tests of these bending girders, a steel plate was welded on each side of the top flange. Each of these two plates had an area of one square inch, had the same distance from the neutral axis as the centroid of the unreinforced flange, and extended over the longest panel of 75 inches. Wherefrom this new I-value is computed.

- G6, G7. These values are the moments of inertia of the sections under the reactions where cover plates were added, that is, E D shown in Fig. 1.2.

- El. The outside cover plates of girder El were terminated 75 inches from its ends and, therefore, two moments of inertia are needed to compute deflections. The value shown in the last column applies to the end portions of the girder. Since the third test produced failure within this region, the values without cover plates must be used for calculations concerning El-T3.
1.5 Reference Moments and Loads

This section is devoted to the computation of the flange moment, yield moment, and plastic moment with the corresponding loads of the latter two. These moments are defined in Ref. 7, p. 34. Their definitions are repeated below with the modifications needed to take into account the different yield stresses of the component plates.

The flange moment, $M_f$, is defined as the moment carried by the flanges alone when the stresses over the flanges are equal to the yield stress. For a symmetrical girder whose yield stresses are the same for both flanges, this would simply be computed as $M_f = A_f \sigma_{yf} h$, where $A_f$ and $\sigma_{yf}$ are the area and yield stress of one flange and $h$ is the distance between the centroids of the flanges. The actual girders tested exhibited a certain degree of dissymmetry in shape and yield stress. Therefore, the area and yield stress of the compression flange are selected to be used for the computation. Incidentally, the alternate use of the tension flange properties would not lead to any great differences. Computations show that their use could only give a value lower by 2.5%. When more than one plate comprises a flange, a weighted yield stress of the compression flange was used. This weighted stress, $\bar{\sigma}_y$, will be defined as $\bar{\sigma}_y = \sum A \sigma_y / \sum A$ where $A$ and $\sigma_y$ are the areas and yield stresses of the component plates and $\sum$ indicates their summation.
The yield moment, $M_y$, is the moment which initiates nominal yielding in the most extreme fiber. In the case where the yield stresses of the flanges would be the same, it would be computed as $M_y = \sigma_y S$, where the smaller value of the section modulus, $S$, would be used. Since the yield stresses of the flanges differed, the definition that $M_y = \sigma_y a S_a$ is adopted, where $\sigma_y$ and $S_a$ are the yield stress and section modulus of the compression flange. As in the case of the flange moment, the value of the yield moment, when computed using bottom flange properties, could be lower than the defined value by only 2.5%. In accordance with the procedure adopted previously, a weighted yield stress was used when the flange was composed of a number of plates.

The plastic moment, $M_p$, is the limiting value of the moment which would be reached upon applying an infinite curvature to a section, neglecting the effect of strain-hardening. Usually it is calculated as the product of the girder's yield stress and plastic modulus, $Z$. This method assumes a section whose yield stress is constant for all its elements. As such, it can not be used in computations involving the test girders since most of their component parts yielded at different stress levels. This moment will be evaluated from the relation that $M_p = \Sigma A \sigma_y y_p$, where the $A$ and $\sigma_y$ are the area and yield stress of a section's elements and $y_p$ is the distance from the plastic neutral axis, $N_{A_p}$, to the centroid of each element.
COMPUTATION OF PLASTIC MOMENT OF E2

\[ \sum [A \sigma_y]_a - \sum [A \sigma_y]_b = 0 \]

\[
16.11 \times 29.4 + 9.37 \times 38.6 + \\
(0.507y_a)34.9 - 0.507(50 - y_a)34.9 - \\
9.44 \times 37.6 - 16.11 \times 29.4 = 0
\]

\[
y_a = 24.8 \text{ in.}
\]

\[ M_p = \sum (A \sigma_y) y_p \\ = 474 \times 26.07 + 362 \times 25.19 + 439 \times 12.4 + \\
446 \times 12.6 + 355 \times 25.59 + 474 \times 26.48
\]

\[ M_p = 54,100 \text{ k-in} \]

This plastic neutral axis is found from the equilibrium condition that the sum of the normal forces over the entire cross section must vanish. Using the subscripts a and b mentioned before, this condition is expressed as

\[ [\sum A \sigma_y]_a - [\sum A \sigma_y]_b = 0. \]

As a sample computation, the plastic moment of E2 has been calculated above. All necessary static yield stresses are listed in Table 1.5.

To calculate the yield and plastic loads, the spans of the girders enter. Again, due to the different test setups, three groups are distinguished: bending, shear, and combined loading.
The bending group has a constant moment over the test section, \( M = 150P \). Thus the yield and plastic loads are simply computed as \( P_y = \frac{My}{150} \) and \( P_p = \frac{Mp}{150} \), where the moments are expressed in kip-inches and the loads in kips.

The shear group, although subjected to a relatively small variable moment, was considered to be under pure shear. Therefore, the yield load is the load that initiates nominal yielding at the neutral axis in the web and is computed from \( V_y = \frac{\tau_{yw}lt}{Q} \) where \( V_y \) is the shear force at first nominal yielding, \( \tau_{yw} \) the shear yield stress of the web, \( Q \) and \( I \) are the static moment and moment of inertia about the neutral axis, and \( t \) is the thickness of the web. From Fig.12 it can be seen that \( V = P \). Substituting this value in the preceding equation and using the Mises yield condition that \( \sigma_{yw} = \sqrt{3} \tau_{yw} \), the yield load will be evaluated as \( P_y = \frac{\sigma_{yw}lt}{\sqrt{3} Q} \) where \( \sigma_{yw} \) is the yield stress of the web. The plastic load is defined as the load which causes the web to yield completely due to shear, \( P_p = \frac{\sigma_{yw}Aw}{\sqrt{3}} \), \( A_w \) being the area of the web, \( A_w = bt \).

The combined group was subjected to both shear and moment. Since the moment varied throughout the girder's length, a cross section in the failed panel was selected at which the reference loads for each test were to be evaluated. This section was chosen to be at a distance of one-half the web depth away from the maximum moment in the panel or at the middle of the panel when its length is less
than its depth. It is realized that this method of evaluating the yield and plastic loads differs from the usual procedures used for a beam. Thus, the yield load is defined as the load which initiates yielding in the critical cross section of a girder. In general, yielding first occurs at the intersection of the web and flange where the yield condition, \( \sigma_{yw} = \sqrt{\sigma^2 + 3\tau^2} \), is used to evaluate the yield load. Substituting the values of \( \sigma = \frac{MV}{I} = \frac{Px}{2I} \) and \( \tau = \frac{VQ}{2It} \) into the equation above, where \( M = \frac{Px}{2} \) and \( V = \frac{P}{2} \) from Fig. 1.3, the yield load for the girders under combined loading will be

\[
P_y = \frac{\sigma_{yw}}{\sqrt{(25x/2I)^2 + 3(q/2It)^2}},
\]

\( x \) being the distance from the end of the girder span to the critical cross section. If yielding does not begin at the aforementioned point, the bending or shear case discussed before applies.

The plastic load of any single test on a girder is defined as the load producing plastification at the critical cross section of the failed panel. The presence of shear in the combined bending and shear group of girders reduced their full plastic moments \( M_p \). For these girders, an expression for a modified plastic moment \( M_{ps} \) was developed from the following considerations. The stress condition sketched on next page is the basis for evaluating \( M_{ps} \) (Ref. 272). It will be assumed that the flanges have fully yielded,
thereby providing the flange moment, \( M_f \), and that a constant normal stress \( \sigma \) is present over the web accompanied by the constant shearing stress \( \tau \). From the sketch, the modified moment is: \( M_{ps} = M_f + \frac{\sigma tb \cdot b}{2} \). An expression for \( \sigma \) is obtained from the yield criterion \( \sigma_{yw} = \sqrt{\sigma^2 + 3\tau^2} \), where \( \tau = \frac{P}{bt} = \frac{M_{ps}}{bt\tau} \). Substituting the value of \( \sigma \) in the first equation gives \( M_{ps} = M_f + \frac{1}{4} tb^2 \sqrt{\sigma_{yw}^2 - 3(M_{ps}/bt\tau)^2} \).

After solving for \( M_{ps} \) and observing that \( M_{ps} = \frac{P}{2} \),

\[
P_p = \frac{2}{xa} \left[ M_f + \sqrt{am^2_w - (a-1)M^2_f} \right]
\]

where the constant \( a = 1 + \frac{3}{16\left|\frac{b}{x}\right|^2} \), and \( M_w \) is the portion of the full plastic moment \( M_p \) contributed by the web, \( M_w = \sigma_{yw}tb^2/4 \). When a negative number results under the radical sign, the shear case explained before must be used to obtain \( P_p \). Physically this result implies that the web yields due to shear before the yield stress is reached in the flanges.

In Table 1.7 are summarized the reference moments and loads for all test girders. Unlike the bending and shear groups, which had constant moments and shears over their test sections, the combined group had a variable moment over its test sections which results in two or more yield and plastic loads for each girder.
1.6 Web Buckling Stresses

An additional reference value with which the obtained ultimate load can be compared is the conventionally computed web buckling stress or load. It is the objective of this section to establish these stresses and loads for all the girders.

The general equation for the ideal critical stress of an isolated web panel is

\[
\sigma_{\text{cri}} = k \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2 = k \frac{\pi^2 E}{12(1-\nu^2)} \cdot \beta^2
\]

where \(\sigma_{\text{cri}}\) and \(\tau_{\text{cri}}\) are the ideal critical normal and shearing stresses, respectively. The factor \(\frac{\pi^2 E}{12(1-\nu^2)}\) is a constant dependent only on the material properties, that is, the modulus of elasticity \(E\) and Poisson's ratio \(\nu\), while \(\beta = b/t\) is the slenderness ratio of the web. Finally, the buckling coefficient \(k\) is a variable depending on the loading and boundary conditions and, in general, also on the panel's aspect ratio \(\alpha = a/b\). Values for this coefficient can be found in such literature as Ref. 73, 116, and 247.

Some detail information must be specified in order that the web buckling stresses of the actual girders can be computed. In general, the procedures of Ref. 21 and 52 have been adopted for the details which follow:
- The constant, \( \frac{\pi^2E}{12(1-\nu^2)} \), for steel plates is equal to 26,750 ksi.

- Web panels are considered pin ended on all sides.

- The proportional limit of the web material is taken as 0.8\( \sigma_y \). If the ideal critical stress \( \sigma_{cri} \) is less than this value, it is equal to the critical stress \( \sigma_{cr} \),

\[ \sigma_{cri} = \sigma_{cr} \]. Whenever it exceeds this value, the critical stress \( \sigma_{cr} \) is found from a reduction procedure,

\[ \sigma_{cr} = \sigma_y \left( 1 - \frac{0.16\sigma_y}{\sigma_{cri}} \right) \], a relation derived from Eq. (64), Ref. 21. Similarly \( \tau_{cr} = \tau_y \left( 1 - \frac{0.16\tau_y}{\tau_{cri}} \right) \), where \( \tau \) is the shear stress.

- When a moment gradient exists in a panel, the critical section is considered to be at a distance of one-half the web depth from the maximum moment in the panel. In the case where the panel's length is smaller than its depth, this section is at the middle of the panel.

- The critical shear force of a panel subjected to pure shear is computed as the product of the critical shear stress and the area of the web, \( V_{cr} = \tau_{cr} A_w \).

- In all cases, the unsupported web depth "b" is taken as the clear web depth, which is 50 inches for all girders.

- Finally, when the neutral axis is 1/2 inch or less away from the web's geometric center, it is assumed to coincide with the centerline.

The general cases of bending, shear, and combined loading are presented next.
For bending, the general formula for the normal critical stress $\sigma_{cri}$ applies. The k-value is the only remaining unknown. Since all the girders except G1-T2 had their neutral axes less than 1/2 inch away from the web's geometric center, the k-value is $k = 23.9$. In G1-T2 the neutral axis shifted 3.62 inches down from the web's centerline and thus the compressive stresses are higher than the tensile stresses. In accordance with Ref. 52, this leads to a k-value of $k = 18.6$. The critical load $P_{cr}$ is determined from the relation that $\sigma_{cr} = M_{cr}Y/I$, where $M_{cr} = P_{cr}x/2$, see Fig. 1.1.

For shear, the general formula for the critical shearing stress $\tau_{cri}$ applies. The buckling coefficient of $k = 4.00 + \frac{5.34}{a^2}$ is used when the panel's aspect ratio $a$ is equal to or less than unity, and $k = 5.34 + \frac{4.00}{a^2}$ is used when $a > 1$. In this case, $P_{cr}$ is evaluated from $\tau_{cr} = V_{cr}/bt$, where $V_{cr} = P_{cr}$, see Fig. 1.2.

For combined bending and shear,

$$\sigma_{vcri} = \frac{\sqrt{\sigma^2 + 3\tau^2}}{\sqrt{(\sigma/\sigma_{cri})^2 + (\tau/\tau_{cri})^2}},$$

where $\sigma_{vcri}$ is the equivalent ideal critical stress for combined loading according to Ref. 52, $\sigma$ the extreme bending stress of the web, $\sigma = MY/I$, and $\tau$ the average shearing stress $\tau = V/bt$. $M$ and $V$ are the applied moment and shear respectively in the considered cross section. The formula above is only applicable to girders whose neutral axes coincide with their web's geometric centers and as such applies to all the
girders of the combined group whose axes were all less than 1/2" away from the centerline of the web. After reducing the ideal stress for inelastic action, if necessary, the critical load $P_{cr}$ is obtained from the equation $\sigma_{vcr} = \sqrt{\sigma^2 + 3\tau^2}$, where $\sigma$ and $\tau$ are both functions of $P$, the applied load.

As an example, the critical load of the first test of E2, a girder under combined loading, shall be computed. From the tables and figures of previous sections the properties of E2-T1 are as follows: $a = 3.0$, $\beta = 99$, $A_w = 25.3$ in$^2$, $I = 39,620$ in$^4$ and $\sigma_{yw} = 34.9$ ksi. From Table 1.1 it is seen that the long panel failed in this first test. Since a moment gradient exists in this panel, the critical cross section is at a distance of 25" to the left of the center bearing stiffener or $x = 125$" from the end of the girder. Knowing this data and using $k_\sigma = 23.9$ and $k_{\tau} = 5.79$ ($a>1$), the critical stresses due to moment and shear are evaluated as $\sigma_{cr1} = 65.2$ ksi and $\tau_{cr1} = 15.8$ ksi. The stress at the compressive edge of the web is $\sigma = \frac{MV}{I} = \frac{P \cdot 125}{2.39,620} \cdot 25 = 0.0394P$ [ksi], and the average shearing stress over the section is $\tau = \frac{V}{A_w} = \frac{P}{2.25.3} = 0.0198P$ [ksi]. Substituting these values into the equation for the combined critical stress,

$$\sigma_{vcr1} = \sqrt{\left(0.0394P\right)^2 + 3\left(0.0198P\right)^2} = \frac{0.0522P}{0.00139P} = 37.5 \text{ ksi}$$

The proportional limit is $0.8\sigma_y = 0.8 \times 34.9 = 27.9$ ksi.
Since \( \sigma_{cr1} > 0.8 \sigma_y \), the reduction for the inelastic range applies as follows:

\[
\sigma_{vcr} = 34.9 \left( 1 - 0.16 \frac{34.9}{37.5} \right) = 29.8 \text{ ksi}
\]

Since \( \sigma_{vcr} = \sqrt{\sigma^2 + 3\tau^2} = 0.0522 P_{cr} \), \( P_{cr} = 29.8 / 0.0522 \), or

\[
P_{cr} = 570 \text{ kips}
\]

In Table 18 and 19, the critical stresses and loads for all girders are summarized. The bending and shear groups never exceed the elastic limit and thus \( \sigma_{cri} = \sigma_{cr} \) and \( \tau_{cri} = \tau_{cr} \). The center panel of E5-T1 was subjected to pure moment and therefore no entries are made under the columns for shear.
1.7 Deflections

In order to check on the elastic behavior of the girders, their predicted deflections are evaluated in this section. Again three groups exist: bending, shear, and combined loading girders. The centerline deflections are obtained for the bending and combined groups while the end deflections are given for the shear girders.

The method of Virtual Work is used to obtain all deflections. In this method a unit load is applied to the girder at the point where the deflection is desired and its resulting moment, \( m \), and shear, \( v \), diagrams drawn. Then the deflection directly under this "dummy" load is computed as the sum of the bending and shear contributions:

\[
v = \int \frac{Mm}{EI} \, dx + \int \frac{Vv}{GAw} \, dx
\]

In this expression \( M \) and \( V \) are the moment and shear due to the actual loading, \( E = 30,000 \text{ ksi} \) is the modulus of elasticity, and \( G = 11,530 \text{ ksi} \) is the shearing modulus. All integrals extend over the entire girder length where the origin of the \( x \)-axis is taken at the end of each girder.
The units for all quantities and dimensions are kips and inches.

To illustrate the procedure followed in calculating deflections, the expression for the centerline deflections of the bending girders is developed now. In the example shown on the next page, the loading is first pictured, together with sketches of the areas and moments of inertia of the girders. Then the moment and shear diagrams, both for the real and dummy loadings, are drawn. Below these diagrams, the integrals are written, the first three representing the moment component and the last one including the shear contribution. Observing the symmetry of the loading and cross sectional properties, the integrals need only be evaluated over half the length of the girder and then doubled to obtain the final value.

Substituting the properties of GI-T1 into the resulting equation (a), where \( I_e, I_m, \) and \( A_e \) equal 15,550 in\(^4\), 14,380 in\(^4\), and 19.10 in\(^2\) respectively, the centerline deflection for the applied load of \( P = 100 \) kips would be 1.172 inches. In this case the shear component is 5.8% of the total deflection.

Using the same procedure as above, the equations needed to evaluate all required deflections are obtained. These expressions are listed on page 31 together with the cases to which they apply.
Centerline Deflection of Bending Girders

\[ v_x = \int \frac{M_m}{EI} \, dx + \int \frac{V_v}{GAW} \, dx \]

\[ = 2 \left[ \int_0^{150} \frac{150P(x/2)}{EI_e} \, dx + \int_0^{183} \frac{(150P)(x/2)}{EI_e} \, dx \right] \]

\[ + \int_0^{150} \frac{270P(x/2)}{EI_m} \, dx + \int_0^{150} \frac{P(1/2)}{A_WG} \]

\[ = 2 \times 10^3 \left[ \frac{562.5P}{EI_e} + \frac{412.1P}{EI_e} + \frac{1478P}{EI_m} + \frac{0.075OP}{A_WG} \right] \]

\[ v_x = P \left[ \frac{64.97}{I_e} + \frac{98.53}{I_m} + \frac{0.01301}{A_e} \right] \]  \hspace{1cm} (a)
All bending girders, (except G1), were reinforced with steel plates after their first test. With a new cross section present, whose moment of inertia $I$ is listed in the "special sections" of Table 1.6 the expression for the centerline deflection in the second test is:

$$v_{\epsilon} = P(64.97/I_e + 54.93/I_m + 43.59/I + 0.01301/A_e) \quad (b)$$

The shear girders have a maximum deflection at their ends. Observing that these girders have cover plates at their reaction points, thus having special moments of inertia $I$, the equation for end deflections is:

$$v_e = P(5.689/I_e + 7.316/I_m + 25.39/I + 0.0132/A_e + 0.0075/A_m) \quad (c)$$

Girder E1, the first of the girders under combined loading, had its second cover plates terminated 75 inches from its ends. Letting the moments of inertia of the section with and without the second cover plates be $I_m$ and $I$, the relation for centerline deflection is:

$$v_{\epsilon} = P(19.88/I_m + 2.34/I + 0.00689/A_w) \quad (d)$$

All other girders under combined loading had constant cross sections throughout their lengths. As such, the centerline deflections can be found from Eq. (d) by substituting $I = I_m$. The resulting equation is:

$$v_{\epsilon} = P(22.23/I_m + 0.00689/A_w) \quad (e)$$

A summary of girder deflections, computed from the above equations, is given in Table 1.10. Here, the bending and shear components of the total deflection are listed, together with the equation that is applied to determine
them. The centerline deflection is given for the bending and combined groups while the end deflection is listed for the shear girders. As a matter of interest, the percentage of the shear contribution to the total deflection is included. All deflections are evaluated for $P = 100$ kips.
<table>
<thead>
<tr>
<th>Girder</th>
<th>Loading</th>
<th>Cross Section</th>
<th>Web Slenderness</th>
<th>Stiffener Spacing: a</th>
<th>Location of Failure and Reinforcements</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>G5</td>
<td>G6</td>
</tr>
<tr>
<td>I</td>
<td>II</td>
<td>III</td>
<td>II</td>
<td>III</td>
<td>II</td>
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Table 1.2

Summary of Cross Sectional Dimensions in inches

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<tr>
<th>Girder</th>
<th>Cross Sect.</th>
<th>Top Width 2c</th>
<th>Flange Thickness d</th>
<th>Bottom Width 2c</th>
<th>Flange Thickness d</th>
<th>Web Test t</th>
<th>End t</th>
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<tbody>
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<td>0.270</td>
<td>0.382</td>
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<tr>
<td>G2</td>
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<td>12.19</td>
<td>0.769</td>
<td>12.19</td>
<td>0.774</td>
<td>0.270</td>
<td>0.507</td>
</tr>
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<td>12.19</td>
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Cover Plates

\[
\begin{align*}
\text{PL A: } & 15.04 \times 0.882, \\
\text{PL B: } & 18.00 \times 0.750, \\
\text{PL C: } & 16.00 \times 1.007, \\
\text{PL D: } & 11.19 \times 0.510.
\end{align*}
\]
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<th>Thickness</th>
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<th>Specification</th>
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<th>Slab No.</th>
<th>C</th>
<th>Mn</th>
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<th>S</th>
<th>Fy</th>
<th>Fu</th>
<th>Elong</th>
<th>Fy</th>
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Note: A refers to coupon taken in transverse direction.
B refers to additional coupon taken next to the original one, e.g., CP 23 and CP 2B
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**Note:** A refers to coupon taken in transverse direction. B refers to additional coupon taken next to the original one, e.g., CP E2 and CP EB.
Table 1.5

Summary of Static Yield Stresses
(kips per square inch, ksi)

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Cover Plates

| PL A: 30.1, PL B: 29.8, PL C: 29.4, PL D: 33.5 |
Table 1.6

Summary of Moments of Inertia and Section Moduli

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Table 1.7

Summary of Reference Moments and Loads

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Table 1.8

Summary of Critical Stresses and Loads

### Bending Girders

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<tr>
<th>Girder</th>
<th>Test</th>
<th>(a)</th>
<th>(\beta)</th>
<th>(k)</th>
<th>(\sigma_{cr}) ksi</th>
<th>(P_{cr}) kips</th>
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<tbody>
<tr>
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<td>82.1</td>
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<td>185</td>
<td>23.9</td>
<td>18.7</td>
<td>82.1</td>
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### Shear Girders

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<th>(\beta)</th>
<th>(k)</th>
<th>(\tau_{cr})</th>
<th>(P_{cr})</th>
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<td>7.12</td>
<td>2.84</td>
<td>27.4</td>
</tr>
<tr>
<td></td>
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<td>0.75</td>
<td>259</td>
<td>13.5</td>
<td>5.38</td>
<td>51.9</td>
</tr>
<tr>
<td></td>
<td>T3</td>
<td>0.50</td>
<td></td>
<td>25.4</td>
<td>10.1</td>
<td>97.6</td>
</tr>
<tr>
<td>G7</td>
<td>T1</td>
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<td>255</td>
<td>3.84</td>
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</tr>
<tr>
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<td>T2</td>
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<td>255</td>
<td>9.34</td>
<td>3.84</td>
<td>37.6</td>
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# Table 1.9

Summary of Critical Stresses and Loads

Girders under Combined Loading

<table>
<thead>
<tr>
<th>Girder</th>
<th>Test</th>
<th>( \alpha )</th>
<th>( \beta )</th>
<th>Bending</th>
<th>Shear</th>
<th>Combined</th>
<th>Pcr</th>
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<td>k</td>
<td>( \sigma_{\text{cri}} ) ksi</td>
<td>k</td>
<td>( \sigma_{\text{cri}} ) ksi</td>
<td>( \sigma_{\text{cr}} ) ksi</td>
<td>( \sigma_{\text{cr}} ) ksi</td>
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<td>7.12</td>
<td>11.1</td>
<td>21.8</td>
<td>21.8</td>
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<td>T1</td>
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Table 1.10
Summary of Girder Deflections
(in inches and under nominal load P=100 kips)

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<th>Girder</th>
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<th>Total</th>
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<td></td>
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<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
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Fig. 1.1: Test Setup of Bending Girders

Fig. 1.2: Test Setup of Shear Girders
Fig. 1.3 Test Setup of Girders Under Combined Loading

Fig. 1.4 Girder Cross Sections
Fig. 1.5 - EVALUATION OF PLATE DIMENSIONS AND COUPON LOCATIONS
**Subject:** Coupon Dimensions and Test Results

<table>
<thead>
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<th>Specimen No:</th>
<th>CP 27</th>
<th>Date:</th>
<th>July 5, 1958</th>
</tr>
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<tr>
<td>Note:</td>
<td>Grosshead Speed 0.10 inches/minute</td>
<td>Tested by: BTY, Ba.</td>
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![Coupon Diagram]

<table>
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<th>Width</th>
<th>Upper Y.P.</th>
<th>Lower Y.P.</th>
<th>Dynamic Y.P.</th>
<th>Static Y.P.</th>
<th>Ult Load</th>
<th>Rupture Load</th>
<th>Reduced Area</th>
<th>Elong. Gage Length</th>
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<td>43,500 lb</td>
<td>73,500 lb</td>
<td>56,400 lb</td>
<td>0.450 in²</td>
<td>10.49 in</td>
</tr>
<tr>
<td>0.770</td>
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<td>46,250</td>
<td>46,000</td>
<td>46,200 lb</td>
<td>43,500 lb</td>
<td>73,500 lb</td>
<td>56,400 lb</td>
<td>0.450 in²</td>
<td>10.49 in</td>
</tr>
<tr>
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<td>46,250</td>
<td>46,000</td>
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<td>43,500 lb</td>
<td>73,500 lb</td>
<td>56,400 lb</td>
<td>0.450 in²</td>
<td>10.49 in</td>
</tr>
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<td>43,500 lb</td>
<td>73,500 lb</td>
<td>56,400 lb</td>
<td>0.450 in²</td>
<td>10.49 in</td>
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<td>0.768</td>
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<td>46,200 lb</td>
<td>43,500 lb</td>
<td>73,500 lb</td>
<td>56,400 lb</td>
<td>0.450 in²</td>
<td>10.49 in</td>
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<td>46,250</td>
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<td>46,200 lb</td>
<td>43,500 lb</td>
<td>73,500 lb</td>
<td>56,400 lb</td>
<td>0.450 in²</td>
<td>10.49 in</td>
</tr>
<tr>
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<td>46,250</td>
<td>46,000</td>
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<td>73,500 lb</td>
<td>56,400 lb</td>
<td>0.450 in²</td>
<td>10.49 in</td>
</tr>
<tr>
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<td>46,200 lb</td>
<td>43,500 lb</td>
<td>73,500 lb</td>
<td>56,400 lb</td>
<td>0.450 in²</td>
<td>10.49 in</td>
</tr>
<tr>
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<td>46,000</td>
<td>46,200 lb</td>
<td>43,500 lb</td>
<td>73,500 lb</td>
<td>56,400 lb</td>
<td>0.450 in²</td>
<td>10.49 in</td>
</tr>
</tbody>
</table>

**Fig. 1.6 - TYPICAL DATA SHEET FOR COUPON TESTS**
Test No. CP 27  Size 1500 x 0.768" Area 152 in²  Yield Point Lbs. Sq. In. 37,600  Ultimate Str. Lbs. Sq. In. 63,800

Elongation In. 7.96" Inches 2.51 in. Per Cent. Elongation 31.5  Per Cent. Reduced Area 60.9%  Date JULY 5, B.T.Y.

Fig. 17 Typical Stress-Strain Curve
Average Thickness = 0.323 in.
Average Diameter = 8.42 in.
Cross Sect. Area, $A = 8.56 \text{ in}^2$
Yield Load $P_y = 300$ kips
Yield Stress $\sigma_y = 35.5 \text{kips}$
$\epsilon = \frac{\delta}{h}$
$\sigma = \frac{P}{A}$

Fig. 1.8 - YIELD STRESS DETERMINATION OF TUBULAR FLANGE