

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

**SPECIFICATION FOR THE
DESIGN, FABRICATION AND ERECTION OF
STRUCTURAL STEEL FOR BUILDINGS**

REVISED 1936

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AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Two years ago the Board of Directors of the American Institute of Steel Construction authorized this Committee to revise the Institute's Standard Specification for the Design, Fabrication and Erection of Structural Steel for Buildings.

The Committee has now completed the task assigned to it, and the Specification presented herewith was officially adopted by the Board, June 24, 1936.

The principal changes are listed just ahead of the Specification proper. Probably the most outstanding is the increase of basic unit stress from 18,000 to 20,000 lbs. per sq. in. The significance of this, as well as of certain other important changes, is explained in an Appendix to the document.

From time to time drafts of the Specification, and of critical portions thereof, have been submitted to members of the Institute's Technical Advisory Board, as well as to other experienced engineers, and their comment has been duly considered in preparing the document in its present form.

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COMMITTEE ON SPECIFICATIONS

PRINCIPAL CHANGES FROM THE PREVIOUS SPECIFICATION

1. The contents have been re-arranged in a manner that should be more convenient for the designer, and also more consistent than heretofore.
2. Certain engineering matter, notably in Section 25, which previously appeared only in the Code of Standard Practice, has been inserted in the Specification.
3. A new series of allowable Unit Stresses are recommended which accord in general with a stress basis of 20,000 instead of 18,000 pounds per square inch in Tension.

The unit values recommended for other kinds of stress, however, have not been always increased in this proportion, each being determined by a separate consideration of its conditions.
4. A marked increase is allowed in Rivet Bearing Values, accompanied however by more severe requirements as to the minimum end or edge distance in the stressed direction. See Sections 10 (a) and 18 (f), and the Appendix.
5. The use of Composite Steel and Concrete Beams is recognized, and their general requirements are specified. See Section 8.
6. Effective Span Length is defined in terms of the conditions of end fixity, and the requirements for the respective end connections are specified. See Sections 9, 16 (d) and 16 (e).
7. In the Design of Plate Girders, the recommended basis for computing moment of inertia is changed from net to gross area. See Sections 14 (a), 14 (b) and 19 (a), and the Appendix.
8. Provision is made for the wider use of Unfinished Bolts in certain heights of structure, and the term "height of building" is defined. See Sections 17 (d) and 17 (e).
9. Provision is made for the elimination of Intermediate Stiffeners in certain classes of plate girders, provided the unit web shear is kept low. In addition, the rules for spacing all Stiffeners are revised and clarified along lines similar to those adopted in the 1935 A. R. E. A. and A. A. S. H. O. Specifications. See Section 19 (e).
10. A new and simplified formula is presented for the Web Crippling of Beams. See Section 19 (h), and the Appendix.
11. Since the use of Hand-driven Rivets has now become obsolete, all reference to them is omitted. Power-driven rivets are defined and are prescribed wherever rivets are specified. See Section 25 (g).
12. The requirements incidental to Field Painting are specified, but the responsibility therefor is considered a matter to be covered by contract. See Section 27 (g).

**SPECIFICATION FOR THE
DESIGN, FABRICATION, AND ERECTION OF
STRUCTURAL STEEL FOR BUILDINGS**

PART I. GENERAL

SECTION 1. SCOPE

- Scope** (a) This Specification defines the practice adopted by the American Institute of Steel Construction in the design, fabrication, and erection of structural steel for buildings.
- Code** (b) In the execution of contracts entered into under this Specification, the Code of Standard Practice for Buildings of the American Institute of Steel Construction shall apply unless otherwise specified or required.

SECTION 2. PLANS AND DRAWINGS

- Plans** (a) The plans shall show a complete design with sizes, sections, and the relative location of the various members. Floor levels, column centers, and off-sets shall be dimensioned. Plans shall be drawn to a scale large enough to convey the information adequately but not less than $\frac{1}{8}$ inch to the foot.
- Shop Drawings** (b) Shop drawings shall be made in conformity with the best modern practice and with due regard to speed and economy in fabrication and erection.

PART II. MATERIAL

SECTION 3. MATERIAL

- Structural Steel** (a) Structural steel shall conform to the Standard Specifications of the American Society for Testing Materials for Steel for Buildings, Serial Designation A 9 (or, if so specified by the Buyer, for Steel for Bridges, Serial Designation A 7), as amended to date.
- Rivet Steel** (b) Rivet steel shall conform to the Standard Specifications of the American Society for Testing Materials for structural Rivet Steel, Serial Designation A 141, as amended to date.
- Other Metals** (c) Alloy steels, cast steel, cast iron and other metals shall conform to the applicable Specifications of the American Society for Testing Materials, as amended to date.
- Stock Material** (d) Stock material shall be of a quality substantially equal to that called for by the specifications of the American Society for Testing Materials for the classifications covering its intended use; and mill test reports shall constitute sufficient record as to the quality of material carried in stock.

Unidentified stock material, if free from surface imperfections, may be used for short sections of minor importance, or for small unimportant details, where the precise physical properties of the material would not affect the strength of the structure.

PART III. LOADS AND STRESSES

SECTION 4. LOADS AND FORCES

- Loads and Forces** (a) Steel structures shall be designed to sustain the following loads and forces:
1. Dead Load
 2. Live Load
 3. Impact
 4. Wind
 5. Erection Loads
 6. Other Forces
- Dead Load** (b) The dead load shall consist of the weight of the steelwork and all material fastened thereto or supported thereby.
- Live Load** (c) The live load shall be that stipulated by the Code under which the structure is being designed or that required by the conditions involved. In general, the live loads should not be less than those recommended by the Building Code Committee of the National Bureau of Standards, November, 1924, under the caption "Minimum Live Loads for use in the Design of Buildings".
- Impact** (d) For structures, carrying live loads inducing impact or vibrational forces, the design stresses shall be increased by a percentage of the live load stresses sufficient to suitably provide for such forces.
- Wind** (e) Proper provision shall be made for stresses caused by wind both during erection and after completion of the building. The wind pressure is dependent upon the conditions of exposure and geographical location of the structure. The allowable stresses specified in Sections 6 (c) and 7 are based upon the steel frame being designed to carry a wind pressure of not less than twenty (20) pounds per square foot on the vertical projection of exposed surfaces during erection, and fifteen (15) pounds per square foot on the vertical projection of the finished structure.
- Erection** (f) Proper provision shall be made for temporary stresses caused by erection.
- Other Forces** (g) Structures in localities subject to earthquakes, hurricanes, and similar extraordinary conditions shall be designed with due regard for such conditions.

SECTION 5. REVERSAL OF STRESS

- Reversal of Stress** (a) Members subject to live loads producing alternate tensile and compressive stresses shall be proportioned as follows:
- To the net total compressive and tensile stresses add 50 per cent of the smaller of the two and proportion the member to resist either of the increased stresses resulting therefrom.
- Connections shall be proportioned to resist the larger of the two increased stresses.

SECTION 6. COMBINED STRESSES

Axial and Bending

(a) Members subject to both axial and bending stresses shall be so proportioned that the quantity

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \text{ shall not exceed unity, in which}$$

F_a = axial unit stress that would be permitted by this Specification if axial stress only existed.

F_b = bending unit stress that would be permitted by this Specification if bending stress only existed.

f_a = axial unit stress (actual) = axial stress divided by area of member.

f_b = bending unit stress (actual) = bending moment divided by section modulus of member.

Rivets

(b) Rivets subject to shearing and tensile forces shall be so proportioned that the combined unit stress will not exceed the allowable unit stress for rivets in tension only.

Wind and Other Forces

(c) Members subject to stresses produced by a combination of wind and other loads may be proportioned for unit stresses 33-1/3 per cent greater than those specified in Section 10, provided the section thus required is not less than that required for the combination of dead load, live load, and impact (if any).

SECTION 7. MEMBERS CARRYING WIND ONLY

Wind Only

(a) Members subject only to stresses produced by wind forces may be proportioned for unit stresses 33-1/3 per cent greater than those specified in Section 10.

SECTION 8. COMPOSITE BEAMS

Composite Beams

(a) The term "composite beam" shall apply to any rolled or fabricated steel floor beam entirely encased in a poured concrete haunch at least four inches wider, at its narrowest point, than the flange of the beam, supporting a concrete slab on each side without openings adjacent to the beam; provided that the top of the beam is at least 1-1/2 inches below the top of the slab and at least 2 inches above the bottom of the slab; provided that a good grade of stone or gravel concrete with Portland cement, is used; and provided that the concrete haunch has adequate mesh, or other reinforcing steel, throughout its whole depth and across its soffit.

(b) Composite beams may be figured on the assumptions that:

1. The steel beam carries unassisted all dead loads prior to the hardening of the concrete, with due regard for any temporary support provided.
2. The steel and concrete carry by joint action all loads, dead and live, applied after the hardening of the concrete.

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**Composite
Beams**

- (c) The total tensile unit stress in the extreme fibre of the steel beam thus computed shall not exceed 20,000 pounds per square inch. [Section 10 (a)].
- (d) The maximum stresses in the concrete, and the ratio of Young's moduli for steel and concrete, shall be as prescribed by the specifications governing the design of reinforced concrete for the structure.
- (e) The web and the end connections of the steel beam shall be adequate to carry the total dead and live load without exceeding the unit stresses prescribed in this Specification, except as this may be reduced by the provision of other proper support.

SECTION 9. EFFECTIVE SPAN LENGTH

Simple Spans

- (a) Beams, girders and trusses shall ordinarily be designed on the basis of simple spans whose effective length is equal to the distance between centers of gravity of the members to which they deliver their end reactions.

End Restraint

- (b) If, on the assumption of end restraint, full or partial, based on continuous or cantilever action, beams, girders and trusses are designed for a shorter effective span length than that specified in Section 9 (a), their sections, as well as the sections of the members to which they connect, shall be designed to carry the shears and moments so introduced, in addition to all other forces, without exceeding at any point the unit stresses prescribed in Section 10.

PART IV. UNIT STRESSES

SECTION 10. ALLOWABLE UNIT STRESSES

All parts of the structure shall be so proportioned that the unit stress in pounds per square inch shall not exceed the following values:

Structural and Rivet Steel

(a) **Structural and Rivet Steel**

Tension

Structural Steel, net section	20,000
Rivets, on area based on nominal diameter.....	15,000

Compression

Columns, gross section For columns with values of l/r not greater than 120.....	$17,000 - 0.485 \frac{l^2}{r^2}$
For columns with values of l/r greater than 120.....	$\frac{18,000}{1 + \frac{l^2}{18,000 r^2}}$

in which l is the unbraced length of the column, and r is the corresponding radius of gyration of the section, both in inches.

Plate Girder Stiffeners, gross section	20,000
Webs of Rolled Sections at toe of fillet [Crippling, see Section 19 (h)].....	24,000

Bending

Tension on extreme fibers of rolled sections, plate girders, and built-up members. [See Section 19 (a)].....	20,000
Compression on extreme fibers of rolled sections, plate girders, and built-up members, for values of l/b not greater than 40.....	$\frac{22,500}{1 + \frac{l^2}{1800 b^2}}$

with a maximum of 20,000

in which l is the unsupported length of the member, and b is the width of the compression flange, both in inches.

Stress on extreme fibers of pins.....	30,000
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Shearing

Rivets, pins, and turned bolts in reamed or drilled holes.....	15,000
Unfinished bolts.....	10,000
Webs of beams and plate girders, gross section.....	13,000

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Structural and Rivet Steel (Continued)

Bearing

	Double Shear	Single Shear
Rivets, and turned bolts in reamed or drilled holes.....	40,000	32,000
Unfinished bolts.....	25,000	20,000
Pins.....	32,000	
Contact Area		
Milled Stiffeners and other Milled Surfaces.....		30,000
Fitted Stiffeners.....		27,000
Expansion rollers and rockers		
(pounds per linear inch).....		600 d
in which d is diameter of roller or rocker in inches.		

Cast Steel

(b) **Cast Steel**
 Compression and Bearing same as for Structural Steel. Other Unit Stresses, 75 per cent of those for Structural Steel.

Masonry

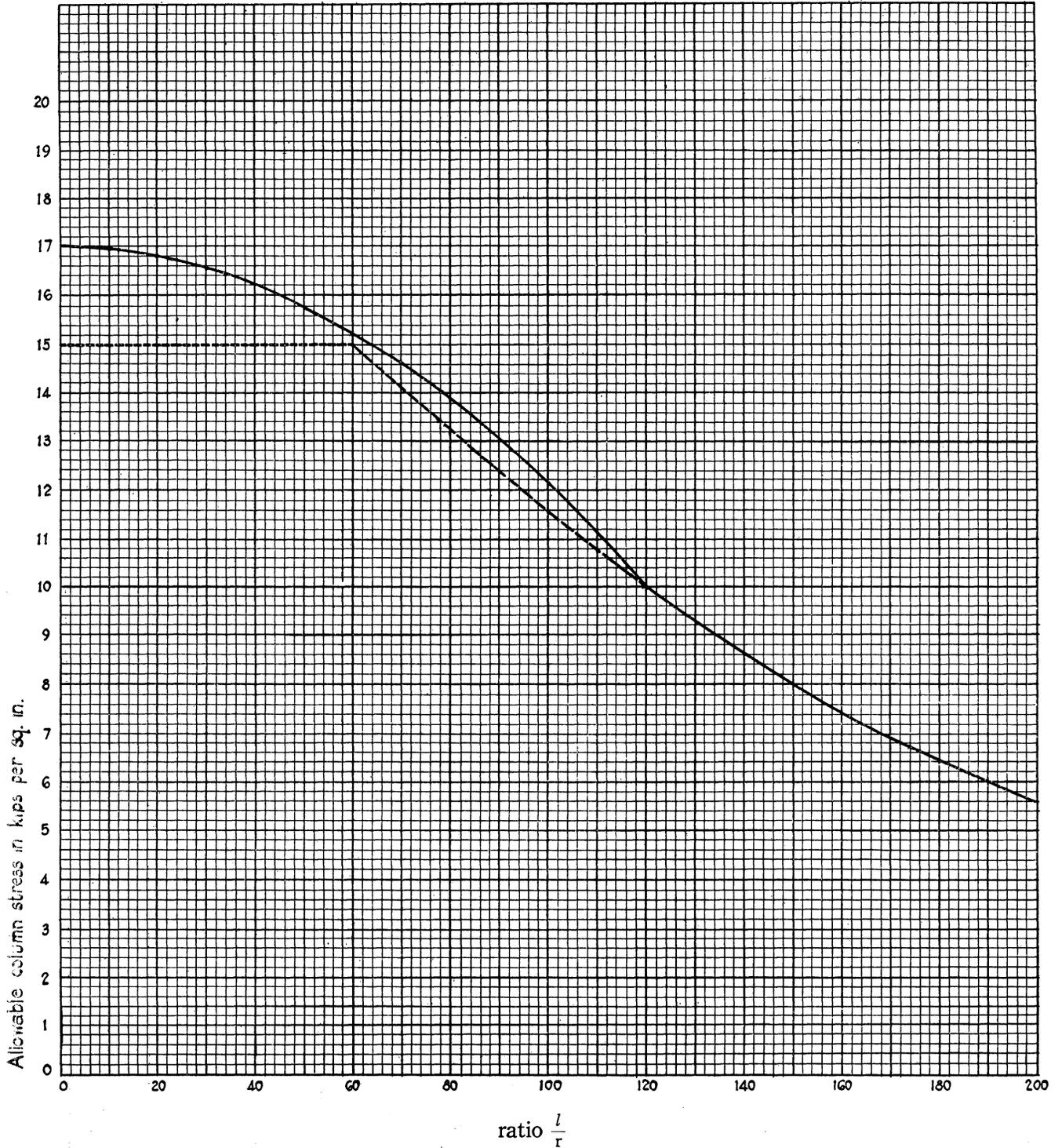
(c) **Masonry [Bearing]**
 Granite..... 800
 Sandstone and Limestone..... 400
 Concrete, unless otherwise specified..... 600

ALLOWABLE UNIT STRESSES								
IN KIPS PER SQUARE INCH FOR								
BEAMS AND GIRDERS								
LATERALLY UNSUPPORTED								
$f = \frac{22500}{1 + \frac{l^2}{1800 b^2}}$			$l =$ Unsupported length in inches. $b =$ Width of flange in inches.					
$\frac{l}{b}$	Unit Stress f	Ratio	$\frac{l}{b}$	Unit Stress f	Ratio	$\frac{l}{b}$	Unit Stress f	Ratio
15.0	20.00	1.000	23.5	17.22	.861	32.0	14.34	.717
15.5	19.85	.993	24.0	17.05	.852	32.5	14.18	.709
16.0	19.70	.985	24.5	16.87	.844	33.0	14.02	.701
16.5	19.54	.977	25.0	16.70	.835	33.5	13.86	.693
17.0	19.39	.969	25.5	16.53	.826	34.0	13.70	.685
17.5	19.23	.961	26.0	16.36	.818	34.5	13.54	.677
18.0	19.07	.953	26.5	16.19	.809	35.0	13.39	.669
18.5	18.91	.945	27.0	16.01	.801	35.5	13.23	.662
19.0	18.74	.937	27.5	15.84	.792	36.0	13.08	.654
19.5	18.58	.929	28.0	15.67	.784	36.5	12.93	.647
20.0	18.41	.920	28.5	15.50	.775	37.0	12.78	.639
20.5	18.24	.912	29.0	15.34	.767	37.5	12.63	.632
21.0	18.07	.904	29.5	15.17	.758	38.0	12.49	.624
21.5	17.90	.895	30.0	15.00	.750	38.5	12.34	.617
22.0	17.73	.887	30.5	14.84	.742	39.0	12.20	.610
22.5	17.56	.878	31.0	14.67	.733	39.5	12.05	.603
23.0	17.39	.869	31.5	14.50	.725	40.0	11.91	.596

ALLOWABLE WORKING STRESS FOR COLUMNS

NEW COLUMN FORMULA FOR COLUMNS WITH VALUES OF $\frac{l}{r}$ NOT GREATER THAN 120.

$$17,000 - 0.485 \frac{l^2}{r^2}$$



Previous column formula curve $\frac{18,000}{1 + \frac{l^2}{18,000 r^2}}$ shown in dotted line. For values of $\frac{l}{r}$ greater than 120 the two curves coincide.

PART V. DESIGN

SECTION 11. SLENDERNESS RATIO

- $\frac{l}{r}$
- (a) The ratio of unbraced length to least radius of gyration (l/r) for compression members shall not exceed:
- | | |
|--|-----|
| For main compression members..... | 120 |
| For bracing and other secondary members in compression.. | 200 |

SECTION 12. UNSUPPORTED COMPRESSION FLANGES

- $\frac{l}{b}$
- (a) The ratio of unbraced length to width of flange (l/b) for compression flanges of rolled sections, plate girders, and built-up members subject to bending shall not exceed 40.

SECTION 13. MINIMUM THICKNESS OF MATERIAL

- Main Material** (a) The minimum thickness of steel except for linings, fillers, and the webs of rolled beams and channels, shall be: for exterior construction—5/16 inch; for interior construction—1/4 inch. (These provisions do not apply to light structures such as skylights, marquees, fire-escapes, light one-story buildings, or light miscellaneous steelwork.)
- Gusset Plates** (b) Gusset plates for trusses with end reactions greater than 35,000 pounds shall be not less than 3/8 inch thick.
- Compression Members** (c) In compression members consisting of segments connected by cover plates or lacing, or segments connected by webs, the thickness of the webs of the segments shall be not less than 1/32 of the unsupported distance between the nearest rivet lines, or the roots of the flanges in case of rolled sections. The thickness of the cover plates or webs connecting the segments shall be not less than 1/40 of the unsupported distance between the nearest lines of their connecting rivets, or the roots of their flanges in case of rolled sections.
- Corrosion** (d) Provision shall be made for parts subject to corrosive agents, either by increasing the thickness of material or by effective protection.

SECTION 14. GROSS AND NET SECTIONS

- Definitions** (a) The gross section of a member at any point shall be determined by summing the products of the thickness and the gross width of each element as measured normal to the axis of the member. The net section shall be determined by substituting for the gross width, the net width computed in accordance with paragraphs (c) to (g) of this Section.
- Application** (b) Unless otherwise specified, tension members shall be designed on the basis of net section. Columns shall be designed on the basis of gross section. Beams and girders shall be designed in accordance with Section 19 (a).

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- Net Width** (c) In the case of a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the chain, and adding, for each gage space in the chain, the quantity $\frac{s^2}{4g}$, where
- s=longitudinal spacing (pitch) of any two successive holes.
g=transverse spacing (gage) of the same two holes.
- The critical net section of the part is obtained from that chain which gives the least net width.
- Angles** (d) For angles, the gross width shall be the sum of the widths of the legs less the thickness. The gage for holes in opposite legs shall be the sum of the gages from back of angle less the thickness.
- Splice Members** (e) For splice members, the thickness shall be only that part of the thickness of the member which has been developed by rivets beyond the section considered.
- Size of Holes** (f) In computing net area the diameter of a rivet hole shall be taken as 1/8 inch greater than the nominal diameter of the rivet.
- Pin Holes** (g) In pin connected tension members, the net section across the pin hole, transverse to the axis of the member, shall be not less than 140 per cent, and the net section beyond the pin hole, parallel with the axis of the member, not less than 100 per cent, of the net section of the body of the member.
- In all pin-connected riveted members the net width across the pin hole, transverse to the axis of the member, shall preferably not exceed 12 times the thickness of the member at the pin.

SECTION 15. EXPANSION

- Expansion** (a) Proper provision shall be made for expansion and contraction.

SECTION 16. CONNECTIONS

- Minimum Connections** (a) Connections carrying calculated stresses, except for lacing, sag bars, and girts, shall have not fewer than 2 rivets.
- Eccentric Connections** (b) Members meeting at a point shall have their gravity axes meet at a point if practicable; if not, provision shall be made for their eccentricity.
- Rivets** (c) The rivets at the ends of any member transmitting stresses into that member should preferably have their centers of gravity on the gravity axis of the member; otherwise, provision shall be made for the effect of the resulting eccentricity. Pins may be so placed as to counteract the effect of bending due to dead load.
- Unrestrained Members** (d) When beams, girders or trusses are designed on the basis of simple spans in accordance with Section 9 (a), their end connections may ordinarily be designed for the reaction shears only. If, however, the eccentricity of the connection is excessive, provision shall be made for the resulting moment.

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- Restrained Members** (e) When beams, girders or trusses are subject both to reaction shear and end moment, due to the restraint specified in Section 9 (b), their connections shall be specially designed to carry both shear and moment without exceeding at any point the unit stresses prescribed in Section 10. Ordinary end connections comprising only a pair of web angles, with not more than a nominal seat and top angle, shall not be assumed to provide for this kind of end moment.
- Fillers** (f) In truss construction when rivets carrying computed stress pass through fillers, the fillers shall be extended beyond the connected member and the extension secured by sufficient rivets to develop the strength of the filler.
- Fillers under plate girder stiffeners at end bearings or points of concentrated loads shall be secured by sufficient rivets to prevent excessive bending and bearing stresses.
- Splices** (g) Compression members when faced for bearings shall be spliced sufficiently to hold the connecting members accurately in place. Other joints in riveted work, whether in tension or compression, shall be spliced so as to transfer the stress to which the member is subject.

SECTION 17. RIVETS AND BOLTS

- Diameter** (a) In proportioning rivets, the nominal diameter of the rivet shall be used.
- Effective Bearing Area** (b) The effective bearing area of pins, bolts, and rivets shall be the diameter multiplied by the length in bearing; except that for countersunk rivets, half the depth of the countersunk shall be deducted.
- Long Grips** (c) Rivets carrying calculated stress, and whose grip exceeds five diameters, shall have their number increased 1 per cent for each additional 1/16 inch in the rivet grip. Special care shall be used in heating and driving such rivets.
- Use of Unfinished Bolts** (d) All field connections may be made with unfinished bolts, except as provided in Par. (e) hereof.
- Use of Rivets** (e) Rivets shall be used for the following connections:
- Connections for supports of running machinery, or of other live loads which produce impact or reversal.
- Column splices in all tier structures 200 feet or more in height.
- Column splices in tier structures 100 to 200 feet in height, if the least horizontal dimension is less than 40 per cent of the height.
- Column splices in tier structures less than 100 feet in height, if the least horizontal dimension is less than 25 per cent of the height.
- Connections of all beams and girders to columns, and of any other beams and girders on which the bracing of columns is dependent, in structures over 125 feet in height.

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Use of Rivets (continued)

Roof-truss splices and connections of trusses to columns, column splices, column bracing, and crane supports, in all structures carrying cranes of over 5-ton capacity.

Any other connections stipulated on the design plans.

The height of a tier structure shall be taken as the vertical distance from the curb level to the highest point of the roof beams, in the case of flat roofs, or to the mean height of the gable, in the case of roofs having a rise of more than one to a $4\frac{1}{2}$ run. Where the curb level has not been established, or where the structure does not adjoin a street, the mean level of the adjoining land shall be used instead of curb level. Penthouses may be excluded in computing the height of a structure.

Use of Turned Bolts

(f) Turned bolts in reamed or drilled holes, as specified in Section 25 (d), may be used in shop or field work where it is impossible to drive satisfactory rivets. The finished shank shall be long enough to provide full bearing, and washers shall be used under the nuts to give full grip when the nuts are turned tight.

Main Members

(g) The end reaction stresses of trusses, girders, or beams, and the axial stresses of tension or compression members which are carried on rivets or bolts, shall have such stresses developed by the shearing and bearing values of the rivets or bolts.

SECTION 18. RIVET SPACING

Minimum Pitch

(a) The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than $4\frac{1}{2}$ inches for $1\frac{1}{4}$ inch rivets, 4 inches for $1\frac{1}{8}$ inch rivets, $3\frac{1}{2}$ inches for 1 inch rivets, 3 inches for $\frac{7}{8}$ inch rivets, $2\frac{1}{2}$ inches for $\frac{3}{4}$ inch rivets, 2 inches for $\frac{5}{8}$ inch rivets, and $1\frac{3}{4}$ inches for $\frac{1}{2}$ inch rivets.

Maximum Pitch Compression Members

(b) The maximum pitch in the line of stress of compression members composed of plates and shapes shall not exceed 16 times the thickness of the thinnest outside plate or shape, nor 20 times the thinnest enclosed plate or shape with a maximum of 12 inches, and at right angles to the direction of stress the distance between lines of rivets shall not exceed 30 times the thickness of the thinnest plate or shape. For angles in built sections with two gage lines, with rivets staggered, the maximum pitch in the line of stress in each gage line shall not exceed 24 times the thickness of the thinnest plate with a maximum of 18 inches.

End Pitch Compression Members

(c) The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets for a length equal to $1\frac{1}{2}$ times the maximum width of the member.

Two-Angle Members

(d) In tension members composed of two angles, a pitch of 3'-6" will be allowed, and in compression members, 2'-0", but the ratio l/r for each angle between rivets shall be not more than $\frac{3}{4}$ of that for the whole member.

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Minimum Edge Distance

(e) The minimum distance from the center of any punched rivet hole to any edge shall be that given in Table I.

TABLE 1

Rivet Diameter, Inches	Minimum Edge Distance (Inches) for Punched Holes		
	In Sheared Edge	In Rolled Edge of Plates and Sections with Parallel Flanges	In Rolled Edge of Sections with Sloping Flanges
1/2	1	7/8	3/4*
5/8	1 1/8	1	7/8*
3/4	1 1/4	1 1/8	1 *
7/8	1 1/2	1 1/4	1 1/8*
1	1 3/4	1 1/2	1 1/4*
1 1/8	2	1 3/4	1 1/2*
1 1/4	2 1/4	2	1 3/4*

*May be decreased 1/8 inch when holes are near end of beam.

Minimum Edge Distance in Line of Stress

(f) The distance from the center of any rivet under computed stress, and that end or other boundary of the connected member toward which the pressure of the rivet is directed, shall be not less than the shearing area of the rivet shank (single or double shear respectively) divided by the plate thickness.

This end distance may however be decreased in such proportion as the stress per rivet is less than that permitted under Section 10 (a); and the requirement may be disregarded in case the rivet in question is one of three or more in a line parallel to the direction of stress.

Maximum Edge Distance

(g) The maximum distance from the center of any rivet to the near edge shall be 12 times the thickness of the plate, but shall not exceed 6 inches.

SECTION 19. PLATE GIRDERS AND ROLLED BEAMS

Proportioning

(a) Riveted plate girders, cover-plated beams, and rolled beams shall in general be proportioned by the moment of inertia of the gross section. No deduction shall be made for standard shop or field rivet holes in either flange; except that in special cases where the reduction of the area of either flange by such rivet holes, calculated in accordance with the provisions of Section 14, exceeds 15 per cent of the gross flange area, the excess shall be deducted. If such members contain other holes, as for bolts, pins, or countersunk rivets, the full deduction for such holes shall be made. The deductions thus applicable to either flange shall be made also for the opposite flange if the corresponding holes are there present.

Web

(b) Plate girder webs shall have a thickness of not less than 1/170 of the unsupported distance between flanges.

Flanges

(c) Cover plates, when required, shall be equal in thickness or shall diminish in thickness from the flange angles outward. No plate shall be thicker than the flange angles.

Unstiffened cover plates shall not extend more than 6 inches nor more than 12 times the thickness of the thinnest plate beyond the outer row of rivets connecting them to the angles.

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Rivets (d) Rivets connecting the flanges to the web shall be proportioned to resist the horizontal shear due to bending as well as any loads applied directly to the flange.

Stiffeners (e) Stiffeners shall be placed on the webs of plate girders at the ends and at points of concentrated loads. Such stiffeners shall have a close bearing against the flanges, shall extend as closely as possible to the edge of the flange angles, and shall not be crimped. They shall be connected to the web by enough rivets to transmit the stress. Only that portion of the outstanding legs outside of the fillets of the flange angles shall be considered effective in bearing.

Intermediate stiffeners shall be required at all points where h/t exceeds $\frac{8,000}{\sqrt{s}}$, in which

h = the clear depth between flanges, in inches

t = the thickness of the web, in inches

s = greatest unit shear in panel, in pounds per square inch under any condition of complete or partial loading.

The clear distance between intermediate stiffeners, when stiffeners are required by the foregoing, shall not exceed 84 inches or that given by the formula

$$d = \frac{270,000t}{s} \sqrt[3]{\frac{st}{h}}$$

d = the clear distance between stiffeners, in inches.

Intermediate stiffeners may be crimped over the flange angles.

Plate girder stiffeners shall be in pairs, one on each side of the web, and shall be connected to the web by rivets spaced not more than 8 times their nominal diameter.

Splices (f) Web splices in plate girders shall be proportioned to transmit the full shearing and bending stresses in the web at the point of splice.

Flange splices shall be proportioned to develop the full strength of the members cut.

Lateral Forces (g) The flanges of plate girders supporting cranes or other moving loads shall be proportioned to resist any lateral forces produced by such loads.

Web Crippling of Beams (h) Rolled beams shall be so proportioned that the compression stress at the web toe of the fillets, resulting from concentrated loads, shall not exceed the value of 24,000 pounds per square inch allowed in Section 10 (a). The governing formulas shall be:

For interior loads $\frac{R}{t(A + 2N)} = \text{not over } 24,000$

For end reactions $\frac{R}{t(A + N)} = \text{not over } 24,000$

R = concentrated interior load or end reactions, in pounds.

t = thickness of web, in inches.

A = length of bearing, in inches.

N = distance from outer face of flange to web toe of fillet, in inches.

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SECTION 20. TIE PLATES

- Compression Members** (a) The open sides of compression members shall be provided with lacing having tie plates at each end, and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In main members carrying calculated stresses the end tie plates shall have a length of not less than the distance between the lines of rivets connecting them to the segments of the member, and intermediate ones of not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between the lines of rivets connecting them to the segments of the members, and the rivet pitch shall be not more than four diameters. Tie plates shall be connected to each segment by at least three rivets.
- Tension Members** (b) Tie plates shall be used to secure the parts of tension members composed of shapes. They shall have a length not less than two-thirds of the length specified for tie plates in compression members. The thickness shall be not less than one-fiftieth of the distance between the lines of rivets connecting them to the segments of the member and they shall be connected to each segment by at least three rivets.

SECTION 21. LACING

- Spacing** (a) Lacing bars of compression members shall be so spaced that the ratio l/r of the flange included between their connections shall be not over $\frac{3}{4}$ of that of the member as a whole.
- Proportioning** (b) Lacing bars shall be proportioned to resist a shearing stress normal to the axis of the member equal to two per cent of the total compressive stress in the member. In determining the section required the compression formula shall be used, l being taken as the length of the bar between the outside rivets connecting it to the segment for single lacing and 70 per cent of that distance for double lacing. The ratio l/r shall not exceed 140 for single lacing nor 200 for double lacing.
- Minimum Proportions** (c) The thickness of lacing bars shall be not less than one-fortieth for single lacing, and one-sixtieth for double lacing, of the distance between end rivets; their minimum width shall be three times the diameter of the rivets connecting them to the segments.
- Inclination** (d) The inclination of lacing bars to the axis of the members shall preferably be not less than 45 degrees for double lacing and 60 degrees for single lacing. When the distance between the rivet lines in the flanges is more than 15 inches, the lacing shall be double and riveted at the intersection if bars are used, or else shall be made of angles.

SECTION 22. ADJUSTABLE MEMBERS

- Initial Stress** (a) The total initial stress in adjustable members shall be assumed as not less than 5,000 pounds.

SECTION 23. COLUMN BASES

- Loads** (a) Proper provision shall be made to transfer the column loads, and moments if any, to the footings and foundations.
- Alignment** (b) Column bases shall be set level and to correct elevation with full bearing on the masonry.
- Finishing** (c) Column bases shall be finished to accord with the following requirements:
1. Rolled steel bearing plates, 2'' or less in thickness, may be used without planing, provided a satisfactory contact bearing is obtained; rolled steel bearing plates, over 2'', but 4'' or less in thickness, may be straightened by pressing (planed on all bearing surfaces if presses are not available) to obtain a satisfactory contact bearing; rolled steel bearing plates, over 4'' in thickness, shall be planed on all bearing surfaces (except as noted under 3).
 2. Column bases other than rolled steel bearing plates shall be planed on all bearing surfaces (except as noted under 3).
 3. The bottom surfaces of bearing plates and column bases which rest on masonry foundations and are grouted to insure full bearing contact need not be planed.

SECTION 24. ANCHOR BOLTS

- Anchor Bolts** (a) Anchor bolts shall be of sufficient size and number to develop the computed stress.

PART VI. FABRICATION

SECTION 25. WORKMANSHIP

- General** (a) All workmanship shall be equal to the best practice in modern structural shops.
- Straightening** (b) All material shall be clean and straight. If straightening or flattening is necessary, it shall be done by a process and in a manner that will not injure the material. Sharp kinks or bends shall be cause for rejection.
- Heating** (c) Rolled sections, except for minor details, shall preferably not be heated or, if heated, shall be annealed, but this restriction does not apply to gas cutting, Section 25 (1).
- Holes** (d) Holes for rivets or unfinished bolts shall be 1/16 inch larger than the nominal diameter of the rivet or bolt. If the thickness of the material is not greater than the nominal diameter of the rivet or bolt plus $\frac{1}{8}$ inch, the holes may be punched. If the thickness of the material is greater than the nominal diameter of the rivet or bolt plus $\frac{1}{8}$ inch, the holes shall be either drilled from the solid, or sub-punched and reamed. The die for all sub-punched holes, and the drill for all sub-drilled holes, shall be 1/16 inch smaller than the nominal diameter of the rivet or bolt.
- Holes for turned bolts shall be 1/50 inch larger than the external diameter of the bolt. If the bolts are to be inserted in the shop, the holes may be either drilled from the solid, or sub-punched and reamed. If the bolts are to be inserted in the field, the holes shall be sub-punched in the shop and reamed in the field. All drilling or reaming for turned bolts shall be done after the parts to be connected are assembled.
- Drifting to enlarge unfair holes shall not be permitted. Holes that must be enlarged to admit the rivets shall be reamed. Poor matching of holes shall be cause for rejection.
- Planing** (e) Planing or finishing of sheared plates or shapes will not be required unless specifically called for on the drawings.
- Assembling** (f) All parts of riveted members shall be well pinned or bolted and rigidly held together while riveting. Drifting done during assembling shall not distort the metal or enlarge the holes.
- Riveting** (g) All rivets are to be power-driven hot. Rivets driven by pneumatically or electrically operated hammers are considered power-driven. Standard rivet heads shall be of approximately hemispherical shape and of uniform size throughout the work for the same size rivet, full, neatly finished, and concentric with the holes. Rivets after driving, shall be tight, completely filling the holes, and with heads in full contact with the surface. Rivets shall be heated uniformly to a temperature not exceeding 1950° F.; they shall not be driven after their temperature is below 1000° F.
- Loose, burned, or otherwise defective rivets shall be replaced.
- Finishing** (h) Compression joints depending upon contact bearing shall have the bearing surfaces truly machined to a common plane after the members are riveted. All other joints shall be cut straight.

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- Lacing Bars** (i) The ends of lacing bars shall be neat and free from burrs.
- Tolerances** (j) Finished members shall be true to line and free from twists, bends, and open joints.
- Compression members may have a lateral variation not greater than $1/1000$ of the axial length between points which are to be laterally supported.
- A variation of $1/32$ inch is permissible in the overall length of members with both ends milled.
- Members without milled ends which are to be framed to other steel parts of the structure may have a variation from the detailed length not greater than $1/16$ inch for members 30 feet or less in length, and not greater than $1/8$ inch for members over 30 feet in length.
- Castings** (k) All steel castings shall be annealed.
- Gas Cutting** (l) The use of a cutting torch is permissible if the metal being cut is not carrying stresses during the operation. To determine the effective width of members so cut, $1/8$ inch shall be deducted from each gas cut edge. The radius of re-entrant gas cut fillets shall be as large as practicable, but never less than 1 inch.

SECTION 26. SHOP PAINTING

- Shop Coat** (a) Before leaving the shop, all steel work shall be thoroughly cleaned, by effective means, of all loose mill scale, rust and foreign matter. Except where encased in concrete, all steel work shall be given one coat of approved metal protection, applied thoroughly and evenly and well worked into the joints and other open spaces. All paint shall be applied to dry surfaces.
- Inaccessible Parts** (b) Parts inaccessible after assembly shall be given two coats of shop paint, preferably of different colors.
- Contact Surfaces** (c) Contact surfaces shall be cleaned, by effective means, before assembly, but not painted.
- Finished Surfaces** (d) Machine finished surfaces shall be protected against corrosion. by a suitable coating.

PART VII. ERECTION

SECTION 27. ERECTION

- Bracing** (a) The frame of all steel skeleton buildings shall be carried up true and plumb, and temporary bracing shall be introduced wherever necessary to take care of all loads to which the structure may be subjected, including erection equipment, and the operation of same. Such bracing shall be left in place as long as may be required for safety.
- Bolting Up** (b) As erection progresses the work shall be securely bolted up to take care of all dead load, wind and erection stresses.
- Erection Stresses** (c) Wherever piles of material, erection equipment or other loads are carried during erection, proper provision shall be made to take care of stresses resulting from the same.
- Alignment** (d) No riveting shall be done until the structure has been properly aligned.
- Riveting** (e) Rivets driven in the field shall be heated and driven with the same care as those driven in the shop.
- Turned Bolts** (f) Holes for turned bolts to be inserted in the field shall be reamed in the field as specified in Section 25 (d).
- Field Painting** (g) All field rivets and bolts, also all serious abrasions to the shop coat, shall be spot painted with the material used for the shop coat, or an equivalent, and all mud and other firmly attached and objectionable foreign materials shall be removed, before general field painting.

Responsibility for this touch-up and cleaning, as well as for general field painting, shall be allocated in accordance with accepted local practices and this allocation shall be set forth explicitly in the contract.

PART VIII. INSPECTION

SECTION 28. INSPECTION

- General** (a) Material and workmanship at all times shall be subject to the inspection of experienced engineers representing the purchaser.
- Cooperation** (b) All inspection as far as possible shall be made at the place of manufacture, and the Contractor or Manufacturer shall cooperate with the Inspector, permitting access for inspection to all places where work is being done.
- Rejections** (c) Material or workmanship not conforming to the provisions of this Specification may be rejected at any time defects are found during the progress of the work.

APPENDIX

EXPLANATORY NOTES ON SECTIONS

10 (a), 18 (f), 19 (a) AND 19 (h) OF THE SPECIFICATION

INCREASE IN ALLOWABLE BASIC UNIT STRESS SECTION 10 (a)

Eleven years ago, a distinguished committee appointed by the American Society of Civil Engineers recommended in a majority report that the basic unit stress (in tension), to be used in structural steel design, be increased from 16,000 to 20,000 pounds per square inch. See A. S. C. E. Proceedings for March, 1925.

With a view to conservatism, the first Specification of the American Institute of Steel Construction prescribed a basic unit stress of only 18,000 pounds. At that time the maximum tensile strength of the A. S. T. M. steel, whose use was contemplated in the Specification, was 65,000 pounds.

Since that time several changes have occurred that affect the situation:—

- (1) The maximum tensile strength of the A. S. T. M. steel in question has been raised to 72,000 pounds.
- (2) The principal foreign countries of the world have increased the allowable basic unit stress permitted for comparable steel to approximately 20,000 pounds.
- (3) Knowledge and current practice in the structural steel industry have greatly improved.

In the year 1934, the A. I. S. C. Committee, which was appointed to revise the Specification, canvassed 21 members of the Institute's Technical Advisory Board on the advisability of increasing the basic unit stress in the A. I. S. C. Specification to 20,000 pounds. Of this number, 12 favored the increase without qualification, 5 favored it with certain restrictions, and 4 were opposed to the increase.

After considering all the facts available, the Committee decided to recommend the use of 20,000 pounds coupled, however, with more rigid requirements as to a number of detailed features, and these requirements have now been incorporated in the revised Specification. The Committee believes the net result will prove to be economical, and at the same time entirely safe.

For columns having slenderness ratios up to 120 (the range in which the increased strength of material governs) the Committee has followed the example of the A. R. E. A. and the A. A. S. H. O. Specifications in approximating the accepted but cumbersome secant formula with a convenient parabolic formula; while above 120 (where modulus of elasticity governs) it has retained the original A. I. S. C. formula. This procedure allows a higher unit stress than formerly on short columns, but not on long ones.

For convenient reference there is given on page 8 a diagram of the column curve, and a table of allowable column values. A table of allowable values for the compression flanges of members subject to bending is given on Page 7.

BEARING PRESSURE ON RIVETS SECTIONS 10 (a) AND 18 (f)

Hitherto it has been the practice in American specifications to allow a rivet bearing pressure of twice the intensity of the allowable rivet shear. Knowing the German building specifications allow over three times, instead of twice, a search was made of the German test literature and their ratio was found to be fully warranted by their test data.

As a verification, a brief series of tests of double-shear single rivet specimens was conducted in one of the Bethlehem Steel Company fabricating shops, using constant unit tension in the plates and constant unit shear in the rivets, and varying the unit bearing only.

These tests showed that, with allowable rivet shear at 15,000 pounds per square inch and allowable rivet bearing at 40,000, those thicknesses which would be controlled in design by double shear would have factors of safety of 3.2 or more at joint yield point and 4.0 or more at joint failure, while those lesser thicknesses which would be controlled in design by bearing would have factors of safety of 3.2 or more at joint yield point and of 3.85 or more at joint failure. Certain tests were stopped at something over yield point but before breaking and, when cut apart for examination, these specimens showed no distress from bearing. No failure was a bearing failure per se.

The complete test data were submitted to the Committee, to the Technical Advisory Board, and to other interested engineers, and the increase of bearing pressure given general approval. The higher values have therefore been incorporated in Section 10 (a) of the Specification.

One interesting fact brought out by the test was, that the thinnest specimens failed by shearing a wedge-shaped piece out of the end of the bar. This action would be prevented, and the tensile value of the bar developed, by increasing the end distance beyond the rivet. Had the specimens contained several rivets in line, this should not have occurred, as the yielding of the end of the bar would no doubt have thrown more load back onto the interior rivets. Since there are structural connections in which this type of premature failure might control the design, and since such failure may sometimes be prevented by increase of thickness and in other cases by adding more rivets (two different means of reducing the shear behind a rivet), the Committee has added the provision contained in Section 18 (f).

DESIGN OF PLATE GIRDERS, SECTION 19 (a)

The use of gross area in the computation of the moment of inertia of plate girders appears to reflect the actual behavior of such members much more closely than does the use of net area.

Thus, in determining the anticipated deflection of large steel bridges, for purposes of erection, it has long been the established practice to compute gross rather than net areas, and the measured deformation of such structures under load has been found to accord with this basis of computation.

Practically all engineers are agreed that well-designed plate girders in service are much stronger than has been assumed, and realize that in those rare instances where their capacity was exceeded, failure occurred by buckling in the compression flange or in the web, rather than in the tension flange.

Various explanations have been offered for the apparent absence of weakness at rivet holes in the tension flange. Some investigators have attributed it to the large amount of stress, within working limits, that is transferred by friction and through the rivet heads, thereby compensating in part for the loss of metal in the holes of tension elements. Tensile tests recently made at Swarthmore College but not yet published show a surprisingly uniform distribution of stress on the gross section midway between open holes and midway between rivets. This would indicate that the gross section should be used notwithstanding the fact that high concentration of stress exists and always has existed around rivets. Further tests are being conducted at Swarthmore to determine what, if any, coefficients should be applied to the working unit stress to take care of such concentrations when using the gross moment of inertia. In the case of the plate girder some consider the fact that stress is being transferred from web to flange in continuous increments, differentiates this type of member from simple tension members.

While recognizing the desirability of a complete theoretical explanation of this most satisfactory situation, the committee which framed this Specification was more concerned with a means of securing therefrom practical benefits to the industry. To this end, the Specification sanctions the use of gross area in the design of riveted plate girders. The saving in weight which is attainable in many cases is of greatest importance. An incidental advantage lies in the reduced time and labor required for computation.

The former requirement, that the gross section of the compression flange shall be not less than that of the tension flange, is omitted because this condition cannot occur under the revised Specification.

Prior to putting this Specification into print the opinion of a number of prominent engineers was solicited as to the propriety of designing plate girders on the basis of gross sectional area as specified in Section 19 (a). Almost all agreed that the former practice of assuming only the net area to be effective in the tension flange penalizes the plate girder too much, and also it is impracticable to make computations in strict conformity with this assumption.

Some of those consulted thought that Section 19 (a) was too liberal, however, especially in those cases where the flange is composed of a large number of plates and in which the area of the holes is a large percentage of the total flange area and they suggested various rules for taking care of such conditions. Others considered that an optional method might be permitted whereby the basic unit stress might be applied to the net section, while a somewhat lower stress would be permitted if applied to the gross section.

The publication of the Specification as a separate pamphlet will afford opportunities for all engaged in the design or fabrication of structural steel to consider this important subject and the Institute will welcome constructive comment so that, if necessary, the matter may be reconsidered prior to the incorporation of the Specification in the next edition of the Manual. It is also hoped that by that time the experimental investigations now in progress may throw additional light on the subject.

WEB CRIPPLING OF BEAMS, SECTION 19 (h)

Previous editions of the Specification purported to provide against web failure by applying a compression formula, based on the mid-height of the web, to a length of web dependent upon the beam depth. A series of experiments, culminating with those made at Lehigh University and reported in the Proceedings of the American Society of Civil Engineers, November, 1934, pages 1354 to 1359, demonstrated that beams, having a ratio of depth to web thickness not greater than that of any listed in the A. I. S. C. Manual, will fail by crippling at the web toe of the fillets before the compression effect that was formerly assumed will come into play.

For this reason the old criterion for bearing length has been replaced by one based on web strength at the toe of the fillets. In the case of the deeper beams, the new basis of design requires a greater bearing length than was previously specified, but not greater than that which a number of experienced engineers have always incorporated into their practice.